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ATC 40

Seismic evaluation and retrofit of concrete buildings

Volume 1

ATC Applied Technology Council

CALIFORNIA SEISMIC SAFETY COMMISSION
Proposition 122 Seismic Retrofit Practices Improvement Program
Report SSC 96-01

Applied Technology Council

The Applied Technology Council (ATC) is a nonprofit, tax-exempt corporation established in 1971 through the efforts of the Structural Engineers Association of California. ATC is guided by a Board of Directors consisting of representatives appointed by the American Society of Civil Engineers, the Structural Engineers Association of California, the Western States Council of Structural Engineers Associations, and four at-large representatives concerned with the practice of structural engineering. Each director serves a three-year term.

The purpose of ATC is to assist the design practitioner in structural engineering (and related design specialty fields such as soils, wind, and earthquake) in the task of keeping abreast of and effectively using technological developments. ATC also identifies and encourages needed research and develops consensus opinions on structural engineering issues in a nonproprietary format. ATC thereby fulfills a unique role in funded information transfer.

Project management and administration are carried out by a full-time Executive Director and support staff. Project work is conducted by a wide range of highly qualified consulting professionals, thus incorporating the experience of many individuals from academia, research, and professional practice who would not be available from any single organization. Funding for ATC projects is obtained from government agencies and from the private sector in the form of tax-deductible contributions.

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The California Seismic Safety Commission consists of fifteen members appointed by the Governor and two members representing the State Senate and State Assembly. Disciplines represented on the Commission include seismology, engineering, geology, fire protection, emergency services, public utilities, insurance, social services, local government, building code enforcement, planning and architecture.

As a nonpartisan, single-purpose body, the mission of the Commission is to improve the well being of the people of California through cost-effective measures that lower earthquake risks to life and property. It sponsors legislation and advocates building code changes to improve buildings and other facilities, provides a forum for representatives of all public and private interests and academic disciplines related to earthquakes, and publishes reports, policy recommendations, and guides to improve public safety in earthquakes.

It works toward long-term improvements in all areas affecting seismic safety by: encouraging and assisting local governments, state agencies, and businesses to implement mitigation measures to make sure that they will be able to operate after earthquakes; establishing priorities for action to reduce earthquake risks; identifying needs for earthquake education, research, and legislation; and reviewing emergency response, recovery, and reconstruction efforts after damaging earthquakes so that lessons learned can be applied to future earthquakes.

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ATC-40

Seismic Evaluation and Retrofit of Concrete Buildings

Volume 1

by

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SEISMIC SAFETY COMMISSION
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Preface

Proposition 122 passed by California's voters in 1990, created the Earthquake Safety and Public Buildings Rehabilitation Fund of 1990, supported by a $300 million general obligation bond program for the seismic retrofit of state and local government buildings. As a part of the program, Proposition 122 authorizes the California Seismic Safety Commission (CSSC) to use up to 1% of the proceeds of the bonds, or approximately $3 million, to carry out a range of activities that will capitalize on the seismic retrofit experience in the private sector to improve seismic retrofit practices for government buildings. The purpose of California's Proposition 122 research and development program is to develop state-of-the-practice recommendations to address current needs for seismic retrofit provisions and seismic risk decision tools. It is focused specifically on vulnerable concrete structures consistent with the types of concrete buildings that make up a significant portion of California's state and local government inventories.

In 1994, as part of the Proposition 122 Seismic Retrofit Practices Improvement Program, the Commission awarded the Applied Technology Council (ATC) a contract to develop a recommended methodology and commentary for the seismic evaluation and retrofit of existing concrete buildings (Product 1.2). In 1995 the Commission awarded a second, related contract to ATC to expand the Product 1.2 effort to include effects of foundations on the seismic performance of existing concrete buildings (Product 1.3). The results of the two projects have been combined and are presented in this ATC-40 Report (also known as SSC-96-01).

Two other reports recently published by the California Seismic Safety Commission, the *Provisional Commentary for Seismic Retrofit* (1994) and the *Review of Seismic Research Results on Existing Buildings* (1994), are Products 1.1 and 3.1 of the Proposition 122 Program, respectively. These two previous reports provide the primary basis for the development of the recommended methodology and commentary contained in this document.

This document is organized into two volumes. Volume One contains the main body of the evaluation and retrofit methodology, presented in 13 chapters, with a glossary and a list of references. This volume contains all of the parts of the document required for the evaluation and retrofit of buildings. Volume Two consists of Appendices containing supporting materials related to the methodology: four example building case study reports, a cost effectiveness study related to the four building studies, and a review of research on the effects of foundation conditions on the seismic performance of concrete buildings.

This report was prepared under the direction of ATC Senior Consultant Craig Comartin, who served as Principal Investigator, and Richard W. Niewiarowski, who served as Co-Principal Investigator and Project Director. Fred Turner served as CSSC Project Manager. Overview and guidance were provided by the Proposition 122 Oversight Panel consisting of Frederick M. Herman (Chair), Richard Conrad, Ross Cranmer, Wilfred Iwan, Roy Johnston, Frank McClure, Gary McGavin, Joel McRonald, Joseph P. Nicoletti, Stanley Scott, and Lowell Shields. The Product 1.2 methodology and commentary were prepared by Sigmund A. Freeman, Ronald O. Hamburger, William T. Holmes, Charles Kircher, Jack P. Moehle, Thomas A. Sabol, and Nabih Youssef (Product 1.2 Senior Advisory Panel). The Product 1.3 Geotechnical/Structural Working Group consisted of Sunil Gupta, Geoffrey Martin, Marshall Lew, and Lelio Mejia. William T. Holmes, Yoshi Moriwaki, Maurice Power and Nabih Youssef served on the Product 1.3 Senior Advisory Panel. Gregory P. Luth and Tom H. Hale, respectively, served as the Quality Assurance Consultant and the Cost Effectiveness Study Consultant. Wendy Rule served as Technical Editor, and Gail Hynes Shea served as Publications Consultant.

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Seismic Evaluation and Retrofit of Concrete Buildings

Products 1.2 and 1.3 of the Proposition 122
Seismic Retrofit Practices Improvement Program

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Glossary

Acceptability (response) limits: Refers to specific limiting values for the deformations and loadings, for deformation-controlled and force-controlled components respectively, which constitute criteria for acceptable seismic performance.

Brittle: see nonductile.

Capacity: The expected ultimate strength (in flexure, shear, or axial loading) of a structural component excluding the reduction (ϕ) factors commonly used in design of concrete members. The capacity usually refers to the strength at the yield point of the element or structure's capacity curve. For deformation-controlled components, capacity beyond the elastic limit generally includes the effects of strain hardening.

Capacity curve: The plot of the total lateral force, V, on a structure, against the lateral deflection, d, of the roof of the structure. This is often referred to as the 'pushover' curve.

Capacity spectrum: The capacity curve transformed from shear force vs. roof displacement (V vs. d) coordinates into spectral acceleration vs. spectral displacement (S_a vs. S_d) coordinates.

Capacity spectrum method: A nonlinear static analysis procedure that provides a graphical representation of the expected seismic performance of the existing or retrofitted structure by the intersection of the structure's capacity spectrum with a response spectrum (demand spectrum) representation of the earthquake's displacement demand on the structure. The intersection is the performance point, and the displacement coordinate, d_p, of the performance point is the estimated displacement demand on the structure for the specified level of seismic hazard.

Components: The local concrete members that comprise the major structural elements of the building such as columns, beams, slabs, wall panels, boundary members, joints, etc.

Concrete frame building: A building with a monolithically cast concrete structural framing system composed of horizontal and vertical elements which support all vertical gravity loads and also provide resistance to all lateral loads through bending of the framing elements.

Concrete frame-wall building: A building with a structural system composed of an essentially complete concrete frame system to support all gravity loads and concrete walls to provide resistance to lateral loads, primarily in shear.

Deformation-controlled: Refers to components, elements, actions, or systems which can, and are permitted to, exceed their elastic limit in a ductile manner. Force or stress levels for these components are of lesser importance than the amount or extent of deformation beyond the yield point (see ductility demand).

Degradation: Refers to the loss of strength that a component or structure may suffer when subjected to more than one cycle of deformation beyond its elastic limit. Degrading components are generally referred to as being force-controlled, brittle, or nonductile. Some or all of their flexural, shear or axial loading must be redistributed to other, more ductile, components in the structural system.

Demand: A representation of the earthquake ground motion or shaking that the building is subjected to. In nonlinear static analysis procedures, demand is represented by an estimation of the displacements or deformations that the structure is expected to undergo. This is in contrast to conventional, linear elastic analysis procedures in which demand is represented by prescribed lateral forces applied to the structure.

Demand spectrum: The reduced response spectrum used to represent the earthquake ground motion in the capacity spectrum method.

Displacement-based: Refers to analysis procedures, such as the nonlinear static analysis procedures recommended in this methodology, whose basis lies in estimating the realistic, and generally inelastic, lateral displacements or deformations expected due to actual earthquake ground motion. Component forces are then determined based on the deformations.

Displacement coefficient Method: A nonlinear static analysis procedure that provides a numerical process for estimating the displacement demand on the structure, by using a bilinear representation of the capacity curve and a series of modification factors, or coefficients, to calculate a target displacement. The point on the capacity curve at the target displacement is the equivalent of the performance point in the capacity spectrum method.

Ductile: see ductility.

Ductility: The ability of a structural component, element, or system to undergo both large deformations and/or several cycles of deformations beyond its yield point or elastic limit and maintain its strength without significant degradation or abrupt failure. These elements only experience a reduction in effective stiffness after yielding and are generally referred to as being deformation controlled or ductile.

Ductility demand: Refers to the extent of deformation (rotation or displacement) beyond the elastic limit, expressed numerically as the ratio of the maximum deformation to the yield deformation.

Elastic (linear) behavior: Refers to the first segment of the bi-linear load-deformation relationship plot of a component, element, or structure, between the unloaded condition and the elastic limit or yield point. This segment is a straight line whose slope represents the initial elastic stiffness of the component.

Elastic limit: See yield point.

Elastic response spectrum: The 5% damped response spectrum for the (each) seismic hazard level of interest, representing the maximum response of the structure, in terms of spectral acceleration S_a, at any time during an earthquake as a function of period of vibration, T.

Elements: Major horizontal or vertical portions of the building's structural systems that act to resist lateral forces or support vertical gravity loads such as frames, shear walls, frame-walls, diaphragms, and foundations. Elements are composed of components.

Force-controlled: Refers to components, elements, actions, or systems which are not permitted to exceed their elastic limits. This category of elements, generally referred to as brittle or nonductile, experiences significant degradation after only limited post-yield deformation.

Nonductile: Refers to a component or behavior that is not ductile and is generally subject to strength degradation beyond the elastic limit. These components are generally force-controlled.

Nonlinear static (analysis) procedure: The generic name for the group of simplified nonlinear analysis methods central to this methodology characterized by: use of a static pushover analysis to create a capacity curve representing the structure's available lateral force resistance, a representation of the actual displacement demand on the structure due to a specified level of seismic hazard, and verification of acceptable performance by a comparison of the two.

Performance-based: Refers to a methodology in which structural criteria are expressed in terms of achieving a performance objective. This is contrasted to a conventional method in which structural criteria are defined by limits on member forces resulting from a prescribed level of applied shear force.

Performance level: A limiting damage state or condition described by the physical damage within the building, the threat to life safety of the building's occupants due to the damage, and the post-earthquake serviceability of the building. A building performance level is the combination of a structural performance level and a nonstructural performance level.

Performance objective: A desired level of seismic performance of the building (performance level), generally described by specifying the maximum allowable (or acceptable) structural and nonstructural damage, for a specified level of seismic hazard.

Performance point: The intersection of the capacity spectrum with the appropriate demand spectrum in the capacity spectrum method (the displacement at the performance point is equivalent to the target displacement in the coefficient method).

a_p, d_p: coordinates of the performance point on the capacity spectrum,

a_{pi}, d_{pi}: coordinates of successive iterations (i=1, 2, etc.) of the performance point,

a_y, d_y: coordinates of the effective yield point on the capacity spectrum.

Primary elements: Refers to those structural components or elements that provide a significant portion of the structure's lateral force resisting stiffness and strength at the performance point. These are the elements that are needed to resist lateral loads after several cycles of inelastic response to the earthquake ground motion.

Pushover curve: see capacity curve.

Pushover analysis: An incremental static analysis used to determine the force-displacement relationship, or the capacity curve, for a structure or structural element. The analysis involves applying horizontal loads, in a prescribed pattern, to a computer model of the structure, incrementally; i.e., "pushing" the structure; and plotting the total applied shear force and associated lateral displacement at each increment, until the structure reaches a limit state or collapse condition.

Retrofit strategy: The basic overall approach adopted to improve the probable seismic performance of the building or to otherwise reduce the existing risk to an acceptable level.

Retrofit system: The specific method used to implement the overall retrofit strategy.

Secant (effective) stiffness: The slope of a straight line drawn from the origin of the capacity curve for a building (or other structural element) to a point on the curve at a displacement "d", beyond the elastic limit, represents the secant or effective stiffness of the structure (or element) when deformed by an earthquake to that displacement. The secant stiffness will always be less than the elastic stiffness of the structure.

Secondary elements: Refers to those structural components or elements that are not, or are not needed to be, primary elements of the lateral load resisting system. However, secondary elements may be needed to support vertical gravity loads and may resist some lateral loads.

Seismic hazard: The level of ground motion or shaking at the site for a given earthquake. Three standard levels of seismic hazard are specified in the methodology;

Serviceability Earthquake (SE); 50% chance of being exceeded in 50 years,

Design Earthquake (DE); 10% chance of being exceeded in 50 years,

Maximum Earthquake (ME); 5% chance of being exceeded in 50 years.

Strength: See capacity.

Target displacement: In the displacement coefficient method, the target displacement is the equivalent of the performance point in the capacity spectrum method. The target displacement is calculated by use of a series of coefficients.

Yield (effective yield) point: The point along the capacity spectrum where the ultimate capacity is reached and the initial linear elastic force-deformation relationship ends and effective stiffness begins to decrease. For larger elements or entire structural systems composed of many components, the effective yield point (on the bi-linear representation of the capacity spectrum) represents the point at which a sufficient number of individual components or elements have yielded and the global structure begins to experience inelastic deformation.

Executive Summary

■ Existing concrete buildings pose a great challenge in California

Concrete is popular as a building material in California. For the most part, it serves its functions well; however concrete is inherently brittle and performs poorly during earthquakes if not reinforced properly. The San Fernando Earthquake of 1971 dramatically demonstrated this characteristic. Shortly thereafter, code writers revised the design provisions for new concrete buildings to provide adequate ductility to resist strong ground shaking. There remain, nonetheless, millions of square feet of nonductile concrete buildings in California.

The consequences of neglecting this general risk are inevitably catastrophic for some individual buildings. The collapse of a single building has the potential for more loss of life than any other catastrophe in California since 1906. The potential defects in these buildings are often not readily apparent. Condemnation of all to mandatory retrofit is an unacceptable economic burden. Unfortunately, procedures to identify and to retrofit efficiently those that are vulnerable to collapse have not been available. As a part of its mandate under the California Earthquake Hazards Reduction Act of 1986, the Seismic Safety Commission is moving aggressively to meet this need by helping to develop standards for the evaluation and retrofit of existing concrete buildings with this document, *Seismic Evaluation and Retrofit of Concrete Buildings (Product 1.2/1.3)*. It contains the combined results of two contracts with the Applied Technology Council (Product 1.2 for the development of an analytical methodology and Product 1.3 for the inclusion of foundation effects).

■ The challenge spans a broad spectrum from highly technical engineering details to general issues of public policy

This document has a dual focus. On a technical level, engineers will find systematic guidance on how to investigate concrete buildings subject to seismic shaking. Depending on the specific characteristics of a particular building, they may select from an array of alternatives. These technical procedures are not alone sufficient for effective evaluation and retrofit. Owners, architects, building officials, and others must make critical decisions based on technical information coming from the engineers. Conversely, policy and management issues affect the course of the technical analysis. The recommended approach advocates a broad context for the process to expand the perspectives of all involved.

■ Multiple performance objectives are the context for defining and managing seismic risk

In *Turning Loss to Gain* (CSSC 1995) its report to Governor Wilson on the Northridge Earthquake, the Seismic Safety Commission identifies a fundamental drawback of the seismic provisions of current building codes. The seismic performance that can be expected from a building designed in accordance with the code is not explicit. The implication is that buildings will not collapse in large earthquakes. Owners rarely recognize that this goal allows for substantial damage contributing to the potential for large capital losses and business interruption. In spite of significant improvements in codes after earthquakes in the past, their traditional approach is not conducive to effective overall management of seismic risks in California. This is particularly true of existing buildings for which codes for new buildings are effectively meaningless when it comes to seismic performance. The Commission concludes that multiple performance objectives are required to define alternatives and quantify acceptable risks.

A seismic performance objective has two essential parts—a damage state and a level of hazard. "Life safety" and "immediate occupancy" are descriptors of damage states that do not constitute performance objectives until they are associated with a specific level of seismic hazard. The hazard might be an earthquake (M7.0 on the Hayward Fault adjacent to a site) or a probability of an intensity of ground shaking (10% chance of being exceeded in 50 years). Defined in this way, a performance objective represents a specific risk. Using the new analysis procedures in this document as a technical tool, it is possible to investigate buildings for multiple performance objectives. This approach provides building owners and others a framework for informed judgments on the acceptability of various risks and the benefits of mitigative action in light of the associated costs.

■ New structural analysis procedures give engineers a more realistic picture of building performance during earthquakes

Traditional retrofit design techniques assume that buildings respond elastically to earthquakes. In reality, large earthquakes can severely damage buildings causing inelastic behavior that dissipates energy. The assumption that buildings remain elastic simplifies the engineer's work but obscures a basic understanding of actual performance. The use of traditional procedures for existing buildings can lead to erroneous conclusions on deficiencies and unnecessarily high retrofit costs. More disturbingly, they can miss important defects in some buildings. Foundations are a good example. Traditional analyses normally assume that buildings are rigid at their base, which can lead to the prediction of high forces implying extensive retrofitting measures for walls and floors. It also can underestimate the structural displacements that control damage to other parts of the structure, such as columns. In reality, foundations often are quite flexible. Rocking or yielding of the supporting soil material might reduce forces and the need to retrofit the shear walls. The foundation movements, however, also lead to larger displacements which may imply potential collapse of columns.

Relatively new analysis procedures described in this document help describe the inelastic behavior of the structural components of a building. These techniques can estimate more accurately the actual behavior of a building during a specific ground motion. The document provides extensive guidance on the use of these procedures including properties for concrete components and detailed information to incorporate foundation effects. Using this information, the engineer formulates a component model of the building structure. The analysis procedure tells how to identify which part of the building will fail first. As the load and displacement increase, other elements begin to yield and deform inelastically. The resulting graphic "curve" is an easy-to-visualize representation of the capacity of the building. Several alternative techniques allow the demand from a specific earthquake or intensity of ground shaking to be correlated with the capacity curve to generate a point on the curve where capacity and demand are equal. This "performance point" is an estimate of the actual displacement of the building for the specified ground motion. Using this performance point, the engineer can characterize the associated damage state for the structure and compare it with the desired performance objective. This allows the engineer to pinpoint deficiencies in each building part and address them directly with retrofit measures only where necessary. In short, the procedure gives the engineer a better understanding of the seismic performance characteristics of the building and results in a more effective and cost-efficient retrofit.

■ The new technologies require extensive engineering judgment

A large team of earthquake engineering experts compiled and generated the information in this document. A panel of respected leaders in the field periodically reviewed the development as representatives of the Seismic Safety Commission. Practitioners from throughout California voiced their opinions at a series of workshops on the document. There is a consensus that the technical procedures are complex. There are several sources and implications of this complexity. The nature of the inelastic analysis itself requires a basic understanding of the principles of structural dynamics and mechanics of materials. The scope of the analysis typically requires computer-aided solutions. While most competent engineers with seismic design experience in California are capable of dealing with these issues, traditional design procedures commonly used in current practice do not demand that they do. Unfortunately, in the competitive design environment, most uninformed owners are not yet willing to pay larger fees for the more time-consuming approach. Although the benefits to owners in reduced construction costs, more reliable building performance, and reduced costs to repair damage due to future earthquakes can justify the higher fees in many cases, this has not yet been widely communicated. In the future, better communication and changes in the marketplace for engineering services could resolve this aspect of complexity.

The document provides guidance applicable to all concrete buildings. Within a general framework for evaluation and retrofit, new procedures for inelastic analysis are alternatives to simpler traditional methods for detailed analysis of some, but not all, buildings. The dividing line between buildings that can benefit from inelastic analysis and those that will not can be subtle, however. Every building has its own characteristics and often only experienced engineers can decide when traditional design methods are adequate. This necessity of experience and judgment on the part of the engineer extends beyond the selection of appropriate analysis techniques. The new inelastic procedures require many decisions on component properties and modeling techniques that involve considerable judgment. The interpretation of results must carefully include consideration of inherent uncertainties and the limitations of basic assumptions. Qualifying experience and judgment is not the exclusive domain of a select few engineers or firms. No one is capable of infallible prediction of the seismic performance of concrete buildings. The solution to this unavoidable complexity is to eliminate complete reliance on the judgment of a single engineer and, instead, rely on constructive and cooperative peer review processes. The Seismic Safety Commission, in *Turning Loss to Gain*, advocates such a change in the California Building Code to require independent peer review of complex buildings.

■ Effective and efficient seismic evaluation and retrofit of concrete buildings demand fundamental changes

The need for technical peer review is only one of the changes to conventional planning and design processes. The design engineers themselves face the challenge to develop and maintain their technical skills beyond those that they currently use in practice. Architects must recognize the impact of seismic risk on building function and the importance of nonstructural damage to building performance. Building officials are accustomed to designs that can be easily checked against prescriptive codes and standards. They must expand or supplement their own skills and implement procedures to monitor performance-based designs. As important as these changes for design and building professionals are, they alone will do little without the demand and support of building owners for change.

The perspective of building owners is the key to progress. If a building meets the code under which it was built and there is no legal requirement to retrofit it, owners generally have been satisfied. Few understand the risks they actually face. Performance-based evaluation of buildings can give them a picture of how earthquakes impact their businesses and investments. They can then begin to make informed decisions to manage and reduce risks in a cost-efficient way. The most basic change that owners will face is the realization that they are the decision maker. Engineers can advise them on relative risks, but acceptability rests with the owner. This concept runs counter to the prevailing attitude that it is the design professional who decides on acceptable risk.

■ Product 1.2/1.3 initiates the transition with a step in the right direction

The new technical analysis procedures, coupled with performance-based evaluation and design concepts, have great promise. Realistically, their implementation and complete development will take some time. The realization of the full potential of the new approaches demands technical information and data not currently available. Significant changes to business as usual are required on the part of all involved in the evaluation and retrofit process. There are several important strategies that can enhance future progress.

California is not acting alone in pursuing effective evaluation and retrofit methodologies. In fact, many of the individuals responsible for this document are also involved on the federal initiative to develop national guidelines for the seismic rehabilitation of buildings. They initially capitalized on the federal effort by using it as a springboard for further development. In return, key enhancements from *Product 1.2/1.3* have been funneled back into the federal document. There are at least two desirable outcomes of this synergism. *Product 1.2/1.3* uses concepts and language compatible with the federal guidelines. This consistency will raise the comfort level of all involved and accelerate the implementation of the procedures. The federal government, through the Federal Emergency Management Agency and the National Science Foundation, has plans to continue the development of performance-based design aggressively. The benefit for existing concrete buildings will include improved information on the inelastic properties of both structural and nonstructural components.

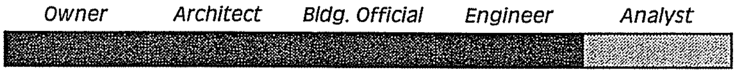
The interest on the part of structural engineers in inelastic analysis procedures is very high. The Structural Engineers Association of Northern California recently sponsored a seminar on the subject and had to turn people away. Future sessions are planned and interest probably will spread through the larger state organization. Focused sessions are required for geotechnical and structural engineers. The importance of foundation effects on the seismic performance of some buildings requires greater communication and cooperation. Training sessions are also essential for building officials throughout California. This document is a natural curriculum for these efforts and the Seismic Safety Commission encourages its use.

A concerted effort to educate building owners on effective seismic risk management is essential. First of all, the benefits of the new procedures need to be documented with extended example building studies. The proposed procedures have been used successfully by others and their stories need to be told. Side by side comparison of the results of the proposed procedures with those of traditional methods including retrofit costs would quantify the differences. State agencies are a natural starting point for workshops and seminars aimed at the management level. These could be expanded to the private sector through organizations such as the Building Owners and Managers Association. This initiative to engage building owners has not yet been implemented in any effective program.

Finally, this document needs to be continually updated to reflect advancement in the state of the art and the valuable lessons from practical application. A repository of information should be established to allow users to submit suggestions and share experiences on evaluation and retrofit projects.

The Seismic Safety Commission is confident that California can meet the challenge of concrete buildings with improved understanding and information. *Product 1.2/1.3* provides the basis for improved understanding of the actual behavior of structures for realistic earthquakes and for informed management of seismic risks. With continued vigilant effort on the part of design professionals, building officials, and owners to enhance the process, the risks to safety and economy posed by earthquakes can be steadily reduced to acceptable levels.

Chapter 1
Introduction



Audience Interest Spectrum

1.1 Purpose

1.1.1 General

A major portion of state and local government buildings in California are cast-in-place concrete structures designed and constructed before the mid to late 1970s. The seismic performance of these older buildings has been observed to be relatively poor compared to the performance of modern, post 1970s concrete buildings. Accordingly, a growing number of these buildings have been evaluated and retrofit in recent years and many more will be retrofit in the near future.

Very little has been, or is currently, available in the way of guidelines for use in the retrofit of existing concrete buildings. Therefore, most of the retrofit design and construction to date has been based on the use of the simple equivalent lateral force analysis procedures prescribed in building codes for the design of new buildings. These procedures do not directly address the actual forces induced in buildings by earthquake ground motions. More importantly, since buildings will respond to the earthquake ground motions in an inelastic manner, the linear elastic equivalent lateral force procedures do not provide a direct method to determine the resulting maximum displacements.

Given these shortcomings of the simple procedures, the concern has arisen that present approaches to retrofit may not deliver appropriate or cost-effective designs. Unrealistic or inadequate assessment of buildings may not identify the true failure modes, leading to unsafe retrofit designs, or may produce overly conservative retrofits where none is needed to meet the Owner's performance objective, leading to unnecessarily costly retrofit designs. Therefore, more sophisticated methods that consider both the actual loading and inelastic responses buildings experience in large earthquakes are needed.

The primary purpose of this document is to provide an analysis and design methodology and supporting commentary for use in the seismic evaluation and retrofit of existing state and local government concrete buildings in California. This methodology is intended to serve as the basis for the future development of building code provisions and standards and to provide guidelines for interim use until the more formal provisions are available.

It is expected that this document will be used by both retrofit design professionals performing seismic evaluations and retrofit designs and government agency personnel and policy makers charged with implementing retrofit programs. Portions of the document will be of interest to others, such as building owners and architects, involved in various aspects of building retrofit projects. However, the engineering expertise of a design professional, in particular the expertise of a structural engineer experienced in building retrofit design, is a prerequisite for appropriate use of the analytical procedures at the core of this methodology.

1.1.2 Proposition 122 Seismic Retrofit Practices Improvement Program

Passed by California's voters, Proposition 122 created the Earthquake Safety and Public Buildings Rehabilitation Fund of 1990, supported by a $300 million general obligation bond program for the seismic retrofit of state and local government buildings.

As a part of the program, Proposition 122 authorizes the Seismic Safety Commission to use up to 1% of the proceeds of the bonds issued and sold, or approximately $3 million, to carry out a range of activities that will capitalize on the seismic retrofit experience in the private sector to improve seismic retrofit practices for government buildings.

The overall purpose of California's Proposition 122 research and development program is to develop state-of-the-practice recommendations and methods to address current needs for uniform seismic retrofit provisions and seismic risk decision tools. It is focused specifically on vulnerable concrete structures consistent with the types of concrete buildings that make up a significant portion of California's state and local government inventories.

The two primary goals of the commission's Seismic Retrofit Practices Improvement Program are:

- To achieve cost-effective expenditure of state and local government funds allocated for the seismic retrofit of government buildings
- To obtain seismic retrofit designs that consistently and reliably achieve their intended seismic performance objectives

A 1991 Commission report titled *Breaking the Pattern* (CSSC 1991a) outlines four products to be developed over the multiyear program:

- **Product 1:** Provisions and commentary for the design of seismic retrofits for existing government buildings
- **Product 2:** Risk-management tools for use in seismic retrofit decision making by facility owners and managers
- **Product 3:** Short-term research projects to support the first two activities
- **Product 4:** Information transfer activities to inform government officials, facility owners and managers, and design professionals about the other products

This document reports the results of two separate but related projects conducted as part of the commission's Proposition 122 Seismic Retrofit Practices Improvement Program: Product 1.2, Development of a Recommended Methodology for the Seismic Evaluation and Retrofit of Existing Concrete Buildings and Product 1.3, Effects of Foundations on the Seismic Performance of Existing Concrete Buildings.

Two other reports recently published by the California Seismic Safety Commission, the *Provisional Commentary for Seismic Retrofit* (CSSC 1994a) and the *Review of Seismic Research Results on Existing Buildings* (CSSC 1994b), are Products 1.1 and 3.1 of the program, respectively. These two previous reports provide the primary basis for the development of the recommended methodology and commentary contained in this document.

1.2 Scope

1.2.1 General

This document provides a comprehensive, technically sound recommended methodology and supporting commentary for the seismic evaluation and retrofit design of existing concrete buildings. Although it is not intended for the design of new buildings, the analytical procedures are applicable. The document applies to the overall structural system and its elements (concrete frames, shear walls, diaphragms, foundations) and components (stiffness, strength, and deformability of columns, beams, walls, slabs, and joints). Consideration of nonstructural systems and components is also included in this document.

The methodology is performance based: the evaluation and retrofit design criteria are expressed as performance objectives, which define desired levels of seismic performance when the building is subjected to specified levels of seismic ground motion. Acceptable performance is measured by the level of structural and/or nonstructural damage expected from the earthquake shaking. Damage is expressed in terms of post yield, inelastic deformation limits for various structural components and elements found in concrete buildings. The analytical procedure incorporated in the methodology accounts for postelastic deformations of the structure by using simplified nonlinear static analysis methods.

This type of performance-based methodology for evaluation and retrofit design represents a fundamental change for the structural engineering profession. This type of analytical procedure is more complex than traditional force-based, prescriptive procedures such as those embodied in building codes for the design of new buildings. Although the use of simplified nonlinear static analysis procedures and their application to evaluation and retrofit design of existing buildings has grown over the past 15 to 20 years, widespread acceptance of these methods by the profession will come only through a considerable information transfer and learning process. Full acceptance will be achieved only when the ability of this method to identify potential structural deficiencies and to produce economical retrofit designs better than conventional practice has been demonstrated.

1.2.2 Uncertainty and Reliability

Uncertainty is a condition associated with essentially all aspects of earthquake related science and engineering and of the evaluation and retrofit of existing buildings. The principle sources of uncertainty lie in the characterization of seismic ground shaking, the determination of materials properties and of existing structural and geotechnical component capacities, and the assignment of the acceptance limits on structural behavior. These uncertainties, for the most part stemming from the lack of and/or the imperfect reliability of the specific supporting data available, affect all analytical methods and procedures applied to the challenge of seismic evaluation and retrofit.

The performance-based methodology presented in this document cannot and does not eliminate these uncertainties. However, through the use of simplified nonlinear static analysis, it provides a more sophisticated and direct approach to address the uncertainties than do traditional linear analysis procedures. By explicit consideration of the post-yield behavior of individual structural components, estimation of the degradation of member stiffness and strength, and representation of foundation effects, the methodology provides a more realistic, generally conservative, estimate or approximation of the actual deformations which will occur in the building in response to seismic ground motion. As a result, it is a useful and reliable design tool for assessment of expected building behavior and verification of proposed retrofit designs.

1.2.3 Procedure for Evaluation and Retrofit Design

The methodology is presented in the form of a step-by-step procedure for both evaluation and retrofit of existing buildings. The procedure recognizes, however, that some steps may be de-emphasized or performed in a different order on a case-by-case basis.

The primary components of the methodology used in various steps of the evaluation and retrofit procedure include:

- Definitions of seismic performance levels and seismic demand criteria for establishing seismic performance objectives

- Guidance for the review of existing conditions, preliminary determination of deficiencies, formulation of a retrofit strategy, and for establishing an appropriate quality assurance program
- Analytical methods or techniques for detailed investigations to assess seismic capacity and expected seismic performance of existing buildings and for verification of retrofit performance
- Materials characteristics rules and assumptions for use in modeling, assignment of capacities, and assessment of acceptable performance

The owner's or building code official's selection of the performance objective that should be achieved by a building retrofit is beyond the scope of this document. This includes the identification of the level of seismic hazard that should be combined with the selected performance level. Once those decisions have been made, however, and a performance objective has thus been established, this methodology provides guidelines to meet that objective. Compliance with the procedures and requirements of this document will be deemed adequate for these purposes. However, due to the uncertainties noted in Section 1.2.2, the seismic performance incorporated into the performance objective is not guaranteed.

1.2.4 Building Types

Two specific types of older, cast-in-place concrete buildings which were designed and constructed prior to the late 1970s, when ductile detailing requirements were first incorporated into building standards, and which are common to California state and local government building inventories, will be the focus of the methodology:

- Concrete frame buildings, generally constructed from the 1940s to the mid-1970s
- Concrete frame buildings with concrete walls, generally constructed from the early 1900s to the mid-1970s

1.2.4.1 Materials, Components, and Elements

Modeling rules and acceptance limits are provided for a variety of reinforced cast-in-place concrete elements and components found in the two building types, including beam-column frames; slab-column frames; solid, coupled, and perforated shear walls; concrete diaphragms; and foundations. Unreinforced masonry infill and precast concrete components are not considered in this document. These rules, assumptions, and limits are included for existing, non-complying elements and components, and for new, complying, elements and components used in retrofits.

1.2.4.2 Foundation-Soil Effects

The methodology includes guidelines for the consideration of foundation-soil effects. Detailed modeling rules and acceptance limits for various types of foundations and foundation-structure combinations in various soil conditions are included in this document.

1.2.4.3 Diaphragms

The methodology includes detailed guidelines for modeling rules and acceptance limits for concrete slab diaphragms, which may be considered to be rigid. Although general guidelines and commentary for the consideration of flexible diaphragms are included, the provision of detailed rules and assumptions for flexible diaphragms is not included in this document.

1.2.5 Alternative Analytical Methods

A variety of alternative analytical methods, using either simple (linear, static) procedures, approximate inelastic (simplified nonlinear static) capacity procedures, or complex inelastic (nonlinear time history) procedures, are available for use within the overall evaluation and retrofit methodology. The type of analytical approach described in this document is simplified nonlinear static analysis. Several methods of performing nonlinear static analyses are presented, and the capacity spectrum method is emphasized. Other analytical methods are also noted and discussion is provided to assist the retrofit professional in the selection of an analytical procedure appropriate for use in the detailed analysis of a particular building.

1.3 Organization and Contents

This document is organized into two volumes. Volume One contains the main body of the evaluation and retrofit methodology, presented in 13 chapters, with a glossary and a list of references. This volume contains all the parts of the document required for application and use of the methodology for evaluation and retrofit of a building. Volume Two consists of various Appendices containing supporting materials related to the methodology; four example building case study reports, a cost effectiveness study report related to the four building studies, and a review of research on foundation effects on the seismic performance of concrete buildings.

1.3.1 Volume One Chapter Summaries

The methodology has been organized into 13 chapters. The first seven address the more general and conceptual aspects of the methodology, which will be of interest to the broader range of the expected audience of building owners and agency representatives, architects, and building officials, as well as structural engineers and analysts. The next five chapters, 8 through 12, address the more technical and analytical aspects of the methodology, expected to be of primary interest only to the structural engineer/analyst members of the audience. The last chapter, 13, provides summary concluding remarks which are of interest to the broader audience. The title page of each chapter contains an audience spectrum bar to assist the reader in assessing the appropriate level of interest.

1.3.1.1 Chapter 1: Introduction

Chapter 1 provides a statement of the purpose and scope of this document, followed by a brief description of the content of each of the chapters and supporting study reports.

1.3.1.2 Chapter 2: Overview

Chapter 2 presents an overview of the general evaluation and retrofit methodology. The broad audience this document is intended to address is discussed and an Audience Interest Spectrum is provided to assist individuals in identifying which portions may be of interest or appropriate to them. Then, following a logical sequence of steps through the evaluation and retrofit design process, this chapter also serves as a road map for use of the document, with references to the appropriate chapters and sections at each step.

1.3.1.3 Chapter 3: Performance Objectives

Chapter 3 presents a detailed discussion of seismic performance objectives and how they are formed. Definitions, or damage state descriptions consistent with those included in other performance-based design documents, are provided for six standard 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

and five standard nonstructural performance levels:

- NP-A, Operational
- NP-B, Immediate Occupancy
- NP-C, Life Safety
- NP-D, Hazards Reduced
- NP-E, Not Considered

Performance levels for a building are formed by combining a structural and a nonstructural performance level to describe a complete building damage state. Performance objectives are then formed by combining a desired building performance level with a given earthquake ground motion. The chapter describes the process of selecting appropriate performance objectives, and one standard performance objective, called the Basic Safety Objective, is defined.

1.3.1.4 Chapter 4: Seismic Hazard

Chapter 4 provides guidelines for quantifying the seismic hazard at a site due to ground shaking for three levels of earthquake hazard: the Serviceability Earthquake, the Design Earthquake, and the Maximum Earthquake. Brief discussions of seismic ground failure hazards other than shaking are also provided.

The chapter describes the primary ground shaking criteria, site geology and soil characteristics, site seismicity characteristics, and site response spectra required for seismic evaluation and retrofit design of buildings. For various combinations of site soil profile types and site seismic zone factors, site seismicity coefficients are defined from which site response spectra may be constructed for any site in California. The chapter also provides guidance and general criteria for the use of acceleration time histories and duration of ground shaking.

1.3.1.5 Chapter 5: Determination of Deficiencies

Chapter 5 provides guidelines for a qualitative, preliminary evaluation of existing cast-in-place concrete frame and frame-wall buildings prior to the performance of detailed or extensive analytical work. A description of the common characteristics of these types of construction is provided, along with a discussion of their past seismic performance and typical deficiencies. Guidelines for collection of as-built information, including physical testing of materials and exploration of existing conditions, appropriate to the level of detail of evaluation or retrofit studies are provided.

Assessment of the seismic characteristics of existing buildings and determination of their seismic deficiencies and their severity, based on the collected data, is discussed. Also, considerations for establishing the extent of further, more detailed analysis that may be required to supplement the available as-built data, and preliminary determination of the need for retrofit, are also discussed.

1.3.1.6 Chapter 6: Retrofit Strategies

Chapter 6 provides an overview of the process of developing retrofit strategies (the basic approaches to improve the seismic performance of buildings) and preliminary retrofit designs for buildings. Discussion of various alternative retrofit strategies and the design constraints affecting retrofit strategy selection is provided. The process of selecting a retrofit strategy after an evaluation has indicated the presence of unacceptable seismic deficiencies and the decision to retrofit has been made is described. Considerations of alternative strategies, evaluation of their applicability given the identified deficiencies, and selection of the most appropriate strategy in light of the existing design constraints are discussed. Guidance for selection of an appropriate retrofit system to implement the chosen strategy and for development of preliminary retrofit designs is also provided.

1.3.1.7 Chapter 7: Quality Assurance Procedures

Chapter 7 provides guidelines for the various quality control procedures that may be required to ensure appropriate application of the methodology. Guidelines for peer review, plan check, and construction quality assurance procedures are presented and discussed. Although comprehensive programs are presented, conditions for which varying levels of review may be appropriate, depending on the complexity of a particular building, are discussed. Minimum requirements for field observation of the retrofit construction are provided, as are guidelines for field verification, testing, and inspection.

1.3.1.8 Chapter 8: Nonlinear Static Analysis Procedures

Chapter 8 presents the generalized nonlinear static analysis procedure characterized by use of a static pushover analysis method to represent the structure's lateral force resisting capacity, a representation of the actual earthquake displacement demand on the structure, and verification of acceptable performance by a comparison of the structure's available capacity to the earthquake's demand. A detailed description of each of the three primary elements of the nonlinear static analysis procedure is presented: the step-by-step development of the *capacity curve* of a structure, the various alternative methods to determine displacement demand by use of reduced *demand spectra* or target displacement coefficients, and the resulting identification of the *performance point* or target displacement and the subsequent check for acceptable performance.

Additional considerations, including the distinction between primary and secondary members, the effects of torsion, and the effects of higher modes, are discussed. An example is provided to demonstrate the application of the nonlinear static procedure to a building. Alternative methods, including linear elastic static and dynamic methods, the secant stiffness nonlinear static method, and nonlinear time history analysis methods, are discussed. The chapter closes with a brief summary of the fundamental structural dynamics basis of the nonlinear static analysis procedures.

1.3.1.9 Chapter 9: Modeling Rules

Chapter 9 provides the guidelines, rules, and assumptions required to develop the analytical model of buildings as two- or three-dimensional systems with nonlinear load-deformation properties. The guidelines for modeling the structural systems include application of loads; global building modeling considerations; materials models; elements models (concrete frames, concrete walls, concrete diaphragms, and foundations); and component models (columns, beams, joints, walls and slabs).

1.3.1.10 Chapter 10: Foundation Effects

Chapter 10 provides guidelines for the inclusion of foundation effects in the overall methodology for evaluation and retrofit design of existing concrete buildings. Guidelines are provided for modeling of geotechnical components for various types of shallow and deep foundation systems. Discussion is provided on response limits and acceptability criteria.

1.3.1.11 Chapter 11: Response Limits

Chapter 11 provides the guidelines, considerations, and assumptions required to assess the acceptability of the seismic response of the various components and elements of the structural and nonstructural systems. Qualitative descriptions of damage states are included for structural and nonstructural systems and quantitative limits are provided for the structural systems. Strength limits are provided for both ductile, deformation-controlled components and brittle, force-controlled components. Component and element deformability limits considering the Immediate Occupancy, Life Safety, and Structural Stability performance levels are presented.

1.3.1.12 Chapter 12: Nonstructural Components

Chapter 12 describes the minimum acceptance criteria that are expected to provide the Operational, Immediate Occupancy, Life Safety, and Hazards Reduced levels of performance for nonstructural systems and components. Acceptance criteria consist of listings of those systems and components that should be investigated for each performance level.

1.3.1.13 Chapter 13: Conclusions and Future Directions

Chapter 13 provides a detailed discussion of the various supplemental engineering studies reported in the six Appendices in Volume 2 and a summary of the principal conclusions drawn from the development of this Product. The conclusions are presented in a discussion of the potential benefits of, and the challenges posed by, the analysis and retrofit design methodology presented in this document. Benefits are discussed in terms of the engineer's improved understanding of seismic performance of buildings as well as the owner's enhanced options for implementing seismic retrofit goals in their buildings. Challenges are discussed in terms of both specific technical issues and broader practice issues. The chapter concludes with recommendations for future action, in terms of basic research to address the technical challenges and training and communication programs to address the practice issues.

1.3.2 Volume Two Appendices Summaries

1.3.2.1 Appendices A-D: Example Building Studies

These four Appendices contain the reports of engineering studies of four example buildings. The studies were performed primarily to test the proposed nonlinear static analysis methodology in three ways by comparisons to actual observed earthquake-caused damage to the selected buildings, to the results of limited linear elastic analyses, and to the results of limited time history analyses. In addition, these studies also demonstrate the application of the proposed methodology to actual buildings. The nonlinear static procedure was used to evaluate the expected performance of the unmodified buildings, and then to develop retrofit concept designs to achieve one or two specified levels of improved structural performance. The reports describe the results of the four studies, discuss observed strengths and limitations of the methodology, and provide some valuable insights into the assumptions and engineering judgments made.

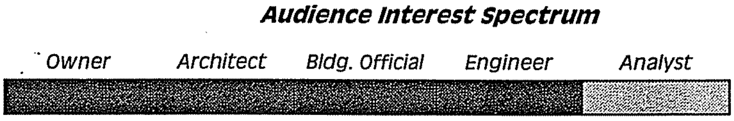
1.3.2.2 Appendix E: Cost Effectiveness Study

This section contains the report of a study performed to evaluate the cost-effectiveness and usability of the evaluation and retrofit methodology. The approximate construction cost of the various retrofit concept designs developed in the four example building studies is estimated and then compared to cost ranges from traditional retrofit approaches and the estimated cost of demolition and replacement. The cost effectiveness relationships observed between the extent of retrofit/improved seismic performance and construction costs is discussed. In addition, the ease of use and consistency of application of the proposed methodology, as demonstrated by the four example building studies, is assessed and discussed.

1.3.2.3 Appendix F: Supplemental Information on Foundation Effects

This section contains a report of a review of research on the affects of foundations on the seismic performance of concrete buildings. Presented in a format similar to the *Review of Seismic Research Results on Existing Buildings* (CSSC 1994b), but in smaller scale, this review provides an overview of the existing, pertinent research supporting Chapter 10 Foundation Effects in Volume One of the document. The report contains discussions of past seismic performance, experimental and theoretical studies, and analysis and design issues for both shallow and deep foundation systems, and a collection of brief review summaries of selected published articles, papers, and reports.

Chapter 2
Overview



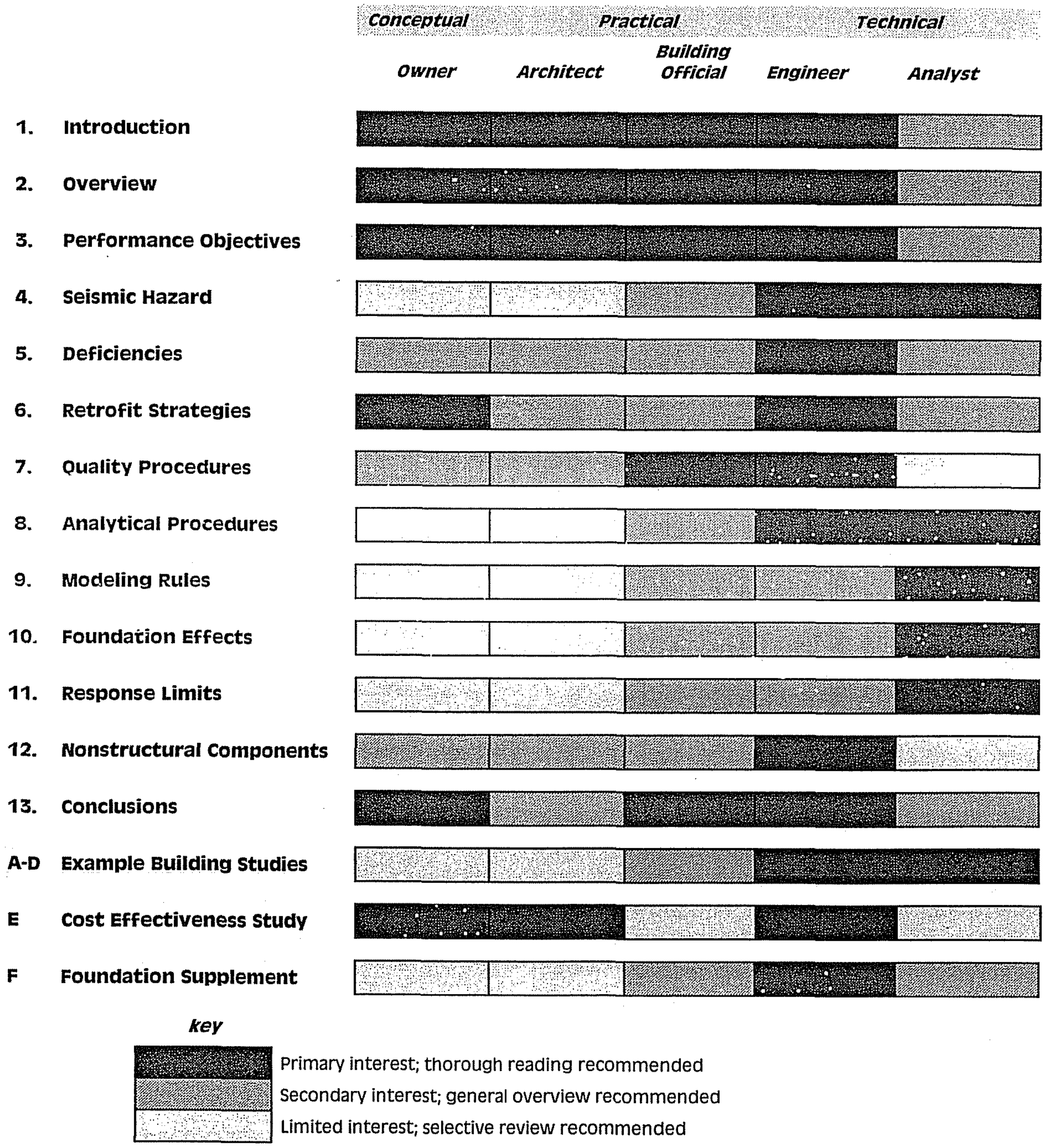
2.1 Introduction

The seismic evaluation and retrofit of existing concrete buildings pose a great challenge for the owners, architects, engineers, and building officials of California. The risks, measured in both lives and dollars, are high. Equally high is the inevitable uncertainty of where, when, and how large future earthquakes will be. The inherent complexity of concrete buildings and of their performance during earthquakes compounds the uncertainty. Traditional design and analysis procedures developed primarily for new construction are not wholly adequate tools for meeting this challenge.

This document presents a general methodology developed specifically to address the seismic evaluation and retrofit of concrete buildings in California. Promising new performance-based technical procedures can provide engineers with valuable insight about the actual performance of buildings during earthquakes. These and other changes from the status quo can greatly improve the process. It is important to emphasize, however, that straightforward, simple solutions that will cost-effectively produce acceptable seismic performance for all buildings do not exist. Within the general methodology described below, there are branches and paths that engineers and owners can select on the basis of the characteristics of a given building, the desired performance, cost limitations, and other project-specific factors. In the future, more direct and definitive processes may emerge. The procedures presented here are a step in the right direction.

The intended audience for this document includes building owners, building officials, architects, engineers, and others who may have a direct or peripheral interest in the seismic evaluation and retrofit of concrete buildings. This document is not a code, or even a comprehensive guideline. In one sense it is a commentary with a very broad perspective. Current technologies are developed and placed in context within the larger picture. Guidance on the selection of alternatives is offered. The objective of the document extends beyond the general, however, to the pragmatic. Using it as a manual, qualified engineers can apply the general principles to the evaluation and retrofit of actual buildings.

Not every chapter of this document is meant for detailed study by all readers. Within the general methodology are some very technical procedures of interest only to structural engineers and analysts. The audience interest spectrum (Figure 2-1) is provided to assist the reader in assessing the appropriateness of each chapter to his or her particular perspective. The audience interest spectrum bar for each chapter is also shown on the respective chapter's title page. Within each chapter, key points and basic concepts are highlighted in sidebars, figures, tables, and bulleted lists for the more casual reader.



Conceptual *Practical* *Technical*

		Owner	*Architect*	*Building Official*	*Engineer*	*Analyst*
1.	Introduction	■	■	■	■	▨
2.	Overview	■	■	■	■	▨
3.	Performance Objectives	■	■	■	■	▨
4.	Seismic Hazard	░	░	▨	■	■
5.	Deficiencies	▨	▨	▨	■	▨
6.	Retrofit Strategies	■	▨	▨	■	▨
7.	Quality Procedures	▨	▨	■	■	░
8.	Analytical Procedures			▨	■	■
9.	Modeling Rules			▨	▨	■
10.	Foundation Effects			▨	▨	■
11.	Response Limits	░	░	▨	▨	■
12.	Nonstructural Components	▨	▨	▨	■	░
13.	Conclusions	■	▨	■	■	▨
A-D	Example Building Studies	░	░	▨	■	■
E	Cost Effectiveness Study	■	■	░	■	░
F	Foundation Supplement	░	░	▨	■	▨

key

■ Primary interest; thorough reading recommended
▨ Secondary interest; general overview recommended
░ Limited interest; selective review recommended

Figure 2-1. Audience Interest Spectrum

2.2 Changes in Perspective

Meeting the challenge of concrete buildings demands a change in perspective on the part of everyone involved. The design and construction of new buildings follow a familiar pattern that has evolved over many years. Evaluation and retrofit are similar in some respects to new construction, but the differences greatly influence the effectiveness of the outcome. Just how should key participants in the process expect seismic evaluation and retrofit to impact their responsibilities?

2.2.1 Building Owners

For new construction projects, building owners rarely make explicit decisions with respect to design criteria. Most owners feel that compliance with prevailing building codes and standards is adequate for their purposes. Few recognize that these are prescriptive standards and, at best, merely imply an unspecified level of seismic performance. Owners rely on design professionals—architects and engineers—and building officials to select, apply, and enforce appropriate design criteria for their projects. A similar situation applies to the retrofit and rehabilitation of buildings. The prescriptive codes in place today normally allow for repairs, additions, or alterations to buildings provided that any new construction conforms to current code requirements. Changes to the building must not diminish its strength or, specifically, its ability to resist seismic forces. Building owners, again, have relied on design professionals and building officials to work out the specifics of these requirements for rehabilitation or retrofit projects.

Performance objectives form the basis of the methodology for evaluating and retrofitting concrete buildings. Building owners must be informed about the alternatives for each specific building. Performance-based guidelines give building owners the flexibility to coordinate seismic performance goals with other goals for the use of their facilities. The inherent uncertainty involved in predicting performance can affect the decision-making process. Owners must understand that they are playing a role in balancing relative risks rather than transferring risk to design professionals or contractors. The process is a change from the conventional; it requires that architects and engineers provide guidance for building owners.

2.2.2 Architects

Most seismic retrofit work is conducted as part of a rehabilitation project of larger scope. In some cases this is because disabled access, fire and life safety, or other issues must be addressed at the same time. Often, too, it is expedient to modernize and correct planning or programmatic problems with buildings at the time that seismic deficiencies are corrected. For this reason, architects generally have had, and will continue to have, an important role in seismic retrofit work.

Performance-based design requires a change in perspective on the part of architects. It is important for architects to learn more of the details of the functional aspects of a facility so that they can assist owners in making decisions about performance goals. For example, certain equipment may be essential for the continued operation of a facility after earthquakes, while other systems could sustain damage without seriously impeding building functions. In addition, some concrete buildings may be historic structures. Historic preservation can often impose restrictions on the type and extent of seismic retrofit measures that may be performed.

Table 2-1, adapted from *Architectural Practice and Earthquake Hazards* (CSSC 1992), provides guidance for architects seeking to improve their practice in seismic hazard mitigation.

Table 2-1. Options For Improving Architectural Seismic Design Practice

- Participate in continuing education programs, with special attention to seismic design and performance.
- Participate in post earthquake site visits to examine damage and study patterns of building behavior.
- Participate in the development of seismic codes and guidelines, work on code committees, and promote the use of design guidelines.
- Work with structural engineers who are experienced in seismic design.
- Develop seismic objectives and expectations for each project, jointly with the owner and other members of the design team.
- Ensure that conceptual and schematic designs are developed with joint architect/engineer participation.
- Develop a scope-of-work definition (a division of tasks between architect, engineer, and builder) for incorporation in each architect/engineer contract.
- Develop formal architect/engineer interaction techniques to deal with basic seismic issues, such as a professional interaction guide for all critical aspects of design (site characteristics, configuration, structural system and performance, and nonstructural components).
- Develop seismic performance guidelines and evaluation reports.
- Seek appropriate compensation for seismic design (based on defined scope of work and services).
- Educate owners on seismic design issues.
- Educate builders on seismic design issues. Encourage owners to discuss seismic design issues with builders.
- Provide independent expert design review for major projects.

2.2.3 Building Officials

The capabilities of building officials and building departments throughout the state of California vary widely. In urban jurisdictions, some state agencies have relatively large building departments, which often include technical staff. Smaller cities and counties may rely on private sector engineers and architects to check design drawings as part of the permitting process. The shift from prescriptive standards to those based on performance is a major change for building officials. Performance guidelines demand a lot of judgment on the part of design professionals, peer reviewers, and building officials.

There is no checklist with true-or-false responses in a performance-based design. It requires some flexibility on the part of building departments in their plan checking and review procedures. Complicated evaluation and retrofit projects for concrete buildings may ultimately be approved by a general consensus among the designers, peer reviewers, and building officials. This is quite different from normal building department operations. Performance-based design, in general, and seismic evaluation and retrofit, in particular, benefit greatly from the involvement of building officials early in the process and on a continuing basis. A basic understanding and spirit of cooperation can be developed from the beginning, and building officials can determine

Seismic Design Approaches

☑ **Prescriptive**		**Performance**
Building codes; checklists	**Basic Format**	Safety/damage/downtime goals for specific seismic hazard
Limited	**Owner's Choices**	Multiple
Routine	**Familiarity of Architects/Engineers**	Relatively new concept
Directly applicable	**New Buildings**	Supplemental enhancement to prescriptive
Partially applicable but limited	**Existing Buildings**	Fully applicable
Plan check normally sufficient	**Review Requirements**	Peer review normally required
Traditional	**Design Effort/Cost**	Higher than for prescriptive only

early on whether they will need the assistance of an outside peer reviewer or whether they might be able to handle some or all of the review with their own staff.

2.2.4 Engineers

For many years engineers have been using unrealistic simplified static lateral force procedures to design buildings to resist seismic forces and displacements. While traditional methods can result in adequate designs, they obscure a basic understanding of actual structural behavior and performance during earthquakes. Most engineers in California, particularly those experienced in seismic retrofit work, are capable of grasping the basic principles of the new procedures emerging for evaluation and retrofit. However, the majority are still unfamiliar with these new methods. The effective use of the new procedures requires a basic understanding of structural dynamics, ductility, and inelastic behavior of structural materials. Since these procedures are relatively untested, peer review becomes an important part of the process. Many individuals are understandably anxious when others review their work. For this reason, peer review should happen early in the process with a cooperative and collegial attitude on the part of all involved.

Performance-based design requires effective communication among the engineer, the architect, and the building owner. The engineer must carefully explain the alternatives for performance objectives and the implications for costs. In many instances, engineers are accustomed to "staying in the back room" and deferring to their architectural colleagues when it comes to communication with the owner. This traditional arrangement impedes the effectiveness of seismic evaluation and retrofit projects.

Within many engineering offices, younger engineers, more familiar with computer software and structural modeling techniques than their mentors, do the detailed structural analyses. The complexity and uncertainty inherent in the behavior of many structural elements and components of concrete buildings make it vitally important that experienced engineers actively participate in the modeling and analysis process. Extensive communication with and guidance from those who can interpret overall structural behavior is required to avoid unrealistic results. If the results do not make sense, there is a good chance they are not right. Modern analysis techniques can only augment the experience and intuition of a qualified engineer when it comes to understanding the seismic behavior of concrete buildings.

2.3 Getting Started

The following sections of this chapter constitute the general methodology for the seismic evaluation and retrofit of concrete buildings. This chapter takes the reader step-by-step through the entire process, following the 12 steps indicated in the "Step-by-Step" sidebar on the following page. Along the way, the purpose and use of each of the subsequent chapters emerges. As with other design and analysis processes, the path to a solution is not always direct. Reconsideration, changes, and recycling through the steps are to be expected.

1 *Initiate the Process*

A building owner might choose to evaluate and possibly retrofit an existing concrete building for a number of reasons. In the past, collapse prevention and the reduction of the risk to life safety for occupants have been the primary goals for most voluntary retrofits. In the future, life safety will remain the primary objective motivation for evaluation and retrofit. Increasingly, however, building owners recognize the benefits of better seismic performance in mitigating potential economic losses as well. This is particularly true with respect to the loss of income from a facility that fails to function after earthquakes.

Seismic issues are rarely the sole consideration for the scope of a change or addition to an existing building. Seismic performance evaluation and improvement may be secondary considerations of a

Step by Step

Pertinent Chapters

S
T
R
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1 *Initiate the process* ②

Jurisdictional Requirements
Architectural Changes
Voluntary Upgrade

2 *Select Qualified Professionals* ②

Structural Engineer
Architect

3 *Establish Performance Objectives* ③④

Structural Stability
Limited Safety
Life Safety
Damage Control
Immediate Occupancy

4 *Review Building Conditions* ④⑤

Review Drawings
Visual Inspection
Prelininary Calculations

5 *Formulate a Strategy* ②⑥

Simplified Procedures
Inelastic Capacity Methods
Complex Analyses

6 *Begin the Approval Process* ②⑦

Building Official
Peer Review

7 *Conduct Detailed Investigations* ④⑤

Site Analysis
Material Properties
Construction Details

8 *Characterize Seismic Capacity* ⑧⑨⑩

Modeling Rules
Force and Displacement

9 *Determine Seismic Demand* ④⑧

Seismic Hazard
Interdependence with Capacity
Target Displacement

10 *Verify Performance* ③⑥⑦⑩⑪

Global Response Limits
Component Acceptability
Conceptual Approval

11 *Prepare Construction Documents* ②⑦

Similarity to New Construction
Plan Check
Form of Construction Contract

12 *Monitor Construction Quality* ⑦

Submittals, Tests, and Inspections
Verification of Existing Conditions
Construction Observation by Designer

major remodeling effort undertaken for any number of reasons. Even if seismic performance improvement is the primary motivation, it is wise to consider a broader potential scope at the beginning of the project. Potential considerations include the following:

- Fire and life safety improvements
- Hazardous material abatement
- Disabled access improvement
- Change in programmatic use
- Functional improvements
- Building systems improvements
- Historic preservation

Some of these are voluntary and may simply make sense to include. Others may be required by law when changes are made to a building. Jurisdictional requirements vary, and it is prudent to make conservative assumptions early in the process. The actual scope may not emerge until later in the project, when more information becomes available. The expert advice of design professionals, including an architect and an engineer, is normally needed to finalize the scope of a project. Table 2-2 is a guideline to issues pertinent to the rehabilitation and retrofit process scope of work.

2 Select Qualified Professionals

Some owners have ongoing relationships with design professionals whom they know and trust. Others may never have dealt with architects or engineers. Public agencies are required to select project teams according to prescribed procedures. In any event, the careful selection of qualified professional assistance is more important with the evaluation and retrofit of existing concrete buildings than with most other projects. This is because of the complexity of the building type, the uncertainties of earthquake technology, and the lack of established precedents in design and analysis methodologies. Therefore, it is usually best to follow a selection procedure similar to the following:

1. Generate a list of potentially qualified candidates. This can be from past experience and general familiarity on the part of an owner, or from references from others who have done similar projects. Public agencies may develop a request for qualifications announcement with minimum criteria specified.
2. Request written submittals of statements of qualifications. The request should state the preliminary scope of the project to the extent possible.
3. Select several qualified candidates to submit formal proposals. In some cases, available drawings or other documents might be provided to the proposers. Site visits are also beneficial.
4. Interview one or more of the candidates on the basis of a review of the proposals. The interview is an opportunity to imagine the working relationship between an owner and the potential design team. Are the personalities compatible? Even if qualifications are excellent, the relationship between the owner and the consultants must also be conducive to success.
5. Thoroughly check references on similar projects. Ask references specific questions about the performance of the candidates and about the results of the job.
6. Make a selection and negotiate a contract. Keep in mind that the scope may change once the evaluation and retrofit strategy is developed in the initial stages of the project.

This selection process can be tailored to the needs of individual projects. In most cases, a team of design professionals led by either an architect or structural engineer is sought. In some cases, a structural engineer might fill the role without architectural assistance. An example would be when a preliminary study to determine deficiencies and develop conceptual remedial structural

Table 2-2. Seismic Design Checklist to Facilitate Architect/Engineer Interaction

Item	*Minor Issue*	*Moderate Issue*	*Significant Issue*	*Resolution*
Goals				
Structural Stability				
Limited Safety (structural)				
Hazards Reduced (non structural)				
Life Safety Structural				
Nonstructural				
Damage Control				
Immediate (continued) Occupancy				
Continued post earthquake function				
Site characteristics				
Near fault				
Ground failure possibility (landslide, liquefaction, subsidence)				
Soft soil (long periods, amplification, duration)				
Structural System				
Vertical discontinuity				
Soft story				
Setback				
Offset				
Resistance elements				
Plan discontinuity				
Adjacency-pounding possibility				
Dynamic resonance				
Diaphragm flexibility				
Torsion				
Redundancy				
Deformation compatibility				
Out-of-plane vibration				
Unbalanced resistance				
Resistance location				
Drift/interstory effects				
Strong column/weak beam condition				
Structural performance				
Ductility				
Inelastic demand				
Constant or degrading stiffness				
Damping				

Table 2-2 (Continued) Seismic Design Checklist to Facilitate Architect/engineer Interaction

Item	*Minor Issue*	*Moderate Issue*	*Significant Issue*	*Resolution*
Energy dissipation capacity				
Yield/fracture behavior				
Special system (e.g., Seismic Isolation)				
Mixed system				
Repairability				
Up-slope or down-slope conditions, collapse-hazard buildings nearby				
Nonstructural Components				
Cladding, glazing				
Deformation compatibility				
Mounting system				
Random infill				
Accessibility (lifelines, access/egress)				
Building Configuration				
Height				
Size effect				
Architectural concept				
Ceiling attachment				
Partition attachment				
Rigid				
Floating				
Replaceable partitions				
Stairs				
Equipment (Mech./Elec./Plumb.)				
Special equipment				
Computer/communications equipment				
Special building contents				

Source: Architectural Practice and Earthquake Hazards (CSSC 1992).

measures is sufficient. An architect might be hired later, as the situation dictates. In either case, the qualifications of the structural engineer are the most pertinent to the seismic evaluation and retrofit process. The key requirements are:

- Demonstrated experience in the analysis, design, and retrofit of concrete buildings in seismically hazardous regions
- Basic familiarity with the principles of performance-based seismic engineering and design
- Experience with inelastic analysis procedures and thorough understanding of reinforced concrete materials behavior
- Fundamental grasp of structural dynamics and the behavior of structures subject to strong ground motion
- Registration as a structural engineering authority in California

There are some very qualified engineers in the state who do not necessarily have all of the credentials above. This does not mean that they should be eliminated from consideration. An example might be an engineer with a small practice who has a good working relationship with a building owner. This individual might be perfectly capable of coordinating and implementing a fairly sophisticated evaluation and retrofit design with the assistance of an expert subconsultant with supplementary qualifications.

2.4 Basic Evaluation and Retrofit Strategy

For the construction of a new project, the path from start to finish is fairly clear from the beginning. The owner has a good idea of what a new building should be like, and with a little advice, the cost is fairly easy to estimate. However, the course of an existing building evaluation and retrofit project is often very different from that imagined at the outset. An owner might have an idea about the desired seismic performance, but rarely is it easy to ascertain the cost of attaining that level of performance early in the project. The architect and engineer are not starting with a clean slate as with a new building. What are the strengths and deficiencies of an existing building? How compatible are these with the owner's objectives? Is it worth while to spend a lot of money on sophisticated analyses? Is it better for the owner to simply accept the performance that can be economically attained? These and other questions can only be answered as the project unfolds. Recognizing this at the start and planning a flexible strategy to adjust to information as it emerges are the keys to success.

3 Establish Performance Objectives

At the beginning of an evaluation project, the design team should meet with the building owner to discuss seismic performance objectives. These are presented in detail in **Chapter 3**. The purpose primary of these initial meetings is to review with the owner the various options for seismic performance. It is important to remember that *attaining a performance objective consists of* achieving a certain level of performance for a specific level of seismic hazard. **Chapter 4** contains a detailed treatment of seismic hazard from ground shaking and addresses the potential for ground displacement due to liquefaction, surface faulting, and landsliding.

The consequences of earthquakes on buildings can be categorized into three types of losses:

- **Life Safety:** deaths and injuries to building occupants and passersby
- **Capital Losses:** costs to repair or replace the building or its contents
- **Functional Losses**: loss of revenue or increase in expenses related to the inability of a facility to function normally after earthquakes

The level of performance for a building during earthquakes is measured by the nature and extent of these potential losses. Obviously, the level of performance is affected by the strength of each earthquake. It is reasonable to expect that a building remain safe, i.e., not cause life loss, for rare large earthquakes and that it remain usable for more-frequent moderate events. A performance objective is a goal that a building achieve a certain level of performance for a specific level of seismic ground shaking hazard. An owner might decide that the goals for a building should be to remain life safe for the Maximum Earthquake hazard level and functional after a Serviceability Earthquake hazard level event.

It is important that building owners understand that the process of seismic evaluation and retrofit is a risk-reduction process. The goals that owners select for building performance are just that—goals. Qualified design professionals who sense this understanding in their clients can be extremely effective in helping owners to manage risk and deal with uncertainty. They will be less effective, on the other hand, for poorly informed clients who try to transfer risk and who expect "guaranteed" building performance.

4 Review Building Conditions

Chapter 5 provides detailed guidelines for a preliminary investigation of a building by an engineer. This process normally involves a site visit to physically inspect the building and a search for and review of existing drawings or other documents that may describe the structural characteristics of the building. The structural engineer might also do some preliminary calculations to determine whether any of the features of the building are potential seismic deficiencies in terms of the preliminary performance goals. **Chapter 5** includes recommendations on the types of information that the engineer should compile. At this point, the engineer should also consider the potential site hazards covered in **Chapter 4.**

At some point during this initial process, another meeting or series of meetings usually takes place with the building owner, engineer, and architect. On the basis of the preliminary field investigation, the engineer can often qualitatively describe to the owner what it might take to meet preliminary performance goals. In light of these discussions, the owner and design team might decide to revise or augment the performance objectives. For example, if a building owner is initially interested only in life safety, the engineer may be able to determine from preliminary investigations of the building that a performance goal beyond life safety could be achieved without a much greater investment. Of course, in many cases the opposite course must be taken: performance goals and expectations may have to be lowered on the basis of what is determined in the field. Supplemental detailed investigations of existing conditions may be required at a later stage in the process.

5 *Formulate a Strategy*

After the owner revises or confirms the performance objectives, the design team should develop a plan for the detailed evaluation and possible retrofit. There are many possible alternatives to mitigate seismic risks, as discussed in **Chapter 6.** The owner should continue to remain involved during this planning process.

This document presents relatively new technology that allows engineers to gain a more realistic picture of the potential seismic performance characteristics of buildings. These nonlinear static procedures constitute an inelastic analysis that considers what happens to buildings after they begin to crack and yield in response to realistic earthquake motions. This approach differs from traditional linear static procedures that reduce seismic forces to levels that allow engineers to design buildings under the assumption that they remain undamaged. Although unrealistic and potentially misleading, this simplistic approach can work well for new buildings and for smaller, simpler, and regular existing buildings. The advantage of the newer, nonlinear static procedures, when applied to existing buildings, is that they credit the good features of buildings at the same time that they identify deficiencies. In spite of its improvements over traditional analysis, this new approach may not be appropriate for all buildings.

Some buildings may be too complex to rely on the nonlinear static procedures. These cases may require time history analyses of the nonlinear behavior of structures during example earthquakes. The kinds of buildings that may require these specialized analyses are those that are highly irregular or complicated. Other examples of building systems that may necessitate more-sophisticated analysis are energy dissipation or base isolation systems.

At the other end of the spectrum are simpler buildings for which use of nonlinear static analysis is not necessary. Although there are no hard and fast rules to identify them, buildings that possess one or more of the following characteristics should be considered candidates:

- Small size
- Low-rise (one or two stories)
- Uncomplicated (regular) structural systems
- Highly redundant lateral force resisting system
- Low occupancy

For those cases, use of simpler, linear elastic analysis procedures may be sufficient. There are a number of such procedures, briefly reviewed in Section 8.4.1, that may be utilized, depending on the specific characteristics of a given building.

In some cases, however, the simplified procedures do not identify specific seismic deficiencies. For example, in shear wall buildings the columns can be subject to excessive displacement and potentially fail when the shear walls begin to crack or rock at the foundation. Simplified procedures generally will not identify this deficiency. Simplified procedures can also result in overly conservative designs. This is appropriate, since the level of detail required for analysis in the simplified procedures is less extensive than that for the more sophisticated approach, so the additional conservatism is warranted. In some cases, however, this additional conservatism can result in excessively high construction costs. Also a consideration is the potential for disruption and possible closure of buildings for retrofit. The simplified procedures tend to require more-extensive retrofits and are more likely to cause disruption and/or the need for vacating buildings.

Another situation where use of simplified procedures is appropriate is when existing structural systems are so inadequate that complete new systems must be installed in any event. In this case, the simplified procedure does not give up much by neglecting the strength of existing systems. Also, if architectural considerations and historical restrictions are not significant, the freedom of placing resisting elements where they are most effective can provide an opportunity for simplified procedures. Furthermore, if a building is highly redundant, not particularly irregular, and of good construction quality, the scope of retrofit, even if determined by simplified procedures, is likely to be small, and the benefits of more-sophisticated analysis may be relatively insignificant.

Simplified procedures have the advantage that most engineers currently find them easier to use than the new, nonlinear static procedures outlined in this document. This is because the simplified procedures more closely parallel the design approaches that have traditionally been used in structural engineering practice. It will take time for engineers to assimilate the new procedures.

In addition, the simpler analysis procedures lead to lower costs for evaluation and retrofit design fees. In a competitive market, the lower design cost approach is attractive to owners. However, professional fees are not the critical cost consideration in seismic evaluation and retrofit. In the long run, owners will begin to recognize the benefits of the better information obtained from use of the more sophisticated analysis procedures. Better information leads to much-more-significant economy in the reduction of both construction costs and actual earthquake losses.

6 Begin the Approval Process

Once a plan for the evaluation of the building is formulated, the basic strategy should be reviewed in detail with the building official. This sequence is somewhat different from that for traditional design and construction, where the building official sometimes is not consulted until the end of the design process. The complexity and uncertainty inherent in the seismic evaluation of concrete buildings demand much greater collaboration between the design team and the building official. At this point in the planning process, the extent of the required peer review for the project should be discussed. The scope of the peer review depends on a number of factors, including the complexity of the building itself and the proposed evaluation procedures, the ability of the building department to understand and review the evaluation and retrofit design, and the capability and experience of the structural engineer and design team. In most cases it is advisable to have a peer review panel engaged early to evaluate the strategy for the evaluation and retrofit. In some instances, however, it may be acceptable to forego formal peer review until the completion of the evaluation and a conceptual design. **Chapter 7** covers some of the more detailed aspects of the peer review process and other quality control measures.

7 *Conduct Detailed Investigations*

The plan for the evaluation and retrofit process often includes detailed tests and inspections to gain more information about the specific characteristics of a particular building. One of the reasons that careful planning and approval by all involved are important in the early stages of the project is that these tests and inspections often can be expensive. Some of the detailed options are reviewed in **Chapter 5.**

In some cases the lack of drawings and other documentation for a particular building may significantly affect the selection of an evaluation procedure. In fact, simplified procedures may be most appropriate for buildings about which little is known. In these cases it often can be most cost-effective to simply provide a completely new seismic force resisting system rather than try to thoroughly investigate and document existing conditions for more-sophisticated analyses.

The activities up to this point in the overall evaluation of the retrofit process can be viewed as a strategic planning effort. The complexity and uncertainty involved in the overall process make this stage extremely important in controlling both the costs and the quality of the work. In some instances, the resulting plan may allow for contingencies and changes based on what is discovered later on. For example, a nonlinear static procedure may be considered the most effective analysis procedure in the beginning. However, detailed investigations of field conditions might lead to the conclusion that a simplified analysis could economically satisfy the performance goals.

2.5 Evaluation and Retrofit Concept

The essence of virtually all seismic evaluation procedures is a comparison between some measure of the "demand" that earthquakes place on a structure to a measure of the "capacity" of the building to resist. Traditional design procedures characterize demand and capacity as forces. Base shear (total horizontal force at the lowest level of the building) is the normal parameter that is used for this purpose. The engineer calculates the base shear demand that would be generated by a given earthquake, or intensity of ground motion, and compares this to the base shear capacity of the building. The capacity of the building is an estimate of a base shear that would be "acceptable." If the building were subjected to a force equal to its base shear capacity some deformation and yielding might occur in some structural elements, but the building would not collapse or reach an otherwise undesirable overall level of damage. If the demand generated by the earthquake is less than the capacity then the design is deemed acceptable.

The first formal seismic design procedures recognized that the earthquake accelerations would generate forces proportional to the weight of the building. Over the years empirical knowledge about the actual behavior of real structures in earthquakes and theoretical understanding of structural dynamics advanced. The basic procedure was modified to reflect the fact that the demand generated by the earthquake accelerations was also a function of the stiffness of the structure.

Engineers also began to recognize the inherently better behavior of some buildings over others. Consequently, they reduced seismic demand based on the characteristics of the basic structural material and system. The motivation to reduce seismic demand for design came because engineers could not rationalize theoretically how structures resisted the forces generated by earthquakes. This was partially the result of their fundamental assumption that structures resisted loads linearly without yielding or permanent structural deformation.

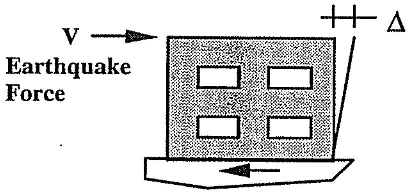
An important measure—the capacity of a structure to resist seismic demand—is a property known as ductility. Ductility is the ability to deform beyond initial yielding without failing abruptly. If a pencil is bent up to and beyond its yield point, it snaps. In contrast, a coat hanger deforms permanently way beyond its initial yield. It is ductile compared to the pencil. This property is a critical component of structural capacity.

Instead of comparing forces, nonlinear static procedures use displacements to compare seismic demand to the capacity of a structure. This approach includes consideration of the ductility of the structure on an element by element basis. The inelastic capacity of a building is then a measure of its ability to dissipate earthquake energy. The current trend in seismic analysis is toward these simplified inelastic procedures. **Chapter 8** is a detailed presentation of how they are used to compare the capacity of a structure to the demand imposed on it by a given ground motion.

8 Characterize Seismic Capacity

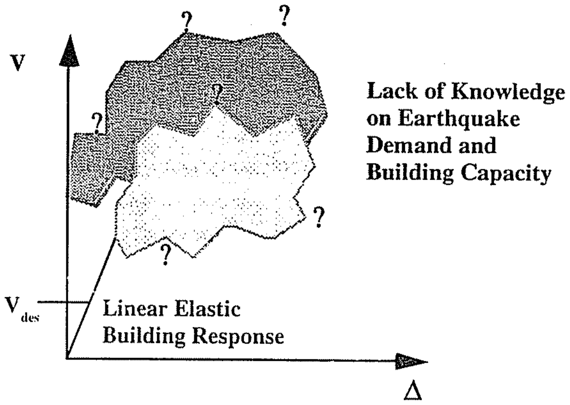
The recommended central methodology in this document concentrates on the formulation of the inelastic capacity curve for the structure as presented in **Chapter 8**. This curve is a plot of the horizontal movement of a structure as it is pushed to one side. Initially the plot is a straight line as the structure moves linearly. As the parts of the structure yield the plot begins to curve as the structure softens. The engineer generates this curve by building a model of the entire structure from nonlinear representations of all of its elements and components. Most often this is accomplished with a computer and structural analysis software. Using the modeling rules of **Chapters 9 and 10** the engineer specifies force and displacement characteristics for each piece of the structure resisting the earthquake demand. These pieces are assembled geometrically to represent the complete lateral load resisting system. The resulting model is then subject to increasing increments of load in a pattern determined by its dynamic properties. The corresponding displacements define the inelastic capacity curve for the building. The generation of the capacity curve defines the capacity of the building uniquely and independently of any specific seismic demand. In this sense, it replaces the base shear capacity of traditional procedures.

Evolution of Seismic Design



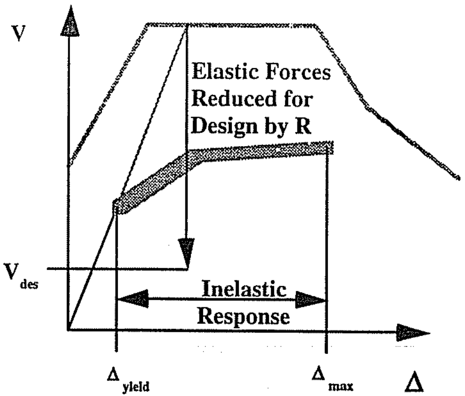
Historical Approach

- Earthquake forces proportional to building mass (V_{des} = 5-10% of Wt.)
- Linear design using "factors-of safety" to account for uncertainty in earthquake demand and building capacity



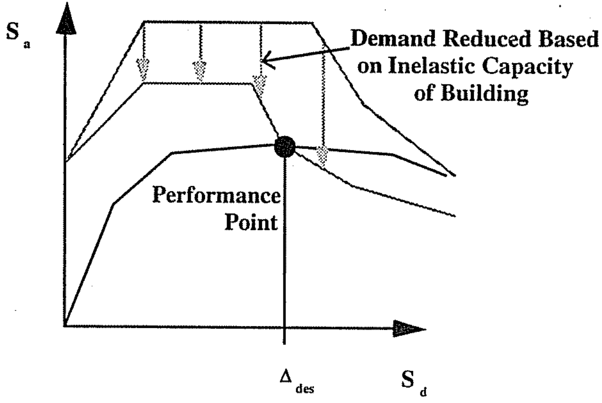
Traditional Code Basis

- Elastic earthquake forces reduced for linear design ($V_{des} = V_{max} / R$)
- R_i varies based on typical inelastic response of structural types
- Reduction justified by expected ductility, $\Delta_{max} / \Delta_{yield}$



Current Trend

- Inelastic earthquake demand based on inelastic capacity of building
- Resolution of demand vs. capacity generates Performance Point
- Design based on displacement, Δ_{des}



When an earthquake displaces the building laterally, its response is represented by a point on this curve. A point on the curve defines a specific damage state for the building, since the deformation of all of its components can be related to the global displacement of the structure.

9 Determine Seismic Demand

The capacity of a particular building and the demand imposed upon it by a given earthquake motion are not independent. One source of this mutual dependence is evident from the capacity curve itself. As the demand increases the structure eventually yields and, as its stiffness decreases, its period lengthens. Conversion of the capacity curve to spectral ordinates (ADRS) outlined in **Chapter 8** makes this concept easy to visualize. Since the seismic accelerations depend on period, demand also changes as the structure yields. Another source of mutual dependence between capacity and demand is effective damping. As a building yields in response to seismic demand it dissipates energy with hysteretic damping. Buildings that have large, stable hysteresis loops during cyclic yielding dissipate more than those with pinched loops caused by degradation of strength and stiffness. Since the energy that is dissipated need not be stored in the structure, the damping has the effect of diminishing displacement demand.

Chapter 8 devotes much attention to the development and presentation of the Capacity Spectrum Method. The Capacity Spectrum Method characterizes seismic demand initially using a 5% damped elastic response spectrum as detailed in **Chapter 4**. This spectrum is plotted in spectral ordinates (ADRS) format showing the spectral acceleration as a function of spectral displacement. This format allows the demand spectrum to be "overlaid" on the capacity spectrum for the building. The intersection of the demand and capacity spectra, if located in the linear range of the capacity, would define the actual displacement for the structure; however this is not normally the case as most analyses include some inelastic nonlinear behavior.

To find the point where demand and capacity are equal, the engineer assumes a point on the capacity spectrum as an initial estimate. Using the spectral acceleration and displacement from this point, the engineer then can calculate reduction factors to apply to the 5% elastic spectrum to account for the hysteretic energy dissipation associated with the specific point. These reduction factors have the effect of pulling the demand spectrum down. If the reduced demand spectrum intersects the capacity spectrum at or near the initial assumed point, then it is the solution for the unique "performance point" where capacity equals demand. If the intersection is not reasonably close to the initial point, then the engineer can assume a new point somewhere between and repeat the process until a solution for the performance point is reached.

Chapter 8 also presents an alternative for estimating the "performance point" where capacity and demand all equal for a given earthquake motion. The proposed federal guidelines (ATC, 1996a) presents one of these called the Displacement Coefficient Method. It uses a series of coefficients to modify the hypothetical elastic response of a building to estimate its inelastic displacement demand.

10 Verify Performance

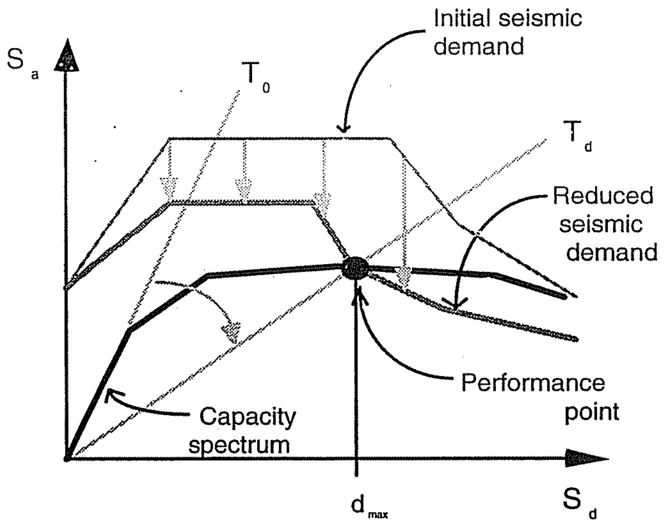
Once the performance point for a demand earthquake is estimated, the engineer checks the resulting performance of the building using the acceptability criteria in **Chapters 10 and 11**. The performance is checked on two levels. First, there are global limits for displacement of the structure for each performance objective. For example, the roof of a building might move four inches during an earthquake for which it should be life safe. For the same building the total roof displacement might be limited to two inches for a more frequent earthquake in order to meet a damage control

Capacity vs. Demand

Demand and capacity are mutually dependent.

As displacements 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 demand to find an intersection with the capacity spectrum where the displacement is consistent with the implied damping.



At the performance point, capacity and demand are equal.

The maximum displacement implies a unique damage state for the building related directly to a specific earthquake or intensity of ground shaking. The damage state comprises deformations for all elements in the structure. Comparison with acceptability criteria for the desired performance goal leads to the identification of deficiencies for individual elements.

performance goal. Similarly, the engineer checks individual structural elements against acceptability limits which depend on the global performance goal. The nature of the element acceptability limits vary according to the specific element. Inelastic rotation is the acceptability parameter for beams, for example. The limits on inelastic rotation are recommended based on observations from tests and performance in past earthquakes. A similar, though somewhat more qualitative procedure measures the acceptability of nonstructural components.

The nonlinear static procedures, including the Capacity Spectrum Method, can appear to some to be tedious and complicated compared to traditional design procedures. There are, however, some distinct benefits.

First, the entire process of generating the capacity curve and dealing directly with the interdependence of capacity and demand gives the engineer a greatly enhanced understanding of the actual performance of the specific building. This enables the engineer to apply the necessary experience and judgment at a much more refined level than traditional procedures.

Also advantageous are the significantly more useful results of the analysis. The performance point at which the seismic capacity equals the demand characterizes performance as a specific building damage state for a specific earthquake intensity. The probability of occurrence of the earthquake intensity defines the risk of occurrence for the damage state.

Although the accuracy of this estimate is limited by unavoidable uncertainties, this explicit relationship between performance and risk is superior to the implicit intent of current code procedures. Component deformations, directly related to damage, are a much better parameter than forces for performance evaluation. In contrast to force-based traditional methods, the damage state from nonlinear static procedures characterizes the deformation of building components for comparison with acceptability criteria depending on the desired performance. With the tabulation of the acceptability of individual element deformations, the engineer can pinpoint deficiencies within the structure. This facilitates a directed retrofit strategy that is both effective and cost efficient.

In order to meet some or all of the performance goals for a building, retrofit may be required to improve performance. If the performance goals are met by the existing structure then retrofit may not be necessary. There are several strategies to develop appropriate retrofit measures as discussed in **Chapter 6**. The choice depends on the type of structure, the nature of the deficiencies, the constraints of the architecture and planning, and the funds available. In some cases, the owner may decide to accept lesser performance in deference to these constraints. The effectiveness of a selected retrofit strategy is tested by adding or modifying elements in the structural model, then re-analyzing the model as before. Often several different strategies are investigated before settling on a preferred concept.

2.6 Final Design and Construction

This document concentrates on the conceptual phase of seismic evaluation and retrofit projects. In reality, the detailed design work represents a major portion of the effort and occurs after the owner's preferred concept is developed. This process, however, is similar to that for conventional construction of new buildings.

11 Prepare Construction Documents

The structural details for the retrofit typically conform to code requirements for new work. As the design continues, the peer reviewer checks to see that the approved concept is faithfully implemented by the engineer, as discussed in **Chapter 7.** These reviews usually occur at the end of the design development phase and near the completion of construction documents. Before issuing a permit for construction, the building official will normally have a plan check made of all project documents.

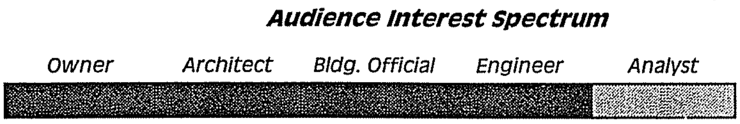
In the absence of legal restrictions, the form of contract for retrofit construction may be one of many, similar to new construction. Lump sum bidding is best suited to projects where the existing conditions of the building are relatively well known and the retrofit measures comprise normal construction practices. Where field conditions are uncertain or retrofit measures are innovative or unique, a construction management approach can be very effective. In fact, engaging a contractor during design can often provide valuable assistance to the engineer in the selection of cost-efficient retrofit techniques and details of construction. A unique contractual arrangement may be appropriate in some cases—for instance, where there are specialized retrofit procedures, such as seismic isolation or energy dissipation devices.

12 Monitor Construction Quality

The actual seismic performance of buildings is extremely sensitive to construction quality. This is especially true for retrofitted buildings. Field conditions in existing buildings routinely vary from those shown on drawings or implied by visual inspections during the evaluation process. These changes can have significant impact on retrofit designs. Some retrofit techniques are sophisticated and require special inspections and tests. It is vitally important that the engineer prepare a project-specific construction quality assurance plan. The plan should identify required tests, inspections, submittals, and personnel qualifications. The engineer of record should make regular inspections and receive immediate notification of any field problems.

Chapter 3

Performance Objectives



3.1 Introduction

A performance objective specifies the desired seismic performance of the building. 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.

Once the building owner selects a performance objective, the engineer can identify the seismic demand to be used in the analysis and the acceptability criteria to be used for evaluation and design of the building's structural and nonstructural systems. While the majority of retrofitted buildings are expected to meet or exceed the assigned performance level when exposed to the ground motion implied by the selected hazard level, such performance should not be considered guaranteed.

Performance Objective =
Desired Building Performance Level for a Given Earthquake Ground Motion

This chapter defines several standard performance levels for structural and nonstructural systems and several commonly used combinations of structural and nonstructural levels, called Building Performance Levels. Standard earthquake hazard levels are introduced and the process of selecting appropriate performance objectives is described. A detailed discussion of seismic hazard is contained in Chapter 4.

3.2 Performance Levels

A performance level describes a limiting damage condition which may be considered satisfactory for a given building and a given ground motion. The limiting condition is described by the physical damage within the building, the threat to life safety of the building's occupants created by the damage, and the post-earthquake serviceability of the building.

Target performance levels for structural and nonstructural systems are specified independently. Structural performance levels are given names and number designations, while nonstructural performance levels are given names and letter designations. Building Performance Levels are a

Table 3-1. Combinations of Structural and Nonstructural Performance Levels to Form Building Performance Levels

Building Performance Levels						
	Structural Performance Levels					
Nonstructural Performance Levels	*SP-1 Immediate Occupancy* ↓	*SP-2 Damage Control (Range)* ↓	*SP-3 Life Safety* ↓	*SP-4 Limited Safety (Range)* ↓	*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 Hazards Reduced →	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

Legend

☐	Commonly referenced Building Performance Levels (SP-NP)
▨	Other possible combinations of SP-NP
NR	Not recommended combinations of SP-NP

combination of a structural performance level and a nonstructural performance level and are designated by the applicable number and letter combination such as 1-A, 3-C, etc. as shown in Table 3-1.

3.2.1 Structural Performance Levels and Ranges

Structural performance levels and ranges are assigned a title and, for ease of reference, a number. The number is called the structural performance number and is abbreviated SP-n (where n is the designated number).

The Structural Performance Levels—Immediate Occupancy, Life Safety, and Structural Stability—are discrete damage states and can be used directly in evaluation and retrofit procedures to define technical criteria. The other structural performance designations—Damage Control, Limited Safety, and Not Considered—are important placeholders in the numbering scheme to allow direct reference to the wide variety of building performance levels that might be desirable to owners for evaluation or retrofit.

Commentary: These descriptions of acceptable damage at various performance levels *are similar to those used in FEMA 273 (ATC 1996a). These descriptions are also similar in concept, if not in terminology, to those proposed in the Vision 2000 Progress Report (SEAOC 1995b).*

- **Immediate Occupancy, SP-1:** The post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral force resisting systems of the building retain nearly all of their pre-earthquake characteristics and capacities. The risk of life-threatening injury from structural failure is negligible, and the building should be safe for unlimited egress, ingress, and occupancy.
- **Damage Control, SP-2:** This term is actually not a specific level but a range of post-earthquake damage states that could vary from SP-1, Immediate Occupancy to SP-3, Life Safety. It provides a placeholder for the many situations where it may be desirable to limit structural damage beyond the Life Safety level, but occupancy is not the issue. Examples of damage control include protection of significant architectural features of historic buildings or valuable contents.

Building Performance Level =

Structural Performance Level

+

Nonstructural Performance Level

***Commentary:** The Damage Control range, also sometimes called Limited Damage, is defined to allow reference to performance levels between Immediate Occupancy and Life Safety. Although not specifically defined in other current documents, the expected performance of most new buildings for the 10 percent/50-year event (see Section 3.3) would probably fall in this range (EERI 1994). A performance equivalent to that expected of new buildings is also sometimes called Repairable Damage, but economic or technological reparability is so undefined that the term is no more useful than Damage Control. It is expected that many projects may have special demands for which criteria greater than Life Safety will be appropriate. Although not a level, per se, it is far simpler to reference this range of performance levels using a placeholder within the context of standard designations (e.g. SP-2) than to formally define both levels and ranges.*

- **Life Safety, SP-3:** The post-earthquake damage state in which significant damage to the structure may have occurred but in which some margin against either total or partial structural collapse remains. The level of damage is lower than that for the Structural Stability level. Major structural components have not become dislodged and fallen, threatening life safety either within or outside the building. While injuries during the earthquake may occur, the risk of life-threatening injury from structural damage is very low. It should be expected that extensive structural repairs will likely be necessary prior to reoccupation of the building, although the damage may not always be economically repairable. This level of structural performance is intended to be less than the level of performance expected of fully code compliant new buildings.
- **Limited Safety, SP-4:** This term is actually not a specific level but a range of post-earthquake damage states that are less than SP-3, Life Safety and better than SP-5, Structural Stability. It provides a placeholder

for the situation where a retrofit may not meet all the structural requirements of the Life Safety level, but is better than the level of Structural Stability. These circumstances include cases when the complete Life Safety level is not cost effective, or when only some critical structural deficiencies are mitigated
(The nonstructural performance level used in this range varies and will depend on the intent of the damage control).

- **Structural Stability, SP-5:** This level is the limiting post-earthquake structural damage state in which the building's structural system is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force resisting system. However, all significant components of the gravity load resisting system continue to carry their gravity demands. Although the building retains its overall stability, significant risk of injury due to falling hazards may exist both within and outside the building and significant aftershocks may lead to collapse. It should be expected that significant major structural repair will be necessary prior to reoccupancy. In the older concrete building types considered in this document, it is very likely that the damage will not be technically or economically repairable.

  Falling hazards are not specifically prevented to achieve this performance level. Therefore NP-E (nonstructural performance not considered) is normally combined with SP-5.

Commentary: *This level is provided primarily to enable a specific verification of continued structural stability for the maximum earthquake ground motions. Although such performance is implied (SEAOC 1990) for new buildings in California, and is considered desirable in all seismic regions, there has previously been no formalized method of verification. The combination of this structural performance level with earthquake ground shaking less than the maximum should be done with caution, as this could imply a high probability of collapse for any larger event*

- **Not Considered, SP-6:** This is not a performance level, but provides a placeholder for situations where only nonstructural seismic evaluation or retrofit is performed.

Commentary: *Although unusual, nonstructural seismic improvements are sometimes made with no review of the structure. This might occur in locations of high and obvious vulnerability, such as at a computer room or for important equipment. The explicit inclusion of a Not Considered structural performance level in the building performance level is also a useful communication tool between designer and owner.*

3.2.2 Nonstructural Performance Levels

Nonstructural performance levels are assigned a title and, for ease of reference, a letter. The letter is called the nonstructural performance letter and is abbreviated NP-n (where n is the designated letter).

The nonstructural performance levels - Operational, Immediate Occupancy, Life Safety, and Hazards Reduced - are discrete damage states and can be used directly in evaluation and retrofit procedures to define technical criteria. The other nonstructural performance designation - Not Considered - is an important placeholder to allow direct reference to the wide variety of building performance levels that might be desirable to owners for evaluation or retrofit.

Commentary: *Performance levels have been selected to allow combinations with structural levels that will correspond to single performance levels proposed by FEMA 273 and Vision 2000 and provide the flexibility to formalize de facto combined performance levels commonly used in practice.*

- **Operational, NP-A:** The post-earthquake damage state in which nonstructural elements and systems are generally in place and functional. Although minor disruption and cleanup should be expected, all equipment and machinery should be working. However, external utilities, which may not be available due to significant off-site damage, must be locally backed up. Contingency plans to deal with possible difficulties with external communication, transportation, and availability of supplies should be in place.

Although this level is defined here, development of design criteria must include building-specific planning and back-up systems and is beyond the scope of this document.

- **Immediate Occupancy, NP-B:** The post-earthquake damage state in which nonstructural elements and systems are generally in place. Minor disruption and cleanup should be expected, particularly due to damage or shifting of contents. Although equipment and machinery are generally anchored or braced, their ability to function after strong shaking is not considered and some limitations on use or functionality may exist. All external utilities may not be locally backed up. Seismic safety status should not be affected.
- **Life Safety, NP-C:** This post-earthquake damage state could include considerable damage to nonstructural components and systems but should not include collapse or falling of items heavy enough to cause severe injuries either within or outside the building. Secondary hazards from breaks in high-pressure, toxic, or fire suppression piping should not be present. Nonstructural systems, equipment, and machinery may not be functional without replacement or repair. While injuries during the earthquake may occur, the risk of life-threatening injury from nonstructural damage is very low.
- **Reduced Hazard, NP-D:** This post-earthquake damage state could include extensive damage to nonstructural components and systems but should not include collapse or falling of large and heavy items that could cause significant injury to groups of people, such as parapets, masonry exterior walls, cladding, or large, heavy ceilings. While isolated serious injury could occur, risk of failures that could put large numbers of people at risk within or outside the building is very low.

Commentary: Nonstructural elements have not been considered in any systematic manner in most retrofit work to date. Major hazards, however, are most often mitigated. This level is therefore an attempt to formalize common practice.

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

Commentary: This is not a performance level, but provides a designation for the common case where nonstructural elements are not surveyed or evaluated unless they have a direct effect on structural response, such as infill masonry walls or other heavy partitions. The designation is needed to accurately describe the Building Performance Level of Structural Stability for which nonstructural elements are, in fact, not considered. Also, it is included to allow it to be coupled with structural level SP-4, a building performance level often encountered. Furthermore, choosing not to consider nonstructural elements may sometimes be a risk management approach used in combination with other higher structural performance levels. The explicit inclusion of the NP-E, Not Considered nonstructural performance level in a building performance level is also a useful communication tool between designer and owner/operator.

3.2.3 Building Performance Levels

Combinations of a structural performance level and a nonstructural performance level form a Building Performance Level to completely describe the desired limiting damage state for a building. Possible combinations are shown in Table 3-1. The four most commonly referenced building performance levels are given titles and are described below.

Commentary: The four building performance levels; Operational, Immediate Occupancy, Life Safety, and Structural Stability; are given titles consistent with the titles of their structural and nonstructural components. Reference can also be made to their SP-NP designations, 1-A, 1-B, 3-C. and 5-E. Rather than developing titles for all other combinations, some of which will seldom be employed, it is recommended that the SP-NP designations be used.

- **Operational, 1-A:** This is the performance level related to functionality. Damage to the building's structure is limited so that continued safe occupancy is not in question, and any required repairs are minor and can be carried out without significant disruption to occupants. Similarly, damage to nonstructural systems and contents related to functionality is minor and will not jeopardize functions in the building. Most importantly, vital services from outside the building such as utilities, transportation, or communications must be provided with back-up facilities or planning as required to allow functions to continue if these services are unavailable. Since important aspects of this performance objective involve contingency planning and design of back-up systems, development of acceptability criteria is not included in this document.
- **Immediate Occupancy, 1-B:** This corresponds to the most widely used criteria for essential facilities. The building's spaces and systems are expected to be reasonably usable, but continuity of all services, either primary or backup, is not necessarily provided. Contents may be damaged.

Commentary: Although most codes require fairly complete and effective seismic anchorage and bracing for building systems and equipment, the actual operational aspects of a facility are normally developed by the owner-operator. Although some operational aspects, such as requirements for more complete back-up utility systems could be put in codes for use of building designers, it may not be practical to attain an Operational building performance level without significant facility-specific input from the owner-operator.

- **Life Safety, 3-C:** This level is intended to achieve a damage state that presents an extremely low probability of threats to life safety, either from structural damage or from falling or tipping of nonstructural building components. User-furnished contents, however, are not controlled, and could create falling hazards or secondary hazards, such as chemical releases or fire. This performance level is intended to be less than the performance that is expected of code designed new buildings.
- **Structural Stability, 5-E:** This damage state addresses only the main building frame or vertical load carrying system and requires only stability under vertical loads. No margin against collapse in aftershocks may be available. Life threatening external or internal falling hazards from cladding, nonstructural finishes, or even structural damage may have occurred. Review of performance of nonstructural elements from expected forces or structural drifts is not required so their performance can be highly unreliable.
- **Other Commonly Used Combinations**
  - **Building Performance Level 3-D:** This level combines life safety structural performance with the reduced hazard nonstructural performance, thus accepting a slight risk to life safety from nonstructural systems. Although large and highly vulnerable nonstructural elements should remain in place, the majority of nonstructural elements such as mechanical/electrical equipment and distribution systems, partitions, and typical

ceilings and light fixtures have not been braced or anchored and could be highly disrupted and produce falling hazards.

Commentary: *The vast majority of all retrofits completed to date have been designed to provide performance similar to this level for the code prescribed 10 percent/50 year earthquake ground motion. A common design criteria has incorporated 75% of the lateral force required for new buildings. Although most of these retrofits were designed to prescriptive provisions and were therefore not performance based, the intent was to achieve structural life safety, and included mitigation of major and obvious falling hazards such as parapets or heavy decorative ceiling in large rooms. Extensive surveys of other nonstructural elements rarely have been included.*

- **Building Performance Level 3-B:** This level presents a risk of structural damage that could prevent the building from being occupied. However, nearly complete nonstructural protection will prevent significant internal disruption, particularly in low levels of shaking. Although only seldom applied to a whole building, this level is more commonly applied to particular areas or rooms, such as computer facilities.

Commentary: *Building Performance Level 3-B might be assigned to buildings where occupancy and/or function is important, but the cost of providing more structural protection is high, or if the structure already meets the Life Safety performance level. In these cases, structural work may be extremely undesirable, but added nonstructural protection is judged cost effective.*

- **Other Less Common Combinations**
  - **Building Performance Level 1-C:** This combination might represent the desire to avoid a red or yellow tag structurally, but the willingness to accept considerable cleanup before the building's spaces are fully usable. (It is unlikely that SP-1 would be combined with less than NP-C.
  - **Building Performance Levels 2-A, 2-B, 2-C, 2-D:** Within the broad structural performance range of Damage Control, an equally wide range of nonstructural protection might be appropriate. It is unlikely, but possible, to find it desirable to combine structural Damage Control with less thorough nonstructural protection than NP-C. It is necessary to develop building specific criteria for these cases to suit the systems needing special protection.
  - **Building Performance Level 3-A:** This building code requirements for new buildings is generally thought to be intended to provide seismic performance similar to Performance Level 2C for the 10 percent/50 years ground motion.
  - **Building Performance Level 3-E:** This level might be used if structural work is minor or localized, or if limited funding precludes expenditures for extensive nonstructural upgrading.
  - **Building Performance Levels 4-C, 4-D, 4-E:** Similar to SP-4, these levels are primarily placeholders for structural risk reduction that does not meet a predescribed level. A variety of nonstructural improvements may also be made.
  - **Building Performance Levels 5-C, 5-D, 6-C, 6-D:** In some cases, improved nonstructural performance may be desirable with little or no consideration of structural performance. As mentioned previously, use of the SP-NP designation forces recognition of structural risks when such decisions are made.
- **Not Recommended Combinations:** Certain combinations of structural and nonstructural performance levels will seldom, if ever, be cost effective because of an imbalance in effort on structural and nonstructural systems. Such imbalances may also create misperceptions of

expectations by owners or tenants. Although assignment of these building performance levels is not prohibited, the *Not Recommended* designation was added both to avoid poor decisions and to simplify Table 3-1.

3.3 Earthquake Ground Motion

Earthquake ground motion is combined with a desired performance level to form a performance objective. The earthquake ground motion can be expressed either by specifying a level of shaking associated with a given probability of occurrence (a probabilistic approach), or in terms of the maximum shaking expected from a single event of a specified magnitude on a specified source fault (a deterministic approach). The level of ground motion is expressed in terms of engineering characteristics for use in design. A response spectra or an equivalent series of simulated recordings of earthquake motions are used for this purpose.

> Earthquake Ground Motion =
> engineering characteristics of the shaking
> at the site for a given earthquake
> or:
> a level of shaking that has a certain
> probability of occurring

The following three levels of earthquake ground motion are defined in Chapter 4.

- **The Serviceability Earthquake (SE):** Ground motion with a 50 percent chance of being exceeded in a 50-year period
- **The Design Earthquake (DE):** Ground motion with a 10 percent chance of being exceeded in a 50-year period
- **The Maximum Earthquake (ME):** Maximum level of ground motion expected within the known geologic framework due to a specified single event (median attenuation), or the ground motion with a 5 percent chance of being exceeded in a 50-year period

Commentary: *The ground motion with a 20 percent chance of being exceeded in 50 years has also been used in some projects. This ground motion typically represents about two-thirds to three-quarters of the demand of the more standard DE of 10 percent/50 years. This reduced seismic demand is roughly equivalent to the lower design force level often used in the past for evaluation and retrofit of existing buildings, e.g., ATC-14 (ATC 1987), FEMA 178 (BSSC 1992), the San Francisco Building Code Section 104f (City and County of San Francisco 1991).*

Although the recommended system of creating and assigning performance objectives allows and encourages use of many levels of performance and ground motions, a common measure is needed to enable comparison of performance objectives with that expected from familiar designs, such as new buildings. The most common and consistent thread for designs for the last 20 years is the 10 percent/50 years ground motion and it is recommended that this motion be maintained for the basic Design Earthquake. Lower criteria that have often been used for existing building, usually in consideration of the high cost of retrofit, should be taken into account directly by setting appropriate performance levels. As noted earlier, the Life Safety Performance Level is intended to be a lesser criteria than the code for new buildings and to accomplish a purpose similar to the use of smaller design forces. However, with performance based design, it is hoped that communication of expectations to owners and tenants will be more straightforward.

3.4 Performance Objectives

A seismic performance objective is defined by selecting a desired building performance level for a given level of earthquake ground motion, as shown in Table 3-2a.

A dual- or multiple-level performance objective can be created by selecting two or more different desired performances, each for a different level of ground motion, as shown in Table 3-2b.

Table 3-2a. Definition of a Performance Objective

Defining a Performance Objective				
	Building Performance Level			
EQ Ground Motion	***Operational***	***Immediate Occupancy***	***Life Safety*** ↓	***Structural Stability***
Serviceability EQ (SE)				
Design EQ (DE) →			✓	
Maximum EQ (ME)				

Table 3-2b. Definition of a Dual-Level Performance Objective

Defining a Dual-Level Performance Objective				
	Building Performance Level			
EQ Ground Motion	***Operational*** ↓	***Immediate Occupancy***	***Life Safety*** ↓	***Structural Stability***
Serviceability EQ (SE) →	✓			
Design EQ (DE) →			✓	
Maximum EQ (ME)				

Table 3-3. The Basic Safety Performance Objective

The Basic Safety Objective				
	Building Performance Level			
EQ Ground Motion	**Operational**	**Immediate Occupancy**	**Life Safety** ↓	**Structural Stability** ↓
Serviceabilty EQ (SE)				
Design EQ (DE) →			✓	
Maximum EQ (ME) →				✓

3.4.1 Basic Safety Objective

The Basic Safety Objective, shown in Table 3-3, is a dual-level performance objective defined as performance achieving the Building Performance Level Life Safety, 3-C, for the Design Earthquake level of ground motion and the Building Performance Level Structural Stability, 5-E, for the Maximum Earthquake level of ground motion.

Commentary: This performance objective is intended to be an enhanced substitute for the Substantial Life Safety performance goal shown in Table 1 of Policy on Acceptable Levels of Earthquake Risk in State Buildings (CSSC 1991b).

Consideration of seismic performance for the ME has been included in several codes (Army 1986; Title 24 for hospitals (CBSC 1995)), but has been ineffectual due to lack of adequate criteria as well as lack of clarity as to intent. It has often been suggested for use nationally to account for the relatively large variation in the ratio of DE to ME across the country. Primarily due to the lack of reliability and the potential brittleness of many systems in existing or retrofit buildings, it is included in FEMA 273 for the nationally recommended Basic Safety Objective. A complete set of acceptability criteria for all materials has also been developed.

The dual-level check may prove unnecessary in California for small, simple buildings or buildings with no brittle elements or at sites with small differences between the DE and the ME. However, in complex buildings that incorporate lateral force resisting elements of widely different characteristics, the range of performance from Life Safety to Structural Stability damage levels is likely to be inconsistent and unpredictable, and a specific check against collapse is probably warranted. Much of the central valley of California is characterized by a significant difference between the DE and the ME, based on actual probabilistic values for the DE (Code seismic zones for these areas are artificially high). This would suggest that a specific check against collapse for a rare large event (ME) should be made if the actual probabilistic values are used for the DE.

The details of exceptions to a dual level criteria for the Basic Safety Objective cannot be developed prior to completion of additional trial designs and fine tuning of verification criteria.

3.4.2 Other Performance Objectives

The wide variety of building performance levels (Table 3-1) can be combined with various levels of ground motion to form many possible performance objectives. Performance objectives for any building may be assigned using functional, policy, preservation, or cost considerations.

Commentary: *Combinations that have been used as performance objectives in the past or that otherwise may form logical objectives are shown in Tables 3-4a, 3-4b, and 3-4c.*

Table 3-5. Seismic Performance Expectations

A. Earthquake Performance of Structural Systems					
	Damage/Risk of Injury				
Earthquake Effects[1,2]	*SP-1 Very Limited*	*SP-2 Minor-Moderate*	*SP-3 Significant*	*SP-4 Major-Extensive*	*SP-5 Substantial (No Collapse)*
	Negligible	*Minor-Low*	*Low*	*Moderate*	*Significant*
Low-Moderate (SE)					
Moderate-Severe (DE)					
Severe-Very Severe (ME)					

B. Approximate Time to Reoccupy[3]					
Earthquake Effects[1,2]	*Immediate (hours)*	*Short (weeks-months)*	*Moderate (months-1 year)*	*Long (more than 1 year)*	*Very Long (perhaps never)*
Low-Moderate (SE)					
Moderate-Severe (DE)					
Severe-Very Severe (ME)					

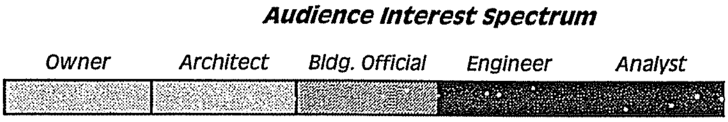
C. Earthquake Performance of Nonstructural Systems					
	Damage				
Earthquake Effects[1,2]	*NP-A Negligible*	*NP-B Minor-Moderate*	*NP-C Moderate-Considerable*	*NP-D Extensive (No major Collapse)*	*NP-E Not Considered*
Low-Moderate (SE)					
Moderate-Severe (DE)					
Severe-Very Severe (ME)					

D. Function Continuance: Nonstructural Systems[4]					
Earthquake Effects[1,2]	*Immediate (hours or next day)*	*Short (few days-one month)*	*Moderate (months-1 year)*	*Long (more than 1 year)*	*Not Considered (perhaps never)*
Low-Moderate (SE)					
Moderate-Severe (DE)					
Severe-Very Severe (ME)					

Notes:
1. Ground Motion Effects of Nearby Earthquakes:
    Low-Moderate Shaking: (SE) Serviceability Earthquake (50%/50 yrs)
    Moderate-Severe Shaking: (DE) Design Earthquake (10%/50 yrs)
    Severe-Very Severe Shaking: (ME) Maximum Earthquake (5%/50 yrs)
2. Classification of earthquake effects and extent of anticipated damage may be modified by site conditions—such as poor soils, ground failure potential, or vulnerable adjacent structures—which may result in stronger shaking and greater damage.
3. Time to reoccupy building is not necessarily directly related to specific structural performance levels.
4. Time to restore functions is not necessarily directly related to specific nonstructural performance levels.

Chapter 4

Seismic Hazard



4.1 Scope

This chapter provides guidelines for quantifying seismic hazard at a site due to ground shaking for each of the three earthquake hazard levels:

- The Serviceability Earthquake (SE)
- The Design Earthquake (DE)
- The Maximum Earthquake (ME)

Commentary: Seismic ground shaking is defined using site soil factors and other terms that have been developed by the SEAOC Seismology Committee as part of a code change proposal (ICBO 1996) for the 1997 Edition of the Uniform Building Code or UBC (ICBO 1994). It is expected that this proposal after review and refinement will be adopted both for the 1997 UBC and for related editions of the California Building Code or CBC (CBSC 1995).

This Section also provides guidelines for determining when the seismic hazard due to ground failure warrants consideration, although the evaluation of such hazards is beyond the scope of this document.

Commentary: The focus of this document is on ground shaking since ground shaking is the predominant cause of earthquake damage to buildings. Ground failure hazard is also included, since ground failure should also be considered for sites susceptible to liquefaction, landsliding or surface rupture.

4.2 Earthquake Ground Shaking Hazard Levels

Three levels of earthquake hazard are used to define ground shaking: the Serviceability Earthquake, the Design Earthquake, and the Maximum Earthquake. These levels of earthquake hazard are defined below in the following sections.

Commentary: The Design Earthquake and Maximum Earthquake hazard levels are based on UBC (and CBC) definitions of ground shaking that will likely remain unchanged until sometime after the year 2000. Other definitions of seismic criteria have been developed for incorporation into the 1997 edition of the NEHRP Provisions (BSSC 1996) for design of new buildings and will also likely be adopted by the FEMA Guidelines (ATC 1996a) for seismic rehabilitation of existing buildings.

4.2.1 Serviceability Earthquake

The Serviceability Earthquake (SE) is defined probabilistically as the level of ground shaking that has a 50 percent chance of being exceeded in a 50-year period. This level of earthquake ground shaking is typically about 0.5 times the level of ground shaking of the Design Earthquake.

Commentary: The SE represents a frequent level of ground shaking that is likely to be felt during the life of the building. The SE has a mean return period of approximately 75 years.

4.2.2 Design Earthquake

The Design Earthquake (DE) is defined probabilistically as the level of ground shaking that has a 10 percent chance of being exceeded in a 50-year period.

Commentary: The DE represents an infrequent level of ground shaking that can occur during the life of the building. The DE has a mean return period of approximately 500 years. The DE has the same definition as the level of ground shaking currently used as the basis for the seismic design of new buildings by the UBC and the CBC.

4.2.3 Maximum Earthquake

The Maximum Earthquake (ME) is defined deterministically as the maximum level of earthquake ground shaking which may ever be expected at the building site within the known geologic framework. In Seismic Zones 3 and 4, this intensity of ground shaking may be calculated as the level of earthquake ground motion that has a 5 percent probability of being exceeded in a 50-year time period. This level of ground shaking is typically about 1.25 to 1.5 times the level of ground shaking of the Design Earthquake.

Commentary: The ME has the same definition as the Maximum Capable Earthquake (MCE) required by the CBC for design of hospitals and by both the CBC and UBC for design and testing of buildings with base isolation systems. This earthquake definition is intended to represent an upper-bound on the level of ground shaking that could be reasonably expected to occur at the building site.

The definition of the ME (and the MCE of the UBC and CBC) is substantially different from the definition of the Maximum Considered Earthquake proposed for both the 1997 NEHRP Provisions and the FEMA Guidelines for rehabilitation of existing buildings. In probabilistic terms, the ME has a return period of about 1,000 years, whereas the Maximum Considered Earthquake has a return period of about 2,500 years (i.e., ground shaking with a 2% probability of being exceeded in 50 years).

4.3 Ground Failure

Ground failure can be the result of the following hazards:

- Liquefaction
- Landsliding
- Surface fault rupture

Liquefaction and landsliding are discussed in the following sections, and guidelines (triggers) are provided for determining when a detailed study of these hazards might be warranted. In general, surface expression of fault rupture below a building is considered too remote a possibility to warrant design consideration.

Commentary: Although unlikely, buildings situated very close to active faults could be destroyed by the surface expression of fault rupture. It is recommended that special consideration be given to buildings located within the Special Studies Zone (Alquist-Priolo Act, January 31, 1979). Relocation, rather than retrofit, may be more appropriate for buildings straddling the trace of an active fault (CDMG 1985).

4.3.1 Liquefaction

Liquefaction can occur in certain types of saturated soils that are shaken strongly enough and long enough for the soil to lose a substantial amount of strength (because of high pore water pressure). Liquefaction can cause settlement as well as lateral spreading or slides of certain soils. In either case, permanent ground surface deformation occurs that can cause the foundation, or a portion of the foundation, of the building to settle or displace downward and/or laterally.

The site's susceptibility to liquefaction is typically described by the terms: *very high, high,*

Table 4-1. Ground Shaking Levels at Which Liquefaction Should Be Considered Possible or Likely

Liquefaction Susceptibility	Effective Peak Acceleration Level (g)	
	Liquefaction is Possible	Liquefaction is Likely
Very high	Any	0.15
High	0.15	0.2
Moderate	0.2	0.3
Low	0.4	Not considered likely
Very low	Not considered possible	Not considered likely

moderate, low, and very low. Table 4-1 (adapted from NIBS 1996) provides guidance for determining the level of ground shaking at which liquefaction should be considered. The level (or levels) of ground motion used to evaluate the likelihood of liquefaction should be the same as that (those) used to perform ground shaking analyses.

Liquefaction should be considered possible at any level of ground shaking when the site has a very high susceptibility to liquefaction, and conversely, liquefaction should not be considered possible (or likely) even for high levels of ground shaking when the site has a very low susceptibility to liquefaction. For sites of moderate liquefaction susceptibility, Table 4-1 indicates that the effective peak acceleration (EPA) of the ground must be at least 0.2g for liquefaction to be considered possible and at least 0.3g for liquefaction to be considered likely.

The guidance given in Table 4-1 is based on the work of Liao, Veneziano, and Whitman (1988) as modified and incorporated into the national earthquake loss estimation methodology being developed by the National Institute of Building Sciences for the Federal Emergency Management Agency (NIBS 1996). The EPA level at which liquefaction is considered "possible" is the level at which, approximately, a 15 percent or greater chance of liquefaction has been determined to exist at the site. Similarly, the EPA level at which liquefaction is considered to be "likely" is the level at which a 50 percent or greater chance of liquefaction has been determined to exist. These EPA levels have been determined based on the assumptions that the site has a relatively high water table (within 10 - 20 feet of the surface) and that shaking is due to a relatively large magnitude event ($M_w \geq 6.5$). Small magnitude events are not expected to shake the ground long enough to trigger significant liquefaction.

Liquefaction susceptibility may be identified from maps developed by the California Division of Mines and Geology (and others), although susceptibility maps are not yet available for most areas within California. For areas that do not have existing liquefaction susceptibility maps, liquefaction susceptibility may be estimated on the basis of the soil type and geologic conditions of the site. A method for rating relative liquefaction susceptibility on the basis of the general depositional environment and the geologic age of deposits has been developed by Youd and Perkins (1978).

A geotechnical engineer would be required to determine soil/geologic conditions at the site and to determine the liquefaction susceptibility. For sites where liquefaction is considered to be possible or likely, the geotechnical engineer should be required to evaluate the amount of permanent ground deformation expected at the site and its effect on the foundation of the building. It is recommended that the geotechnical engineer investigate liquefaction effects for the ME, even if

Table 4-2. Ground Shaking Levels at Which Landsliding Should be Considered Possible

	Effective Peak Acceleration (g) at Slope Angles:						
Geologic Group	*0°–5°*	*5°–10°*	*10°–15°*	*15°–25°*	*25°–35°*	*35°–45°*	*≥ 45°*
Dry Conditions (Groundwater Below Level of Slope)							
Strongly cemented rock (crystalline rock and well-cemented sandstone)	None	None	None	0.7	0.5	0.3	Any
Weakly cemented rock and soil (sandy soils and poorly cemented sandstone)	None	None	0.5	0.4	0.2	0.1	Any
Argillaceous rock (shales, clayey soil, existing landslides, poorly compacted fills)	None	0.3	0.2	0.1	Any	Any	Any
Wet Conditions (Groundwater Level At Ground Surface)							
Strongly cemented rock (crystalline rock and well-cemented sandstone)	None	None	0.5	0.4	0.2	0.1	Any
Weakly cemented rock and soil (sandy soils and poorly cemented sandstone)	None	0.3	0.2	0.1	Any	Any	Any
Argillaceous rock (shales, clayey soil, existing landslides, poorly compacted fills)	None	0.1	Any	Any	Any	Any	Any

the ME is not required for ground shaking analysis.

4.3.2 Landsliding

Earthquake-induced landsliding of a slope can occur when earthquake and gravity forces within the slide mass temporarily exceed slope stability. The value of ground acceleration within the slide mass required to initiate slope instability is called the critical or yield acceleration. Landsliding can cause relatively minor slides or affect a large hillside. Typically, slide displacement accumulates (gets larger and larger) with each cycle of earthquake shaking that exceeds the critical acceleration level. Large landslides can affect buildings situated on the slide mass as well as buildings just below the slide.

Whether landsliding should be considered in the design depends on the site's susceptibility to landsliding, which depends on the soil/geologic conditions, the slope angle, and the critical acceleration (i.e., the level of shaking required to initiate landsliding). Table 4-2 (adapted from NIBS 1996) provides guidance for determining when landsliding should be considered; the factors to be considered are site susceptibility and shaking level. The level (or levels) of ground motion used to evaluate landsliding should be the same as that (those) used to perform ground shaking analyses.

According to Table 4-2, landsliding should be considered possible in strongly cemented rock (with dry slope conditions) only when the EPA exceeds 0.7g for 15^0 - 25^0 slopes, 0.5g for 25^0 - 35^0 slopes, and 0.3g for 35^0 - 45^0 slopes; and it should be considered possible for all EPA values when slope angle exceeds 45^0. Conversely, Table 4-2 indicates that landsliding should be considered possible in existing landslide areas (with wet slope conditions) for all significant EPA levels, except for sites that are essentially flat (i.e., with a slope angle less than 5^0).

The guidance given above is based on relationships developed by Wilson and Keefer (1985) as modified and incorporated into the national earthquake loss estimation methodology (NIBS 1996). The work of Wilson and Keefer conservatively represents the most landslide-susceptible geologic types likely to be found in a geologic group and may be considered to represent a shaking level (EPA) that has about a 25 percent chance of producing a landslide at a given site.

A geotechnical engineer would be required to determine the soil/geologic conditions at the site and to determine the landsliding susceptibility. For sites where it is considered possible to have landsliding, the geotechnical engineer would be required to evaluate the extent of landsliding expected and the effect of such landsliding on the foundation of the building.

It is recommended that the geotechnical engineer investigate landsliding effects for the ME, even if the ME is not required for ground shaking analysis.

4.4 Primary Ground Shaking Criteria

This section specifies the primary ground shaking criteria for the evaluation of buildings. Primary ground shaking criteria are those criteria that will be required for the design of all buildings. Primary criteria include the following:

- Site geology and soil characteristics
- Site seismicity characteristics
- Site response spectra

Commentary: Site geology and soil characteristics and site seismicity characteristics are based directly on the requirements proposed by the SEAOC Seismology Committee (ICBO 1996) for the 1997 UBC. These requirements are repeated (with editorial modification) in the following sections.

4.4.1 Site Geology and Soil Characteristics

4.4.1.1 General

Each site is assigned a soil profile type based on properly substantiated geotechnical data using the site categorization procedure of Section 4.4.1.3.

Exception: When the soil properties are not known in sufficient detail to determine the Soil Profile Type, Type S_D may be used. Soil Profile Types S_E or S_F need not be assumed unless the building official determines that Types S_E or S_F may be present at the site or in the event that Types S_E or S_F are established by geotechnical data.

4.4.1.2 Soil Profile Type

Soil Profile Types S_A, S_B, S_C, S_D, S_E are defined in Table 4-3 (adapted from ICBO 1996). Soil Profile Type S_F is defined as soils requiring site-specific evaluation, as follows:

- Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils
- Peats and/or highly organic clays where the thickness of peat or highly organic clay exceeds 10 feet
- Very high plasticity clays with a plasticity index greater than 75 (PI > 75) and where the depth of clay exceeds 25 feet
- Very thick soft/medium-stiff clays where the depth of clay exceeds 120 feet

Table 4-3. Soil Profile Types

Soil Profile Type	*Soil Profile Name/Generic Description*	*Average Soil Properties for Top 100 Feet of Soil Profile*		
		Shear Wave Velocity, $\bar{v}_s$ (feet/second)	*Standard Penetration Test, $\bar{N}$ [or N_{CH} for cohesionless soil layers] (blows/foot)*	*Undrained Shear Strength, $\bar{s}_u$ (psf)*
SA[1]	Hard Rock	$\bar{v}_s > 5{,}000$	Not Applicable	
SB	Rock	$2{,}500 < \bar{v}_s \leq 5{,}000$	Not Applicable	
SC	Very Dense Soil and Soft Rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	$\bar{N} > 50$	$\bar{s}_u > 2{,}000$
SD	Stiff Soil Profile	$600 \leq \bar{v}_s \leq 1{,}200$	$15 \leq \bar{N} \leq 50$	$1{,}000 \leq \bar{s}_u \leq 2{,}000$
SE[2]	Soft Soil Profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1{,}000$
SF[3]	Soil Requiring Site-Specific Evaluation			

1 Soil profile type SA (hard rock) is not applicable to sites in California.

2 Soil profile type SE also includes any soil profile with more than 10 feet of soft clay defined as a soil with PI > 20, WMC ≥ 40 % and $\bar{s}_U < 500$ psf. The plasticity index (PI) is determined in accordance with ASTM D4318-93 and the moisture content (WMC) is determined in accordance with ASTM D2216-92.

3 See Section 4.4.1.2 for description of soils requiring site-specific evaluation.

4.4.1.3 Site Categorization

The soil profile type for the site is established by using the following procedure.

Step 1: Check for the four categories of Soil Profile SF requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Soil Profile SF and conduct a site-specific evaluation.

Step 2: Check for the existence of a total thickness of soft clay > 10 ft, where a soft clay layer is defined by: undrained shear strength, $\bar{s}_U < 500$ psf, moisture content, WMC≥ 40 percent, and plasticity index, PI > 20. If all of these criteria are satisfied, classify the site as Soil Profile Type SE.

The plasticity index (PI) is determined in accordance with ASTM D4318-93 and the moisture content (WMC) is determined in accordance with ASTM D2216-92.

Step 3: Categorize the site using one of the following three methods with average soil profile properties computed in all cases in accordance with Section 4.4.1.4 and the criteria of Table 4-3.

- Use $\bar{v}_S$ for the top 100 feet of the soil profile (the $\bar{v}_S$ method)
- Use $\bar{N}$ for the top 100 feet of the soil profile (the $\bar{N}$ method)
- Use $\bar{N}_{CH}$ for cohesionless soil layers (PI < 20) and $\bar{s}_U$ for cohesive soil layers in the top 100 feet of the soil profile. If $\bar{N}_{CH}$ and $\bar{s}_U$ criteria differ, the soil profile with the larger seismic coefficient is used for design ($\bar{s}_U$ method)

The shear wave velocity for rock (soil profile type SB) is either measured on site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock is either measured on site for shear wave velocity or classified as soil profile type Sc.

The soil profile type should not be taken as rock (soil profile type SB) if there is more than 10 feet of soil between the rock surface and the bottom of the spread footing or mat foundation.

4.4.1.4 Average Soil Properties

Average soil properties are calculated by using the formulas of this section. Soil profiles containing distinctly different soil layers should be subdivided into those layers. Each soil layer is designated by a number that ranges from i = 1 at the top to i = n at the bottom, where there are a total of n distinct layers in the upper 100 feet of the soil profile.

The average shear wave velocity, $\overline{v}_S$, is determined by the following formula:

$$\overline{v}_S = \frac{d_S}{\sum_{i=1}^{n} \frac{d_i}{v_{S,i}}} \qquad (4\text{-}1)$$

where: d_i = thickness of layer i, in feet

d_S = total thickness of soil profile (100 feet)

$v_{S,i}$ = shear wave velocity of layer i, in feet/sec.

The average standard penetration resistance, $\overline{N}$ or $\overline{N}_{CH}$, is determined by the following formulas:

$$\overline{N} = \frac{d_s}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad (4\text{-}2)$$

$$\overline{N}_{CH} = \frac{d_{CH}}{\sum_{i=1}^{n} \frac{d_{CH,i}}{N_{CH,i}}} \qquad (4\text{-}3)$$

where: d_{CH} = total thickness of all cohesionless soil layers, in feet

$d_{CH,i}$ = thickness of cohesionless soil layer i, in feet

N_i = standard penetration resistance of layer i, directly measured in the field without corrections in accordance with ASTM D1586-84, but not to exceed 100 blows/ft

$N_{CH,i}$ = standard penetration resistance of cohesionless soil layer i, directly measured in the field without corrections in accordance with ASTM D1586-84, but not to exceed 100 blows/ft.

The average undrained shear strength, $\overline{s}_U$, is determined by the following formula:

$$\overline{s}_U = \frac{d_c}{\sum_{i=1}^{n} \frac{d_i}{s_{U,i}}} \qquad (4\text{-}4)$$

where: d_C = total thickness of cohesive soil layers, in the top 100 feet (100 - dCH)

$s_{U,i}$ = undrained shear strength, in psf, measured in accordance with ASTM D2166-91 or D2850-87, but not to exceed 5000 psf.

4.4.2 Site Seismicity Characteristics

4.4.2.1 General

Seismicity characteristics for the site are based on the seismic zone, the proximity of the site to active seismic sources, and site soil profile characteristics.

4.4.2.2 Seismic Zone

Each site is assigned a seismic zone in accordance with the requirements of the *California Building Code* (CBSC 1995). Each structure is assigned a seismic zone factor Z, in accordance with Table 4-4.

Commentary: *Traditionally, all of California has been classified as either seismic zone 3 or 4, although the ground shaking hazard at sites in seismic zone 3 situated far from active faults may be significantly overestimated by a seismic zone factor of Z= 0.3. For these sites, response spectra based on contour maps or site-specific hazard analysis would be expected to be significantly less than response spectra based on Z= 0.3.*

4.4.2.3 Near-Source Factor

Each site is assigned a near-source factor in accordance with Table 4-5 (ICBO 1996) and based on seismic source type, as specified in Table 4-6 (ICBO 1996).

Commentary: *Values of the near source factor given in Table 4-5 may significantly underestimate ground shaking at certain near-source sites. In the direction normal to the plane of fault rupture, ground shaking may be as much as 50 percent greater than that predicted using the N factors of Table 4-5 (Somerville 1996).*

4.4.2.4 Seismic Coefficients

For each earthquake hazard level, the structure is assigned a seismic coefficient C_A in accordance with Table 4-7 (ICBO 1996) and a seismic coefficient C_V in accordance with Table 4-8 (ICBO 1996). In lieu of a site-specific seismic hazard analysis, the seismic coefficient, C_A, may be taken to be the default value of the effective peak acceleration (EPA) of the ground.

4.4.3 Elastic Site Response Spectra

Elastic response spectra for a site are based on estimates of C_A and C_V using one, or more, of the following:

- Site seismic coefficients (Tables 4-7 and 4-8)
- Spectral contour maps (developed by the USGS for Project 97 (Frankel et al. 1996))
- Site-specific hazard analysis studies

The construction of elastic response spectra using estimates of C_A and C_V is described in Section 4.4.3.3.

Commentary: *In all cases, elastic site response spectra are described by a standard (two-domain) shape defined by the coefficients C_A and C_V. Elastic response spectra are described by a standard shape to simplify the application of these spectra to nonlinear static analysis procedures (Chapter 8).*

4.4.3.1 Spectral Contour Maps (USGS)

Spectral contour maps for rock sites developed by the USGS for Project 97 (Frankel et al. 1996) may be used to construct elastic response spectra for a site, provided the basis for these maps is consistent with the definition of the earthquake level(s) of interest (Section 4.2).

For sites situated on soil type S_B, the value of C_A should be taken to be equal to 0.4 times the spectral response acceleration (units of g) at a period of 0.3 seconds and the value of C_V should be taken to be equal to 1.0 times the spectral response acceleration (units of g) at a period of 1.0 second. Alternatively for sites situated on soil S_B, ME values of C_A and C_V may be based on the formulas:

$$C_A = 0.4 S_{MS} \tag{4-5}$$

$$C_V = S_{M1} \tag{4-6}$$

where:

S_{MS} = spectral acceleration in the short-period range for Site Class B for MCE, as prescribed by 1997 NEHRP Provisions (BSSC 1996).

S_{M1} = spectral acceleration at a 1.0-second period for Site Class B for MCE, as prescribed by 1997 NEHRP Provisions (BSSC 1996).

For sites situated on other soil types, the values of C_A and C_V based on soil type S_B should be increased in proportion to the increase in the site coefficients of Tables 4-7 and 4-8, respectively, for the soil type of the site.

The values of C_A and C_V should not be taken as less than 80 percent, and need not be taken to be greater than 100 percent, of the values specified in Table 4-7 and Table 4-8, respectively.

Table 4-4. Seismic Zone Factor Z

Zone	1[1]	2A[1]	2B[1]	3	4
Z	0.075	0.15	0.20	0.30	0.40

1 Seismic zones 1, 2A and 2B are not applicable to sites in California.

Table 4-5. Near Source Factor, N_A and N_V[1]

Seismic Source Type	Closest Distance to Known Seismic Source[2,3]							
	≤ 2 km		5 km		10 km		≥ 15 km	
	N_A	N_V	N_A	N_V	N_A	N_V	N_A	N_V
A	1.5	2.0	1.2	1.6	1.0	1.2	1.0	1.0
B	1.3	1.6	1.0	1.2	1.0	1.0	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

1 The near-source factor may be based on the linear interpolation of values for distances other than those shown in the table.

2 The location and type of seismic sources to be used for design shall be established based on approved geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

3 The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the near-source factor considering all sources shall be used for design.

Table 4-6. Seismic Source Type

		Seismic Source Definition	
Seismic Source Type	Seismic Source Description	Maximum Moment Magnitude, M	Slip Rate, SR (mm/year)
A	Faults that are capable of producing large magnitude events and which have a high rate of seismic activity	$M \geq 7.0$	$SR \geq 5$
B	All faults other than types A and C	Not Applicable	Not Applicable
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity	$M < 6.5$	$SR < 2$

Table 4-7. Seismic Coefficient, C_A

	Shaking Intensity, $ZEN^{1,2}$					
Soil Profile Type	***= 0.075***	***= 0.15***	***= 0.20***	***= 0.30***	***= 0.40***	***> 0.40***
S_B	0.08	0.15	0.20	0.30	0.40	1.0(ZEN)
S_C	0.09	0.18	0.24	0.33	0.40	1.0(ZEN)
S_D	0.12	0.22	0.28	0.36	0.44	1.1(ZEN)
S_E	0.19	0.30	0.34	0.36	0.36	0.9(ZEN)
S_F	Site-specific geotechnical investigation required to determine C_A					

1 The value of E used to determine the product, ZEN, should be taken to be equal to 0.5 for the Serviceability Earthquake, 1.0 for the Design Earthquake, and 1.25 (Zone 4 sites) or 1.5 (Zone 3 sites) for the Maximum Earthquake.

2 Seismic coefficient C_A should be determined by linear interpolation for values of the product ZEN other than those shown in the table.

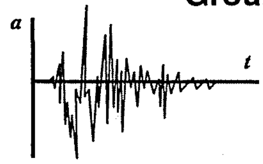
Table 4-8. Seismic Coefficient, C_V

	Shaking Intensity, $ZEN^{1,2}$					
Soil Profile Type	***= 0.075***	***= 0.15***	***= 0.20***	***= 0.30***	***= 0.40***	***> 0.40***
S_B	0.08	0.15	0.20	0.30	0.40	1.0(ZEN)
S_C	0.13	0.25	0.32	0.45	0.56	1.4(ZEN)
S_D	0.18	0.32	0.40	0.54	0.64	1.6(ZEN)
S_E	0.26	0.50	0.64	0.84	0.96	2.4(ZEN)
S_F	Site-specific geotechnical investigation required to determine C_V					

1 The value of E used to determine the product, ZEN, should be taken to be equal to 0.5 for the Serviceability Earthquake, 1.0 for the Design Earthquake and 1.25 (Zone 4 sites) or 1.5 (Zone 3 sites) for the Maximum Earthquake.

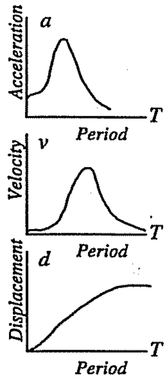
2 Seismic coefficient C_V should be based on the linear interpolation of values for shaking intensities other than those shown in the table.

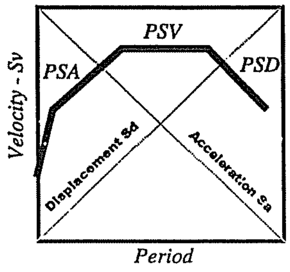
Ground Motion & Response Spectra



Ground motion recordings (accelerograms) indicate that ground shaking is an extremely complex waveform, containing oscillatory motion components over a broad range of frequencies.

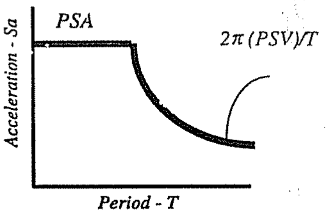
By performing a time history analysis of a structure it is possible to determine the peak acceleration, velocity and displacement of the structure's response to a ground motion. If such analyses are performed for a series of single degree of freedom structures, each having a different period, *T*, and the peak response accelerations, velocities and displacements are plotted vs. the period of the structures, the resulting graphs are termed respectively acceleration, velocity and displacement response spectra.

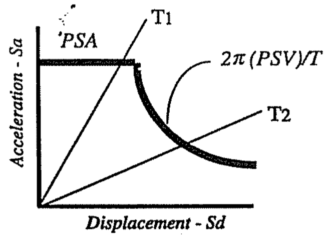




Researchers commonly display response spectra on a 3-axis plot known as a tri-partite plot in which peak response acceleration, velocity and displacement are all plotted simultaneously against structural period. Researchers (Newmark and Hall, 1982) have found that response spectra for typical records can be enveloped by a plot with three distinct ranges: a constant peak spectral acceleration (PSA), constant peak spectral velocity (PSV) and constant peak spectral displacement (PSD).

Response spectra contained in the building code indicate the constant acceleration and velocity ranges plotted in an acceleration vs period domain. This is convenient to the code design procedure which is based on forces (or strength) which are proportional to acceleration.





For nonlinear analysis, both force and deformation are important. Therefore, spectra are plotted in an acceleration vs. displacement domain, which has been termed ADRS (acceleration-displacement response spectra) (Mahaney et al.,1993). Period in these ADRS are represented by a series of radial lines extending from the origin of the plot. See also **Converting to ADRS Spectra** in Chapter 8.

4.4.3.2 Site-Specific Hazard Analysis Studies

Site-specific hazard analysis studies should be performed for buildings situated on site soil profile type S_F, and are recommended for buildings on site soil profile type S_E. Site-specific studies should also be performed for certain buildings situated near active sources and for buildings with special design requirements (e.g., hospitals, base-isolated buildings). In all cases, the assumptions and methods used in the site-specific studies should be consistent with the definition of the earthquake hazard level(s) of interest given in Section 4.2.

The site specific studies should develop estimates of short- and long-period response for each hazard level of interest. The value of C_A should be taken to be equal to 0.4 times the spectral response acceleration (units of g) at a period of 0.3 seconds. The value of C_V should be taken to be equal to the greater of either 1.0 times the spectral response acceleration (in units of g) at a period of 1.0 seconds or $1/T_{eff}$ times the spectral response acceleration (in units of g) at the effective period T_{eff} of the building. The effective period T_{eff} is based on the secant stiffness at the point of maximum response, as described in Chapter 7.

For each hazard level of interest, the values of C_A and C_V should not be taken as less than 80 percent, and need not be taken to be greater than 100 percent, of the values specified in Table 4-7 and Table 4-8, respectively.

4.4.3.3 Construction of an Elastic Response Spectrum

An elastic response spectrum, for each earthquake hazard level of interest at a site, is based on the site seismic coefficients C_A and C_V, defined in the previous sections. The seismic coefficient C_A represents the effective peak acceleration (EPA) of the ground. A factor of about 2.5 times C_A represents the average value of peak response of a 5 percent-damped short-period system in the acceleration domain. The seismic coefficient C_V represents 5 percent-damped response of a 1-second system and when divided by period defines acceleration response in the velocity domain. Figure 4-1 illustrates the construction of an elastic response spectrum.

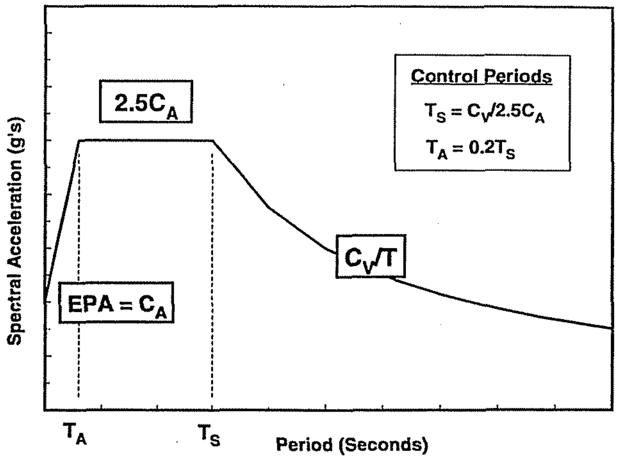


Figure 4-1. Construction of a 5 Percent-Damped Elastic Response Spectrum

4.5 Specification of Supplementary Criteria

4.5.1 Acceleration Time Histories

When required for analysis, not less than three pairs of horizontal time history components should be selected from earthquake ground motion records. A set of seven or more pairs of time history components is recommended and would be necessary for the design to be based on the average (rather than the maximum) value of the response quantity of interest.

Recorded earthquakes should be selected to have a magnitude, source characteristics, and distance from source to site that is the same as (or consistent with) the magnitude, source characteristics and source-to-site distance of the event that dominates the ground shaking hazard at the building site. Recorded earthquakes should also be selected to have site conditions that are the same as (or consistent with) the site conditions of the building. When three appropriate recorded ground

Table 4-9. Earthquake Records at Soil Sites Greater Than 10 Km from Sources

	Earthquake Source			Earthquake Recording	
No.	**Magnitude**	**Year**	**Earthquake**	**Station Name**	**Owner**
1	7.1	1949	Western Washington	Station 325	USGS[1]
2	6.5	1954	Eureka, California	Station 022	USGS
3	6.6	1971	San Fernando, California	Station 241	USGS
4	6.6	1971	San Fernando, California	Station 458	USGS
5	7.1	1989	Loma Prieta, California	Hollister, South & Pine	CDMG[2]
6	7.1	1989	Loma Prieta, California	Gilroy #2	CDMG
7	7.5	1992	Landers, California	Yermo	CDMG
8	7.5	1992	Landers, California	Joshua Tree	CDMG
9	6.7	1994	Northridge, California	Moorpark	CDMG
10	6.7	1994	Northridge, California	Century City LACC North	CDMG

1. USGS: United State Geological Survey
2. CDMG: California Division of Mines and Geology

motion time history pairs are not available, appropriate simulated ground motion time history pairs may be used to make up the total number required.

The intent of these requirements is that each pair of time history components have an appropriate duration, contain near-source pulses (for sites within 10 km of active faults) and include other time domain characteristics that represent the ground shaking expected at the building site.

Each pair of horizontal ground motion components should be scaled in the time domain such that the average value of the spectra of all scaled time history components matches the site response spectrum over the period range of interest. The period range of interest includes, but is not limited to, periods at or near the effective period of the building associated with the performance point determined by the nonlinear static analysis procedure (Chapter 8). If higher-mode effects are being considered, then the period range of interest should also include periods at or near each higher-mode period of interest.

***Commentary:** A ground motion expert should assist the structural engineer in the selection and scaling of appropriate time histories.*

4.5.1.2 Earthquake Ground Motion Records

Two sets of 10 earthquake records each have been identified as suitable candidates for time history analysis. One set contains records at sites at least 10 km from fault rupture and the other set contains records at sites near fault rupture (e.g., sites within about 5 km of fault rupture). Tables 4-9 and 4-10 list these earthquake records, respectively, and summarize key attributes.

All ground motion records listed in Tables 4-9 and 4-10 meet the following criteria:

- Free-field station (or ground floor of a small building)
- Stiff or medium soil site conditions
- Large-magnitude earthquake ($M \geq 6.5$)
- Peak ground acceleration of at least 0.2g (before scaling)

These records, after appropriate scaling, are suitable for time history analysis of buildings at all sites (except soft or very soft soil sites) for ground shaking of 0.2 or greater EPA.

Table 4-10. Earthquake Records at Soil Sites Near Sources

	Earthquake Source			*Earthquake Recording*	
No.	***Magnitude***	***Year***	***Earthquake***	***Station Name***	***Owner***
1	6.5	1979	Imperial Valley, California	El Centro Array Station 6	USGS[1]
2	6.5	1979	Imperial Valley, California	El Centro Array Station 7	USGS
3	7.1	1989	Loma Prieta, California	Corralitos	CDMG[2]
4	7.1	1989	Loma Prieta, California	Capitola	CDMG
5	6.9	1992	Cape Mendocino, California	Petrolia	CDMG
6	6.7	1994	Northridge, California	Newhall Fire Station	CDMG
7	6.7	1994	Northridge, California	Sylmar Hospital	CDMG
8	6.7	1994	Northridge, California	Sylmar Converter Station	LADWP[3]
9	6.7	1994	Northridge, California	Sylmar Converter Sta. East	LADWP
10	6.7	1994	Northridge, California	Rinaldi Treatment Plant	LADWP

1. USGS: United States Geological Survey
2. CDMG: California Division of Mines and Geology
3. LADWP: Los Angeles Department of Water and Power

Figure 4-2 is a plot of 40 percent-damped response spectra of the two horizontal components of each of the 10 earthquake time histories listed in Table 4-9 and the mean and mean + 1 standard deviation of these 20 spectra. A damping level of 40 percent is used to represent the equivalent viscous damping of a building that has yielded significantly, as described in Chapter 8. Each of the 10 pairs of horizontal components have been scaled such that their 5 percent-damped response spectra approximately match the response spectrum shown in Figure 4-1 for $C_A = 0.4$ and $C_V = 0.6$.

Figure 4-2 compares the response spectrum of each earthquake component and the mean (or mean + 1 sigma) response spectrum of the ensemble of all earthquake components. Significant differences typically exist between mean (or mean + 1 sigma) response and that of individual components, even though each component represents the same site and source conditions and has been scaled to match a common target spectrum. Figure 4-2 illustrates that predictions of response (and performance) using smooth design spectra can significantly underpredict or overpredict response (and performance) that the building actually experiences during the design earthquake.

4.5.2 Criteria for Duration of Ground Shaking

The duration of ground shaking should be considered when selecting time histories and when determining an appropriate level of effective damping for the structural system. Effective damping is used to determine the response level of demand spectra, as described in Chapter 8.

Structural systems that degrade with repeated cycles of high seismic demand will have reduced energy absorption (damping) capacity. The amount of degradation and the associated reduction in effective damping increases with the number of cycles at or near the peak demand level.

Two distinctly different earthquake scenarios should be considered when evaluating duration effects on potential structural degradation and reduction in damping capacity. The first

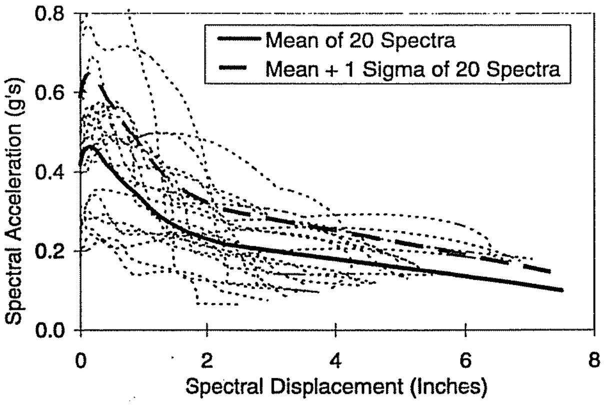


Figure 4-2. Response Spectra (40% Damping) of the Earthquake Records Listed in Table 4-9

earthquake scenario is important for sites near a seismic source (fault). In this case, a relatively short duration of very strong shaking would be expected because of the proximity of the site to fault rupture. The ground may shake for a considerable period of time (depending on the earthquake magnitude), but there would likely be only a few cycles of very strong pulses at the level of response described by the site spectrum (i.e., large spectral demand over a relatively short duration). For the purpose of determining an appropriate level of effective damping (Section 8.2.2), sites with a near-source factor, $N \geq 1.2$, may be assumed to have short-duration ground shaking.

The second earthquake scenario is important for sites far from fault rupture (far from the causative source). In this case, a much longer duration of ground shaking would be expected at the level of response described by the site spectrum (i.e., small/moderate spectral demand over a relatively long duration). Although ground shaking is not as strong in the second scenario, a longer duration of shaking increases the potential for degradation of the structural system. For the purpose of determining an appropriate level of effective damping (Section 8.2.2), sites located in seismic zone 3 should be assumed to have long-duration ground shaking unless a properly substantiated geotechnical study recommends otherwise.

For sites located in seismic zone 4 (with a near-source factor of $N < 1.2$), ground shaking duration (in terms of the number of cycles of demand at or near the peak demand level described by the site spectrum), will depend on the magnitude of the earthquake and the site soil profile (and possibly other factors). For the purpose of determining an appropriate level of effective damping (Section 8.2.2), sites located in seismic zone 4 (with a near-source factor of $N < 1.2$) should be assumed to have long-period ground shaking unless either the seismic source that governs ground shaking hazard at the site has a maximum moment magnitude of $M \leq 6.5$ and the site soil profile is rock or stiff soil (i.e., soil profile type S_B, S_C or S_D), or a properly substantiated geotechnical study recommends otherwise.

Commentary: *An important potential contributor to duration could be long-period resonance at soil sites due to basin effects. Long duration ground shaking should be assumed for soft soil sites unless a geotechnical study recommends otherwise.*

Chapter 5

Determination of Deficiencies

Audience Interest Spectrum

Owner	*Architect*	*Bldg. Official*	*Engineer*	*Analyst*
▨	▨	▨	■	▨

5.1 Introduction

A general sense of expected building performance should be developed before performing a detailed analysis. This chapter is intended to help develop such a sense. Emphasis is placed on qualitative assessment relative to typical concrete buildings.

The following sections discuss observed performance of existing concrete buildings and present the basic steps of a preliminary evaluation using simplified analysis. Preliminary evaluation involves acquisition of building data, review of the seismic hazard, identification of building attributes, limited analysis, and characterization of potential seismic deficiencies. With sufficient data from documents, tests, and site visits, a useful set of potential seismic deficiencies can be identified and used to inform engineering decisions or more detailed investigations. The *NEHRP Handbook for the Seismic Evaluation of Existing Buildings* (BSSC 1992), referred to as FEMA 178, is adopted as the primary basis for the preliminary evaluation procedures discussed in this Chapter.

5.2 Description: Typical Layouts and Details

Two main building types are discussed. Both feature a concrete frame carrying vertical gravity loads. In concrete frame buildings, frames also function as the lateral force resisting systems. In concrete frame-wall buildings, walls provide all or part of the lateral force resisting systems. In some instances, such as at stair and elevator cores, concrete walls may also carry some local vertical gravity loads.

5.2.1 Concrete Frame Systems

Concrete frames are monolithically cast systems of horizontal framing beams and vertical framing columns that provide lateral resistance through bending of horizontal and vertical framing elements (see Figure 5-1). Concrete frame buildings commonly include interior beam-column frames and perimeter pier-spandrel frames. Other interior framing systems, designed only for vertical loads, should also be considered in the evaluation of the building's ability to resist lateral forces. These interior gravity load supporting frame systems often include horizontal one or two-way flat slabs or joist framing in lieu of beams.

Code minimum required proportions and details for reinforced concrete buildings changed dramatically in the early 1970s. Where earlier codes focused on providing strengths to resist code-specified lateral forces, around 1970 codes began to focus on aspects of proportioning and detailing to achieve overall ductility or deformability as well as strength requirements. Nonductile concrete frames, although often

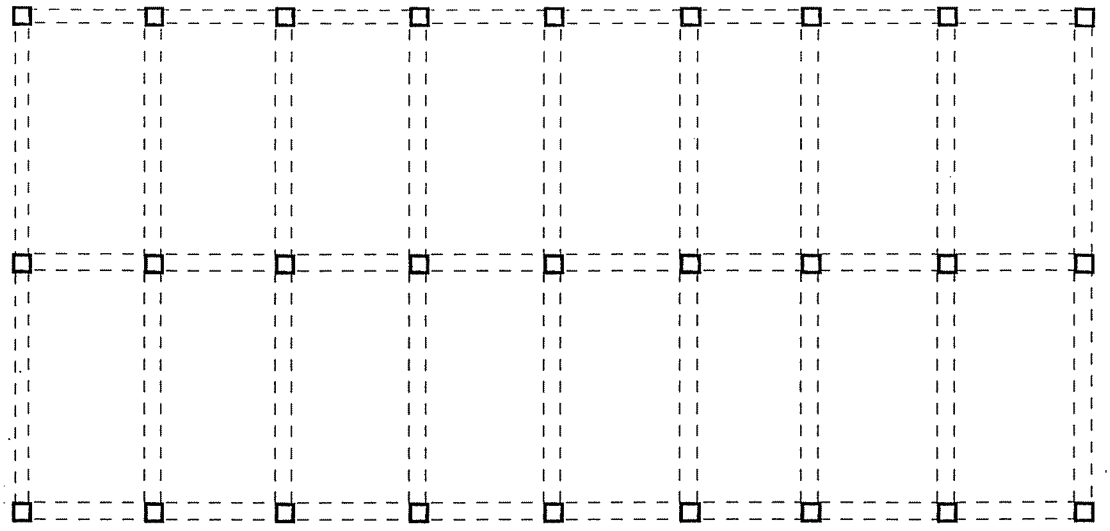


Figure 5-1. Typical Concrete Frame Layout

designed to resist lateral forces, did not incorporate the special detailing provisions now required for ductile concrete.

In pre-1970s concrete frame construction, it is common to find inadequate lap splices for main reinforcing bars and a lack of adequate transverse reinforcing ties (see Figure 5-2). Lap splices in columns generally occurred just above the floor level where stress levels are highest. In addition, the column lap splices were generally very short, often only 30 bar diameters or less in length, and were typically not confined with closely spaced column ties. Main beam reinforcing was generally designed for code force levels, not considering effects of post-yield behavior. Beam top bars were often terminated 6 to 8 feet from the column face. Beam bottom bars were typically discontinued at the face of the supporting column or provided with only a short lap centered on the column. The spacing of beam and column ties was typically wide by today's standards. Column ties often consisted of a single hoop with 90 degree hooks spaced at 12" to 18" on center. Beam ties, often sized only for gravity shear loads, were spaced closely near the column face but widely spaced or discontinued throughout the mid-span region of the beam.

The affect of these typical conditions and deficiencies on seismic performance is discussed below.

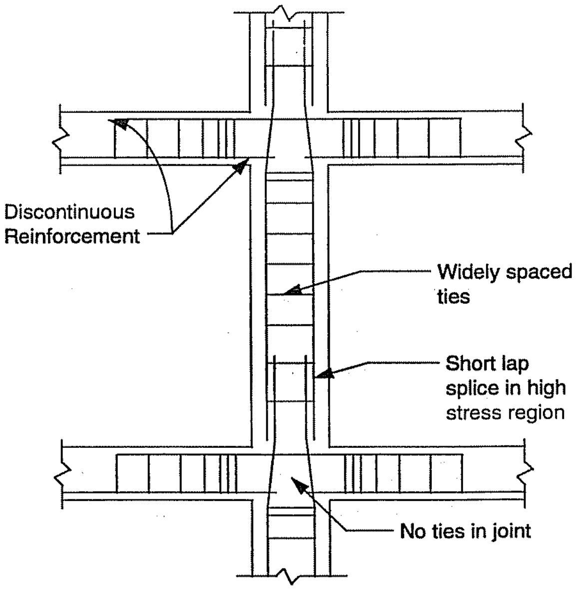


Figure 5-2. Typical Pre-1970s Frame Elevation

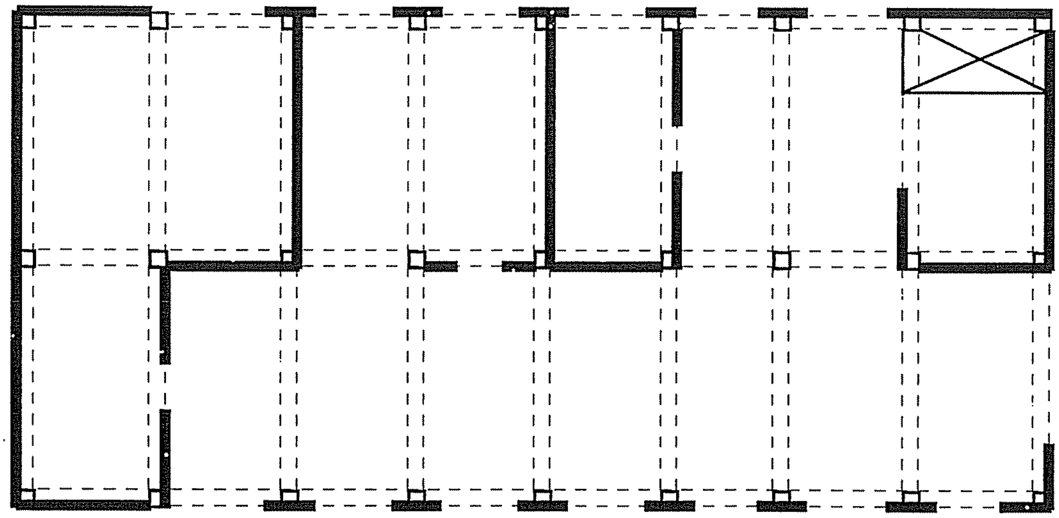


Figure 5-3. Typical Concrete Frame-Wall Layout

5.2.2 Concrete Frame-Wall Systems

Concrete frame-wall buildings typically possess a complete gravity frame system, essentially independent of the concrete walls, to support vertical loads (see Figure 5-3). However, in some instances concrete walls may carry some local vertical gravity loads.

Concrete frame-wall buildings can include, for example, exterior perforated walls with short piers and deep spandrels, interior concrete walls near stair and elevator cores, or perimeter frames with concrete infill walls. The buildings in this category generally have monolithically cast-in-place reinforced concrete horizontal floor and roof systems. Various concrete floor and roof framing systems used with this building type include flat plate, pan joist or beam, one-way slab, and two-way slab or waffle slab systems. Concrete frame-wall buildings were popular for institutional uses, such as government offices, hospital wards, schools, university buildings, court buildings, and prisons.

The layout of wall locations was, to a large part, dictated by functional considerations. In many older buildings it was the only consideration. Floor plans that induce torsion, lack significant torsional rigidity or have eccentric or skewed walls often resulted. Perimeter walls typically contain numerous window and door openings. In multistory buildings, the openings were frequently arranged in a regular grid. This pattern of openings gives rise to the pier and spandrel system composed of relatively deep spandrel beams and relatively short and wide wall piers. Floor framing systems were often not designed to function as diaphragms, with collectors, drags and struts, capable of distributing floor inertial loads to the isolated interior and/or perimeter walls. Also, adequate provisions to transfer the lateral load out of the walls into the foundation system are often lacking.

In existing, pre-1970s construction, it is common to find walls without confined boundary elements (see Figure 5-4). Vertical reinforcement lap splices were usually designed for compression loads only, in which case they may be inadequate for the flexural tension that may develop under realistic earthquake loads. Horizontal reinforcement also may be inadequately anchored.

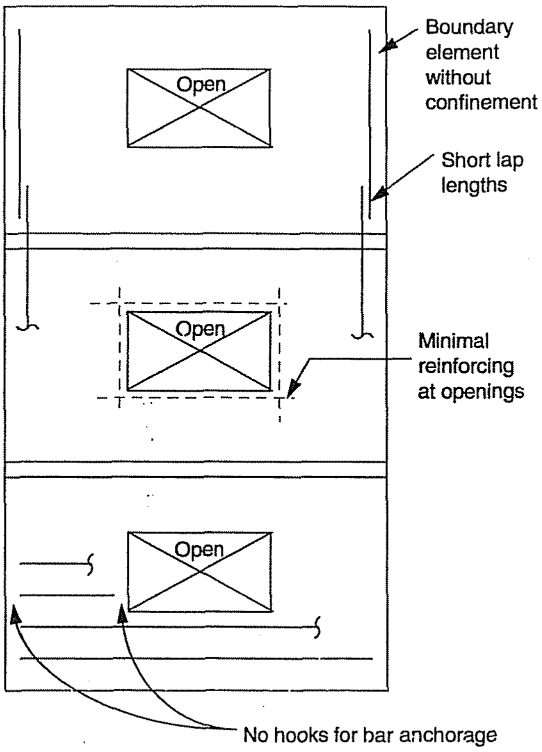


Figure 5-4. Typical Pre-1970s Wall Configuration

Wall thickness and transverse reinforcement were often sized to resist code-specified lateral forces rather than the shear corresponding to development of the wall's flexural capacity, so almost all walls in these older buildings are shear critical. Often, adequate attention was not paid to construction joints, resulting in there being weakened planes in the vertical wall system.

Walls with openings have been designed and detailed in a variety of ways, the choice depending on the expected behavior of the wall. Analysis methods used in design were commonly based on the assumption that the openings did not interrupt the monolithic form of the wall; interrupted reinforcement is placed at the boundary of the opening and diagonal steel may be placed across the corners to control cracking.

Where openings are more substantial, current U.S. practice is to calculate stresses at the boundaries of the wall segments. If the stress exceeds $0.2f'_c$, confined boundary steel is placed. This boundary reinforcing steel is often not present in older construction.

Where substantial wall openings are aligned vertically, the connecting segments may be designed as coupling beams. Ductile coupling beams in buildings designed to meet current U.S. codes are detailed with continuous top and bottom (and possibly diagonal) reinforcement that is well anchored into the wall. As required by some current codes, deeper coupling beams may be designed to have diagonal reinforcement. Coupling beams in older construction often lack many of these detailing characteristics.

Beam transverse reinforcement with closed hoops having 135 degree hooks, provides shear strength, confines the concrete, and restrains longitudinal bar buckling. Shear reinforcement is now proportioned to resist the shear corresponding to development of beam flexural strengths.

In pre-1930s construction, it is common to find concrete infill walls. The concrete infill was typically on the order of 6 to 8 inches thick and generally contained light reinforcement, such as 3/8-inch diameter bars at 24 inches on center, in a single curtain near the center of the wall. In many cases, the reinforcing of the infill did not continue into the surrounding gravity frame, creating a weakened plane around the infill boundaries. Furthermore, the concrete used in the infill was generally of lower quality than that used in the surrounding gravity frames. The concrete frames for these buildings were designed exclusively as gravity-load-supporting frames. As a result, these frames do not have the lateral stiffness and strength to resist code-level design lateral forces. Furthermore, they generally lack the reinforcing—continuous beam bottom bars through joints, continuous top beam bars at midspans, and shear reinforcement or ties throughout the full beam span—necessary to resist even moderate lateral forces.

The affects of these typical conditions on seismic performance is discussed below.

5.3 Seismic Performance

5.3.1 General

The seismic performance of a structure is dependent upon the performance characteristics of its critical components. The critical components are those that are necessary for vertical stability and those that comprise the seismic load path. In all buildings, seismic inertial forces originate in the components of the structure and are transferred through connections to horizontal diaphragm systems. The horizontal diaphragm systems distribute these inertial forces to vertical lateral force resisting elements which in turn transfer these forces to the foundations. When connections or elements along this seismic load path are subjected to forces and/or deformations that produce unacceptable damage states, deficiencies exist. When the elements of the structure that provide vertical stability can no longer maintain that capability, deficiencies exist.

Through observation of building performance in past earthquakes, a number of general building characteristics have been identified that have been responsible for localized component deficiencies. These characteristics include a discontinuous seismic load path, lack of redundancy in the vertical shear resisting system, vertical irregularities (abrupt changes in stiffness, strength, or mass), plan configuration irregularities, and the presence of adjacent structures which may potentially interact under seismic excitation. Any of these characteristics could occur in concrete frame or concrete frame-wall buildings. Detailed analytical procedures can be used to establish to what extent these building characteristics cause unacceptable building performance. However, recognizing that these factors exist in a building is a key consideration in developing an understanding of the building's expected seismic performance.

Component and connection seismic performance can be thought of in terms of response characteristics that are either force controlled or deformation controlled. Force controlled behavior is characterized by essentially elastic behavior that abruptly terminates in material rupture and subsequent rapid deterioration or disintegration. Once a critical force is reached in the element/connection, it experiences a rapid drop in stiffness and strength. This is frequently described as brittle behavior. Deformation controlled behavior is characterized by essentially elastic performance until a critical force threshold is attained beyond which deformation continues with little or no increase, or decrease, in capacity. At this plateau, the component can continue to deform until material strain limits are exceeded, at which point rupture, crushing, fracture or slippage occurs. At this limiting deformation, a sudden loss of stiffness and/or strength may also occur. This is frequently described as a ductile behavior pattern. Unacceptable seismic performance in concrete frame and frame-wall buildings, especially those designed and constructed before the early 1970s, is most often attributable to force controlled or brittle failure mechanisms. General discussions of the seismic performance of concrete systems and individual members found in frame and frame-wall buildings are available in several recent publications such as Paulay and Priestly (1992) and Ferguson et al. (1988).

5.3.2 Concrete Frame Systems

5.3.2.1 General

Concrete frame construction has several potential failure modes that directly threaten the structure's ability to sustain vertical loads and maintain stable lateral behavior. The largest concern is a brittle column failure mode caused by shear failure or compression crushing (due to combined axial, flexural, and P-Δ effects) of the concrete. Systems that exhibit some (limited) yielding modes can also eventually form dangerous collapse mechanisms as a result of stiffness or strength degradation at sections without ductile detailing. Punching shear failure in two-way slab systems, for example, can instigate local collapse of the floor. Also, localized concentrations of drift due to soft or weak story configurations are of serious concern. The challenge for the engineer is to identify all possible ways a frame building can fail, determine the sequence of failure, and then find ways to preclude catastrophic modes of response to seismic ground motion.

5.3.2.2 Typical Configuration Deficiencies

Configurational deficiencies commonly found in concrete frame structural systems, such as an incomplete load path, vertical and/or horizontal irregularities, and inappropriate column/beam relative strengths can lead to failure of individual members and connections.

5.3.2.2.1 Incomplete Load Path. A complete and continuous seismic load path is essential for the proper seismic behavior of a structure. Missing links in the load path must be identified. Load path evaluation should begin by establishing the source of all lateral loads generated throughout the building, and then tracking how those forces travel through the structural systems; from the diaphragms to the vertical lateral force resisting elements, through various joints and connections, to the foundations.

5.3.2.2.2 Vertical Irregularities. Vertical irregularities typically occur in a story which is significantly weaker, more flexible or heavier (due to a greater mass) than stories above or below. These irregularities are normally due to significant changes in building configuration such as setbacks, discontinuous vertical elements, or changes in story heights. However, they sometimes arise due to more subtle and less easily observed changes to column dimensions, size and number of main reinforcing steel, or column tie spacing. Vertical irregularities are difficult to appreciate by a visual inspection or simplified estimation. FEMA 178 (BSSC 1992) provides quick checking procedures for concrete frame story capacities and drifts that can provide preliminary assessment for vertical irregularities.

5.3.2.2.3 Horizontal Irregularities. Horizontal irregularities of the concrete frame buildings are typically due to odd plan shapes, re-entrant corners, or diaphragm openings and discontinuities. These deficiencies often cause significant differences between the building's centers of mass and rigidity (in one or more stories), resulting in the torsional response of the building to seismic ground motions. Such torsional irregularities lead to concentrated demands on diaphragms and excessive deflections at building ends. In many cases, conditions at adjacent stories can affect the stiffness properties of the irregular story.

5.3.2.2.4 Weak Column/Strong Beam. Optimum seismic performance is gained when frame members have shear strengths greater than bending strengths and when bending strengths of the columns are greater than the beams. These features provide a controlled failure mode and, in multistory frames, increase the total energy absorption capacity of the system. In older concrete frame buildings, where the beams are often stronger than the columns, column hinging can lead to a story mechanism, creating large P-delta effects and inelastic rotations in the columns. Column hinging is undesirable since this may lead to loss of the column's gravity load carrying ability after only a very few cycles. Furthermore, although isolated column hinging may be tolerable in some circumstances, hinging of most or all of the columns on a single level will result in the loss of lateral stability. FEMA 178 provides local joint analyses for frames to evaluate the potential for these effects.

5.3.2.3 Detailing Concerns

In general, most concrete frame buildings built before 1973 will contain an array of nonductile detailing of the reinforcing steel. As a result, these nonductile frame buildings exhibit a wide range of generally poor seismic performance, especially once the elastic deformation limits of the concrete members are exceeded. Reinforcement detailing conditions which should be investigated for deficiencies in the preliminary evaluation include the following:

- Quantity, size, and spacing of column transverse reinforcement ties
- Column ties in exterior column/beam joints
- Location and length of column and beam main bar splices in critical regions
- Continuity of top and bottom beam bars through column/beam joints
- Use of bent longitudinal beam bars for shear reinforcement
- Anchorage of beam stirrups and column ties into the concrete core with 135 degree hooks
- Continuous bars at flat slab (or plate) joints with columns acting as a frame

5.3.2.3.1 Beams. Transverse reinforcement in pre-1973 concrete buildings was designed to resist code specified lateral forces rather than the shear corresponding to the development of the beam's flexural capacity. Also, concrete was assumed to contribute to the shear strength in beam hinge regions. The resulting beam may develop shear failure before or shortly after development of flexural yielding and, as a result, may undergo progressive deterioration and loss of deformation capacity at the face of the column.

5.3.2.3.2 Columns. In pre-1973 buildings, the longitudinal column reinforcement bars were commonly designed to resist moments generated by code-specified lateral forces rather than the moments associated with the capacities of the connecting beams. As opposed to current ductile detailing requirements, which allow columns to form plastic hinges and sustain frame displacements beyond their elastic limit while maintaining vertical load carrying capability, these older columns are often weaker than the beams, leading to early column hinging and an undesirable column side sway mechanism. However, it is worth noting that the column design formulas and allowable stresses in these older codes were often more conservative than current provisions, resulting in columns with substantial elastic capacity in some cases.

5.3.2.3.3 Beam/Column Joints. The lateral stability of the frame is dependent upon the beam-column joint remaining stable under large frame deformations. Adequate strength and toughness must be provided in the joint to sustain repeated cyclic stress reversals without the loss of joint integrity. In pre-1973 buildings, it is common for beams to frame eccentrically to the column. Also, bottom longitudinal reinforcement may terminate a short distance into the joint creating potential for bar pullout under moment reversals. In exterior joints, hooked bars were normally bent away from the joint rather than into the joint. Column bars were often poorly distributed around the joint perimeter and spliced just above the beam-column joint. In addition, transverse reinforcement in the joint was often minimal or none at all. The lack of joint transverse reinforcement may reduce the strength and ductility of the joint or the adjacent framing members. Also, the presence of column lap splices and discontinuous beam bars in adjacent framing members can limit the input energy from those members so that premature joint failure is avoided.

5.3.2.3.4 Flat Slab/Column Frame Systems. Older slab-column frames require special attention in moment frame buildings, which may experience large drifts during earthquakes. Pre-1973 slab-column frames commonly do not have continuous slab reinforcement, and in particular, continuous bottom slab reinforcement, through the column cage. Continuous bottom reinforcement acts as flexural reinforcement when the moments reverse under lateral load. It also acts to suspend the slab through catenary action if a punching shear failure of the slab occurs.

5.3.3 Concrete Frame-Wall System

5.3.3.1 General

Historically, there have been relatively few cases of the collapse or partial collapse of frame-wall buildings in past earthquakes. This generally good seismic performance is largely due to the presence of the relatively stiff walls that prevent the frames from experiencing very large lateral

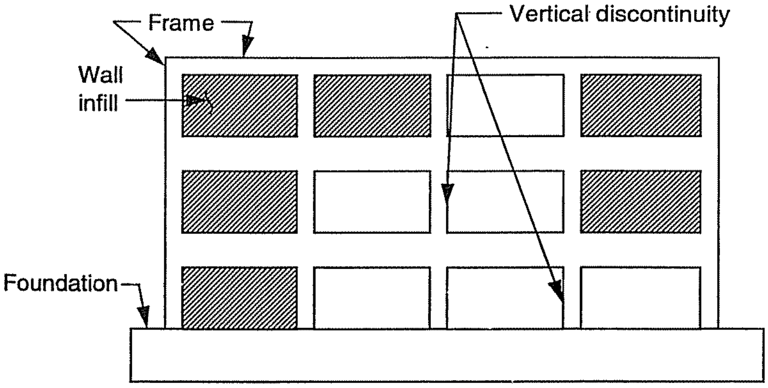


Figure 5-5. Typical Vertical Discontinuity

displacements and deformations. Also, the presence of an essentially complete vertical load carrying space frame eliminates the need for the walls to retain their vertical load carrying resistance throughout the seismic response. Where collapses have occurred, they have been traced to irregular wall layouts, insufficient connections along the seismic load path, poor concrete quality, inadequate reinforcement anchorage, or a grossly inadequate quantity or distribution of the walls.

The presence of these building attributes can give rise to a range of specific building component/connection failure mechanisms. These mechanisms include vertical discontinuities, weak stories, perforated walls, coupling beams, shear cracking, diagonal tension/compression, sliding shear, reinforcement anchorage and confinement and foundation anchorage and uplift. The challenge to the engineer is to identify the possible ways frame-wall buildings can degrade, to determine the sequence of failure, and to find ways to control the degradation to preclude catastrophic modes of response.

5.3.3.2 Typical Deficiencies

5.3.3.2.1 Vertical Discontinuity. The primary negative structural impact of a vertically irregular placement of wall panels, in which an upper level wall (particularly one at an interior column line) is not continued directly down to its foundation, is that the columns below the ends of the discontinued wall are subjected to large, concentrated overturning reactions (see Figure 5-5). Frequently, these columns have been designed to resist only gravity loads. Under extreme seismic loading, the columns can experience forces well in excess of their compressive capacity and suddenly lose their ability to sustain gravity loads. These columns may also experience net uplift tension forces for which insufficient reinforcement has been provided. These types of force controlled failures can be sudden and catastrophic. They are considered serious deficiencies when the seismic force levels exceed the available column axial load capacities. A secondary structural impact of this type of deficiency is that the diaphragm at the level of the discontinuity is required to transfer large shear forces from the discontinued wall and redistribute it to other available walls below. Often, the floor diaphragms are not designed for this redistribution of wall shears and can become very highly overloaded

5.3.3.2.2 Weak Stories. Irregular placement of interior wall panels can result in significant changes in strength and stiffness from floor to floor. Of particular concern is a reduction in the number of shear wall panels at the ground floor level. This abrupt strength and stiffness reduction results in a floor level whose vertical shear resisting elements are likely to reach their limit response states before the walls in the floors above have

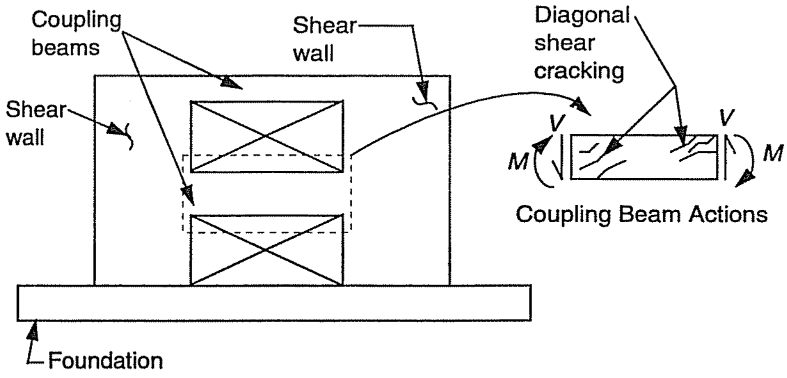


Figure 5-6. Typical Coupling Beam

reached the elastic limit of their response. As a result, the weak or soft story will undergo large lateral displacements in comparison to the floors above. This concentration of lateral displacement can lead to severe demands on the columns that provide the structure's vertical stability. A weak story is considered a deficiency if it can lead to a loss of vertical stability at the seismic hazard level associated with the target performance objectives.

5.3.3.2.3 Perforated Walls and Pier/Spandrel Conditions. Perimeter walls frequently feature a large number of openings for windows and doors. These openings typically occur on a repeated module that provide an appealing symmetry. The perforated wall that results; acts to resist seismic forces as a wall/frame wherein the piers act as column elements and the spandrels act as beams. In older frame-wall buildings, the piers are often only lightly reinforced, for both shear and flexure. While concentrated amounts of vertical reinforcing may be found at locations where a concrete column occurs, most often the column occurs at the middle of the wall piers and does not appreciably contribute to the lateral capacities of the wall pier. As a result, the piers are under-reinforced for both the shear forces and the concentrated flexural tensile and compressive forces that result in the pier due to seismic actions. The limited amount of tensile reinforcement can result in relatively low flexural capacity of these elements. The spandrel elements will perform in a manner similar to frame beams. As noted for coupling beams, these elements can be a deficiency if they are not reinforced adequately for the forces they develop in the piers. The behavior of these elements will typically be deformation controlled but can be a serious deficiency if yielding occurs at very low lateral force levels. This is true for all performance levels.

5.3.3.2.4 Coupling Beams. Beam elements that are attached to adjacent shear walls tend to couple the seismic resistance of the walls as if they were columns joined in a moment frame. If the depth of the beam is sufficient, its stiffness may be great enough to restrain the rotation of the walls significantly. However, the spandrel beam must be strong enough to sustain the corresponding shear forces and end moments associated with this frame-like behavior and, in many older buildings, there often is not sufficient reinforcement provided to accomplish this. Differences in taller wall pier stiffnesses due to axial tension and compression forces can cause tension to occur in coupling beams as the pier, in compression, resists more load than the pier in tension. As a consequence, shear wall coupling beams can exhibit rapid diagonal tension failures (see Figure 5-6) similar to that described in Section 5.3.3.2.5 below.

The failure of these elements can result in spalling of the concrete at or near the joint to the

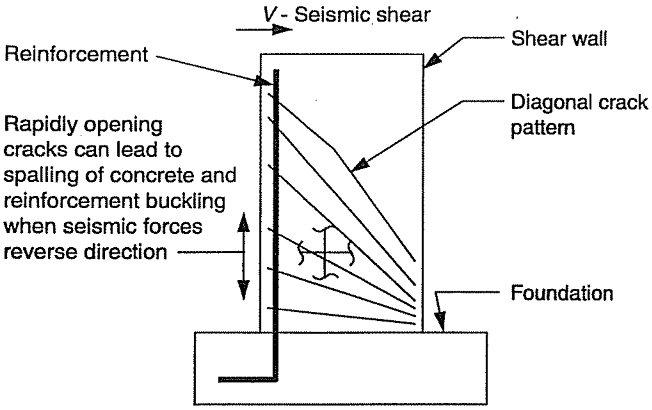


Figure 5-7. Typical Shear Cracking Pattern

wall sections. Since coupling beams are frequently located either on the perimeter of the building or over areas of the structure that serve as exit corridors, the degradation of these elements can create a falling hazard, which poses a minor to moderate life safety risk to passersby or persons exiting from the building.

Additionally, failure of these elements decouple the adjacent shear walls, thereby changing their effective stiffness. Often, this can represent a beneficial effect. Thus, coupling beam deficiencies may be considered acceptable seismic performance for both the Life Safety and Structural Stability performance objectives provided that any resulting permanent offset of the structure is within permissible bounds, stability under gravity loads is maintained, and falling debris does not jeopardize life safety.

5.3.3.2.5 Shear Cracking and Diagonal Tension/Compression. Walls that resist earthquake forces can exhibit diagonal patterns of cracking that are attributable to the development of stresses in the concrete that exceed the concrete's principal tensile stress. The onset of shear cracking is generally not considered a serious deficiency unless there is an insufficient amount of reinforcing steel in the wall to maintain deformation controlled behavior.

Minor shear cracking is considered an acceptable damage state for even the Immediate Occupancy performance objective if ductile, deformation controlled, behavior is maintained. More extensive shear cracking is considered acceptable for both the Life Safety and Structural Stability performance objectives. In all instances, shear cracking is an acceptable damage state for concrete shear walls provided that adequate reinforcement is provided to control the width of the cracks. When the reinforcement is insufficient, the widths of cracks open rapidly, which can lead to buckling of the reinforcement and degradation of the wall's ability to sustain vertical loads (see Figure 5-7). A shear wall that is substantially under-reinforced can thus exhibit brittle, force controlled behavior and represent a serious deficiency, once the force level associated with initial cracking is reached, for higher levels of desired performance.

Walls that are more heavily reinforced and are subjected to extreme seismic demands may also exhibit force controlled behavior related to shear cracking. This can occur when compressive stresses are developed that exceed the concrete's compressive strength. When diagonal tension cracks form, it is generally held that a series of inclined tension and compression struts act to resist the shearing forces in the wall panel. When the compression strut force exceeds the compressive capacity of the concrete, crushing will occur (see Figure 5-8). Such a failure mode, when subjected to repeated reversing cycles of earthquake loading, can lead to the generally gradual loss of the

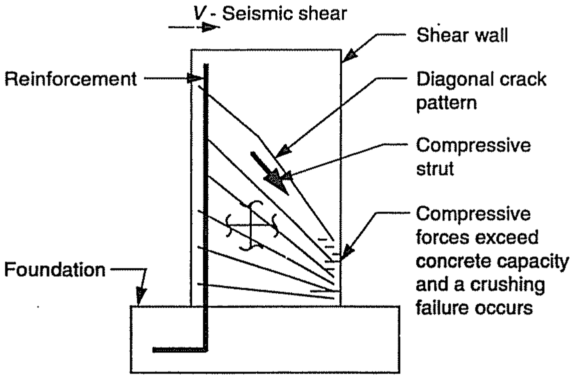


Figure 5-8. Typical Combined Shear Cracking / Compression Failure

wall's ability to sustain large gravity loads. A deficiency of this type can occur when there is a grossly inadequate amount of wall provided, weak or disintegrated concrete is present, and the wall is subjected to large gravity loads. Similar to coupling beam failures, shear cracking/diagonal tension failures can be considered acceptable if permanent offsets are within permissible bounds and stability under gravity loads is maintained.

5.3.3.2.6 Sliding Shear. Sliding shear failure can occur at a weak plane, such as a construction joint, or along flexural cracks that have opened up or propagated, after several cycles of loading, to form a shear failure plane. In addition, formation of a shear failure plane can occur after concrete crushing has occurred at both ends of a wall.

Once a crack has been established across the section, shear forces are resisted only by the doweling action of the reinforcing steel crossing the crack or joint. Continued cycles of seismic load reversal can cause kinking or fracturing of this reinforcement. Once a significant number of reinforcing bars have been bent or broken, resistance to seismic shearing forces can only be mobilized through frictional resistance along the shear plane. This form of resistance is force controlled and can lead to significant permanent offsets which, if great enough, can result in vertical instability. Sliding Shear failures can be considered acceptable seismic performance for both the Life Safety and Structural Stability performance objectives provided that any resulting permanent offset of the structure is within permissible bounds and stability under gravity loads is maintained.

5.3.3.2.7 Reinforcement Anchorage and Confinement. The lengths of reinforcing bar lap splicing used in older concrete buildings is often not sufficient to develop the yield capacity of bars. In shear walls, where flexural behavior is important, insufficient lap lengths of the boundary reinforcing can lead to bar slip before the development of yield stresses in the reinforcing. Inadequate confinement of the concrete at wall boundaries can also limit the effectiveness of the lap splices used for vertical boundary reinforcing and of the anchorage of horizontal wall reinforcing. Premature slippage of reinforcing is a significant modeling concern that should be accounted for by the use of a reduced reinforcing capacity. Once reinforcing slippage begins to occur, a brittle, force controlled mechanism can result. In the case of vertical boundary reinforcement, slippage can be accompanied by a loss of flexural resistance which leads to a progressive deterioration of the wall boundaries. The slippage of horizontal wall reinforcement can aggravate development of shear cracking in the wall. The damages resulting from this type of deficiency may be considered to be acceptable seismic performance for either the Life Safety or Structural Stability performance objectives provided that any resulting permanent offset of the structure is within permissible limits and stability under gravity loads is maintained.

5.3.3.2.8 Foundation Anchorage and Uplift. Analogous to the lap splice condition described in Section 5.3.3.2.7 above, the length of embedment used in older buildings for anchorage of the vertical wall reinforcing into the foundations is often not sufficient to develop the yield capacity of the bars. The resulting slippage of the reinforcement can lead to the same kind of performance as described for inadequate lap splices.

In older concrete buildings, the footings that support concrete walls were sized for gravity loads. The large concentration of force at the ends of shear walls resisting seismic loads can exceed both the tributary dead load at the foundation or the compressive bearing strength of the soils. Such conditions can give rise to foundation rocking. The phenomena of foundation rocking tends to reduce the effective stiffness of the wall. This can have a variety of effects, either beneficial or detrimental, on the building's seismic performance. Rocking could lengthen the building period sufficiently to actually reduce the force demand, thus actually protecting force critical elements in some cases. Rocking could, however, also lead to significantly larger displacements than would be predicted by fixed base modeling and thus contribute to potential vertical load carrying instabilities due to the $P - \Delta$ effect. A detailed discussion of foundation effects on seismic performance is provided in Chapter 10.

Matrix for Use of Data Collection Tables

Level of Seismic Evaluation	Original Drawings Available	Original Drawings Not Available
Preliminary Evaluation	Table 5-1	Table 5-2
Detailed Evaluation	Table 5-3	Table 5-4

The potential for foundation flexibility or rocking should be assessed during the preliminary evaluation of deficiencies.

5.4 Data Collection

5.4.1 Introduction

The seismic evaluation of existing concrete buildings depends on data collection as a factual basis. The data collection process includes acquisition of available documents, field observations, field investigations, materials testing, and documentation. While the extent of the data acquisition process will vary from building to building and will depend on the availability of drawings and the level of evaluation being performed, i.e. preliminary evaluation, detailed evaluation and analysis, or preparation of final retrofit construction documents, accurate building information is necessary, in any event, in the following areas:

- Building geometry, configuration, and mass (including structural, architectural and mechanical systems)
- Elements of the seismic load path, including frames, walls, diaphragms, foundations, and connections
- Configuration and layout of structural members, including size of members, size of reinforcing, tie spacing, splice locations, and concrete cover
- Properties of the materials used in the structural system, such as concrete and steel reinforcing
- Anchorage of nonstructural elements

Tables 5-1 through 5-4 summarize the data collection process as it relates to availability of drawings and level of evaluation. The tables provide guidance for the applicability of the specific data collection items for which the following sections provide more detailed descriptions.

Commentary: The amount of information gathered should be sufficient to perform the various levels of evaluation and analysis discussed in this document, yet it may be imprudent to collect more information than necessary for a specific stage in the evaluation process. For example, if it is not clear whether retrofit will be required, the data collection process should be geared towards gathering just enough information to make that determination, whereas once the determination to retrofit has been made, the data collection process will usually be much more extensive.

It may not be necessary to identify the configuration and layout of all structural members in the building. However, as required by the level of evaluation, information should be acquired for all structural members that either are part of the seismic load path (e.g. walls and diaphragms) or are necessary for vertical stability of the structure and may be susceptible to failure because of building accelerations or displacements during earthquakes.

5.4.2 Acquisition of Available Documents

The single most important step in the data collection process is the acquisition of documents describing the existing construction. The review of construction drawings will simplify field work and lead to a more complete understanding of the building, especially considering that no amount of field investigation and testing can substitute for the information available in the original drawings. Acquisition of the original architectural and structural construction drawings is often critical to an accurate and cost effective evaluation. Potential resources include the current building owner, building departments, and the original architects /engineers. In some cases, drawings may also be available from architects or engineers who have performed prior evaluations for the specific building. If drawings cannot be located, substantial effort will be required to document the existing construction, even for preliminary evaluation of deficiencies, and a detailed evaluation of the structure will require extensive field testing.

In addition to construction drawings, it may be helpful to acquire, to the extent possible, the following documents:

- Structural calculations
- Site seismicity/geotechnical reports
- Foundation reports
- Prior building assessments

5.4.3 Field Observations

It is of paramount importance that the evaluating engineer conduct field observations of the building. Though the extent will vary depending on the evaluation requirements, primary purposes and types of the field observations typical for most projects are as follows:

- Verification of the accuracy of the original drawings or determination of basic building information if no drawings are available
- Identification of major alterations not shown on the original construction documents. Major additions and alterations are not uncommon and may have significant impacts on the seismic performance of the building
- Identification of visible structural damage, such as concrete cracking or spalling. Structural damage or cyclic degradation caused by prior earthquakes, deterioration, or poor quality original construction may reduce the available structural capacity of the building
- Identification of potential nonstructural falling hazards, including ceilings, partitions, curtain walls, veneer, mechanical systems, fixtures, and other nonstructural building elements
- Documentation of existing conditions with photographs at key locations. These photographs often serve as a useful verification tool during the evaluation process

Table 5-1. Information Required For Preliminary Seismic Evaluation When Original Construction Drawings are Available

Item	*Required*		*Comment*
	Yes	*No*	
Structural calculations		X	Helpful but not essential
Site seismicity, geotechnical report		X	Helpful but updated report should be done
Foundation report		X	Helpful but not essential
Prior seismic assessment reports		X	Helpful but not essential
Condition survey of building	X		
Alteration and as built assessment	X		
Walk through dimensioning		X	Unless required by undocumented alterations
Nonstructural walk through	X		Identify falling hazards, weight.
Core testing		X	Unless concrete appears substandard
Rebound hammer testing		X	Unless concrete appears substandard
Aggregate testing		X	
Reinforcement testing		X	
Reinf. location verification		X	Unless insufficient info. on drawings
Nonstructural exploration		X	

Table 5-2. Information Required For Preliminary Seismic Evaluation When Original Construction Drawings are Not Available

Item	*Required*		*Comment*
	Yes	*No*	
Structural calculations		X	Could minimize scope of site work
Site seismicity, geotechnical report		X	Could minimize scope of site work
Foundation report		X	Could minimize scope of site work
Prior seismic assessment reports		X	Could minimize scope of site work
Condition survey of building	X		
Alteration and as built assessment	X		
Walk through dimensioning	X		Sufficient to define primary elements
Nonstructural walk through	X		Identify falling hazards, weight
Core testing (limited)	X		Minimum 2 per floor, 8 per building
Rebound hammer testing		X	Could be helpful, especially if concrete appears substandard
Aggregate testing	X		Several cores
Reinforcement testing		X	
Reinforcement location verification		X	Could be helpful
Nonstructural exploration		X	

Table 5-3. Information Required For a Detailed Seismic Evaluation When Original Construction Drawings are Available

Item	Required		Comment
	Yes	No	
Structural calculations		X	Could be very helpful
Site seismicity, geotech rpt.		X	Helpful but not essential
Foundation report		X	Helpful but not essential
Prior seismic assessment reports		X	Helpful but not essential
Condition survey of building	X		
Alteration and as built assessment	X		
Walk through dimensioning		X	Spot checking is appropriate
Nonstructural walk through	X		Identify falling hazards, weight
Core testing	X		Minimum 2 per floor, 8 per building
Rebound hammer testing	X		Minimum 8 per floor, 16 per building
Aggregate testing	X		Each core
Reinforcement testing		X	Optional
Reinforcement location verification	X		Pachometer @ 10% of critical locations, visual @ 2 locations
Nonstructural exploration	X		Verify anchorage and bracing conditions for components sensitive to Building Performance

Table 5-4. Information Required For a Detailed Seismic Evaluation When Original Construction Drawings are Not Available

Item	Required		Comment
	Yes	No	
Structural calculations		X	Could be very helpful
Site seismicity, geotech rpt.		X	Helpful but not essential
Foundation report		X	Helpful but not essential
Prior seismic assessment reports		X	Helpful but not essential
Condition survey of building	X		
Alteration and as built assessment	X		
Walk through dimensioning	X		Must be done very thoroughly, particularly if structure will be retrofitted
Nonstructural walk through	X		Identify falling hazards, weight
Core testing	X		Minimum 2 per floor, 8 per building
Rebound hammer testing	X		Minimum 8 per floor, 16 per building
Aggregate testing	X		Each core
Reinforcement testing	X		2 per type
Reinforcement location verification	X		Pachometer for all critical elements, visual on 25%
Nonstructural exploration	X		Verify anchorage and bracing conditions for components sensitive to Building Performance

5.4.4 Materials Testing

If original construction documents are available and no deterioration has been observed during the field observations, a preliminary evaluation may be completed by using materials strength data from the documents. However, the techniques for detailed evaluations and analyses require accurate material properties for evaluating the strength and stiffness of the building systems, elements, and connections. If original construction data regarding the strengths of the materials are not available, a materials testing program should be established to determine the in situ strength of the concrete and reinforcing steel that serve as the primary structural materials. Even when the specified material strength data are available, a testing program should be undertaken to establish and/or verify the actual materials strength data for detailed analytical studies.

A typical detailed testing program should include pachometer (resistance meter) testing to verify the existence and spacing of reinforcing bars, concrete core testing, rebound hammer testing of concrete, unit weight testing of concrete aggregate, and testing of steel reinforcement. Also, the concrete cover should be removed in a limited number of locations to verify the accuracy of the pachometer readings and the bar sizes. The evaluating engineer should determine the types and extent of testing based on the requirements of each specific project.

5.4.4.1 Core Testing

Core testing provides a reliable method for the determination of concrete compressive strength to be used in the evaluation and analysis process. Core samples should be taken from critical structural elements such as shear walls, frames, and diaphragms. Because of the destructive nature of the coring process, however, testing can generally be limited to two concrete core samples for each floor level, with a minimum of eight samples per building. However, if testing results vary widely, additional core samples may be required. Prior to coring, the existing steel should be located (e.g. by pachometer or exploration) in order to avoid damaging the existing structural system.

Commentary: *The in situ concrete compressive strength is typically higher than specified on the construction documents. The testing, therefore, will typically benefit the evaluation process by demonstrating the actual capacity of the existing building materials. In some situations, however, testing may identify low concrete strengths attributable to poor quality of the original construction or to deterioration due to adverse environmental conditions.*

5.4.4.2 Rebound Hammer Testing

Since the number of core samples that can be taken is limited because of the destructive nature of sampling, rebound hammer testing should be performed to supplement the concrete strength data. A minimum of eight rebound hammer tests should be taken at each floor level, with a minimum of 16 test locations per building. Calibration of the rebound hammer testing should occur adjacent to core test locations.

5.4.4.3 Aggregate Testing

The unit weight of the concrete should be tested to determine whether lightweight aggregates were used in the concrete construction. This information can generally be obtained from the core samples used for compression testing.

5.4.4.4 Reinforcement Testing

Reinforcing samples should be taken from the building to determine the strength and deformability of the steel. Minimum test data should include stress and strain information at yield and at rupture. When required by the analysis procedures, a minimum of two samples should be tested from each type of reinforcement (e.g., beam longitudinal steel, beam ties, column longitudinal steel, column ties, slab reinforcing, wall reinforcing). In addition, if potential retrofit schemes include the welding of existing reinforcing, chemical analysis of the steel samples should be performed to determine equivalent carbon content.

5.4.5 Detailed Field Investigation

If adequate construction documents are available, detailed field investigation is not necessary for a preliminary evaluation. Well-developed construction drawings, as confirmed by field observations, are generally adequate to identify the seismic load path, identify reinforcing configurations, calculate the weight/mass of building systems, and identify the anchorage of nonstructural elements.

If construction documents for the building are not available, detailed field investigations will be necessary to collect the building data described above. Field investigations for the collection of building data can be a daunting task, especially in reinforced concrete structures. The field investigation program must be tailored to each specific project, striking a balance between obtaining the necessary data and keeping the cost of the investigation program reasonable. In many cases the structural elements are covered with architectural finishes, and some removal and/or soft demolition may be required in order to obtain the necessary information. Furthermore, it will not be possible to perform a detailed investigation on each element of the structural system. Sound engineering judgment will be required to conservatively extrapolate results from a representative sample of detailed inspections to the global building system. Detailed building survey data should be recorded in a set of drawings that depict the buildings primary, vertical and lateral force resisting system.

"Potential" deficiencies are based on drawing review and very limited analysis. They are usually qualitative.

To perform detailed analytical evaluations, it is necessary that a materials testing program, similar to the one described above, be used, whether drawings are available or not. To predict seismic performance using the analytical procedures in this document, it is necessary that concrete and reinforcing strengths be well defined. Determination of the onset of force controlled and deformation controlled component behavior cannot be made in a reliable manner without this information.

Several techniques are useful in determining the as-built reinforcement configuration in existing buildings. In general, nondestructive testing such as pachometer or X-ray testing can be used to determine the location and quantity of the reinforcement. Exploratory probes should be made in localized areas to remove concrete cover and expose the reinforcement and confirm the results of the nondestructive testing.

As information is collected, record drawings should be developed for the building to document the gathered information and identify areas where additional information/investigation is required. As a minimum, record drawings should include structural plan layouts of each floor level, details of reinforcing conditions, identification of materials strengths, and typical details used to brace nonstructural elements. The documentation should contain sufficient information to make it possible to evaluate the strength and deformability of the building systems.

5.5 Review of Seismic Hazard

Site characterization and seismic hazard derivation are discussed in Chapter 4. There, 5 percent damped elastic response spectra for various hazard levels are defined in terms of coefficients C_A and C_V. These coefficients may also be used with FEMA 178 base shear equations for preliminary analyses described in Sections 5.6 and 5.7.

The potential for ground failure should be reviewed as described in Section 4.3. When ground failure is deemed likely, the preliminary evaluation described in Sections 5.6 and 5.7 is not sufficient.

Commentary: *The simplified analysis methods used for preliminary evaluation are not considered adequate to account for ground failure or complex soil-structure effects. When ground failure is* *indicated, a geotechnical engineer should recommend mitigation measures or provide parameters for a detailed analysis.*

5.6 Identification of Potential Deficiencies

5.6.1 Introduction

Potential deficiencies are conditions that *might* lead to unacceptable performance at a local or global level. Identification of potential deficiencies requires only quick and limited analyses based on available building information. These conservative, simplified analyses are intended to find potential deficiencies that require detailed evaluation.

Commentary: *A list of potential deficiencies will generally serve as the basis for more detailed analyses (Chapters 8-11) used to check acceptability of the existing or retrofitted building. In some instances it may be possible to identify deficiencies of such a serious nature that the decision to retrofit or abandon the structure can be made without further study. This may be particularly true for Performance Objectives requiring Immediate Occupancy.*

The engineer should understand the limited precision of simplified analyses and should define potential deficiencies accordingly. Deficiencies identified in this manner may not represent conditions hazardous enough to jeopardize structural stability or life safety. In most cases, more detailed investigations are required to check acceptability at the desired performance levels, hence the label "potential."

5.6.2 Definition of Seismic Load Path

To identify potential seismic deficiencies, the first step is to define the components (including connections) that constitute the seismic load path. A complete and adequate seismic load path is fundamental to acceptable seismic performance. All existing buildings have a lateral load path of some kind. Non-engineered structures and older buildings not designed for lateral load resistance may have a lateral load path that relies partly on friction or on extremely weak or brittle components. When components on the seismic load path are subjected to forces or deformations that produce unacceptable damage, deficiencies exist.

To define the seismic load path, first locate the principal mass elements. In concrete buildings, floor and roof diaphragms usually account for most of the mass, and the lateral load resisting system itself accounts for much of the rest. Next, identify shear-resisting elements in each story that link the masses to each other, typically walls and frame columns. Determine the method(s) of connecting vertical shear resisting elements to horizontal diaphragms.

5.6.3 Selection of Evaluation Statements

The recommended procedure for identifying potential seismic deficiencies is based on the True/False "evaluation statement" methodology of FEMA 178 (BSSC 1992). After a seismic load path for the structure has been defined, the most appropriate set of FEMA 178 evaluation statements is selected.

Commentary: For most concrete buildings, only three sets of evaluation statements will be needed: General Evaluation Statements for foundations and nonstructural elements, Building Type 8 statements for Concrete Moment Frames, and Building Type 9 statements for Concrete Shear Walls.

For purposes of preliminary evaluation, FEMA 178 evaluation statements are most useful as indicators of qualitative deficiencies. Their systematic use by experienced engineers can be expected to find deficiencies associated with irregular building configurations, incomplete or poorly-conceived structural systems, and some inadequate details. They should not be expected to identify specific local deficiencies without further analysis. Refer to Section 5.6.4.3 commentary.

5.6.4 Simplified Analysis

5.6.4.1 General

A simplified analysis provides *relative* quantitative data for the preliminary evaluation discussed further in Section 5.7. Simplified analysis results can also yield insights useful for modeling elements and components for detailed analysis (Chapter 9).

Commentary: *Because simplified analyses with linear elastic properties do not account for inelastic deformations, degrading strength, or force redistributions, component forces are best understood in relative terms. For example, where inelasticity is anticipated, simplified analysis results can indicate which components have the highest demands or are likely to hinge first, but can not reliably predict absolute stresses or plastic rotations. In general, the analysis procedures of FEMA 178 are acceptable. Refer to Section 5.6.4.3 commentary.*

Code demands and capacities may reflect specific Performance Objectives. If the building's objective is different, code criteria will be useful in a qualitative sense only.

5.6.4.2 Mass

Seismic forces are proportional to the inertial mass of the structure. The simplified analysis requires mass estimates at each level of the structure, considering both structural and architectural elements. The estimate should include ceilings, roof coverings, floor toppings, wall covering, and other items considered permanent parts of the building.

Commentary: *In general, simplified analysis for identifying potential deficiencies requires relative forces only. However, if a detailed analysis per Chapters 8 through 11 is anticipated, the use of dead load plus likely live load (see Section 9.2) for this simplified analysis can avoid some duplication of effort. Also, useful demand/capacity ratios for columns and other components subject to axial-flexural load may require reasonably accurate estimates of gravity load.*

5.6.4.3 Global Demand

Because the preliminary evaluation is intended to identify broad patterns of expected behavior, useful results can be obtained with any set of lateral forces that approximates the building's modal response. A building code or FEMA 178 force level and distribution is simple, convenient, and sufficiently accurate. (If FEMA 178 demands are used, C_A and C_V values from Chapter 4 may be used.) It is important to keep in mind, however, that codes and handbook documents like FEMA 178 often reflect particular performance objectives that may be different from those chosen for the subject building (see Chapter 3). Thus, while simplified analysis with these demands may yield useful results, actual expected demands (Chapters 3 and 4) may be quite different.

Commentary: *FEMA 178 demands and response coefficients (R values) imply a Performance Objective of Life Safety in a modified Design Earthquake. If the subject building's performance objective involves a different earthquake or a different desired performance level, FEMA 178 acceptance criteria might not be appropriate. For preliminary evaluation, however, the goal is a general understanding of expected response, so absolute demand levels and quantitative acceptance criteria are unimportant.*

5.6.4.4 Component Demand

The seismic demand forces derived in accordance with Section 5.6.4.3 must be distributed to the various components (including connections) that constitute the seismic load path. The distribution may be made on the basis of relative rigidities of horizontal diaphragms and vertical shear resisting elements. Particular attention should be paid to portions of the load path where vertical elements are discontinuous.

Commentary: Approximate force distributions and hand calculations are appropriate for and consistent with the precision of this simplified analysis. Labor intensive refinements will not appreciably improve the precision of the FEMA 178 methodology.

Preliminary evaluation should provide a sense of expected inelastic performance but in most cases will also indicate a need for more detailed analysis.

5.6.4.5 Deficiency Definition

To identify potential deficiencies, the force actions are compared to corresponding capacities as demand/capacity ratios (DCRs). Demands for DCR calculations must include gravity effects. Evaluation statement responses should be reviewed and completed in light of calculated DCRs. High DCRs and conditions that yield False statements indicate potential deficiencies.

Commentary: Capacities for DCR calculations may be established with the FEMA 178 guidelines and other reference sources that provide an estimate of usable strength. "High DCRs" must be judged considering the demands and capacities used and the building's actual performance objective. Assuming FEMA 178 demands and capacities, and a performance objective consistent with that assumed by FEMA 178, DCRs approaching 1.0 (or higher) indicate potential deficiencies, whose significance must be evaluated by the engineer (refer to Section 5.6.4.3 commentary).

5.7 Preliminary Evaluation of Anticipated Seismic Performance

The analyses and assessment methods described in this chapter constitute a "preliminary evaluation" in contrast to the "detailed" approaches described in Chapters 8 through 11. The preliminary evaluation is intended to help develop a general sense of the structure's expected performance in the absence of a detailed analysis. The preliminary evaluation is based on potential deficiencies determined in accordance with Section 5.6.

Commentary: Potential deficiencies include high demand/capacity ratios and False evaluation statements. As noted in Section 5.6.4.3, the global demand used for simplified analysis may or may not approximate the seismic forces derived in accordance with Chapter 4 for the building's performance objective. If they do, then rough thresholds for acceptable DCRs may be set based on engineering judgment and project-specific requirements. If they do not, then absolute DCR values must be read with careful consideration of differences between applied and expected demands. In general, the location and distribution of the critical, highest DCRs is most instructive.

The most critical potential deficiencies are those which, if realized in an earthquake, can alter the building's overall inelastic response. The consequences of potential inelastic response should be considered at both local and global levels, as discussed in the following sections.

5.7.1 Member Inelasticity

For the highest demand/capacity ratio at each floor and at each part of the load path (the locally highest DCR), determine whether the associated behavior is force-controlled or deformation-controlled (refer to Figure 5-9 in Section 5.7.2, and Section 9.5.4.1). Force-controlled actions more frequently produce unacceptable seismic performance. The highest force-controlled DCR should be determined. Review the list of typical

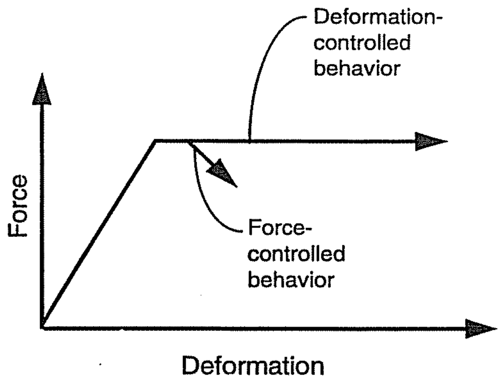


Figure 5-9. Comparison of Force—vs. Deformation—Controlled Behavior

deficiencies (Sections 5.3.2.2 and 5.3.3.2) for Concrete Frame and Concrete Frame-Wall buildings and identify corresponding DCRs.

Locally highest DCRs in force-controlled actions such as shear are critical deficiencies, and locally highest DCRs in deformation-controlled components *may be* critical deficiencies. Very high DCRs in deformation-controlled components are likely to reflect deficiencies since early yielding of these elements will concentrate inelastic deformations among just a few members as opposed to a more desirable distribution among many members.

5.7.2 Response Scenario

Global inelastic response can be understood as the sequential yielding and degrading of individual load path components. The sequence of yielding can be approximated by listing demand/capacity ratios from highest to lowest. Combined with considerations of horizontal and vertical DCR distributions, such a list suggests a "response scenario," i.e. a conceptual representation of the structure's anticipated seismic performance.

***Commentary:** Because forces in a real structure are redistributed after some components hinge or degrade, this approximation of hinging sequence is only reasonable for the first few hinges, and only if their DCRs are relatively close in value.*

DCR demands must include gravity effects. If the proportion of capacity used for gravity load varies widely among critical components, the sequence of hinging may be better approximated by D_E/C_E ratios as recommended by FEMA 178. In this ratio, the demand is due to seismic loads only, and the capacity is reduced to that available for seismic loads after gravity loads have been applied.

The horizontal distribution of the highest demand/capacity ratios should be noted. High DCRs concentrated at one end or in one portion of the structure, indicate a potential for local failure or loss of torsional stability. High ratios that are scattered suggest that the onset of post elastic structural behavior will be well distributed, a beneficial attribute.

The vertical distribution and magnitude of the highest demand/capacity ratios should be noted. High ratios in vertical shear-resisting elements concentrated at a particular level indicate a potential weak story. As previously noted, such a deficiency is common to Concrete Frame-Wall buildings.

5.8 Preliminary Evaluation Conclusions and Recommendations

5.8.1 General

Preliminary evaluation findings should be reported in a context that facilitates appropriate decision making. In general, the preliminary evaluation will support one of three basic options (although others are certainly conceivable): Retrofitting Recommended, Detailed Evaluation Recommended, Performance Acceptable.

5.8.2 Retrofitting Recommended

When the preliminary evaluation reveals a significant number of critical force-controlled deficiencies, then a more detailed analysis may not be needed to conclude that retrofit is required. Conditions that may lead to this conclusion include critical DCRs in force-controlled load path elements without backup systems, locally highest DCRs clustered at one floor (suggesting a weak story), and critical deficiencies at the perimeter of the building (suggesting torsional instability). Even when preliminary evaluation suggests strongly that retrofit is required, detailed evaluation (Chapters 8-11) is still expected to provide a more thorough understanding of expected performance.

5.8.3 Detailed Evaluation Recommended

For structures whose critical elements and components are deformation-controlled, detailed evaluation (Chapters 8 through 11) is usually appropriate. Unless the critical demand/capacity ratios are clustered or are extremely high, deformation-controlled deficiencies may not be so severe so as to prevent acceptable performance. A detailed evaluation may demonstrate acceptable performance in structures that appear from simplified analysis to be deficient.

5.8.4 Acceptable Performance

Only in very rare circumstances can acceptable performance be reliably assured without detailed analysis. As such, no general criteria for reaching this conclusion are available, although it is clear that acceptable performance based on preliminary evaluation would certainly require very low demand/capacity ratios (considering the actual Performance Objective), "True" responses to all or most evaluation statements, and reliable building response scenarios.

5.8.5 Other Considerations

5.8.5.1 Related Structural Hazards

Adjacent structures that may interact seismically with the subject building should be considered in the preliminary evaluation conclusions and recommendations. If pounding appears likely to affect the expected performance, then retrofit modifications to mitigate its effects should be recommended even if the building is otherwise acceptable. This is true for potential ground failures as well.

5.8.5.2 Potential Nonstructural Deficiencies

Identification of nonstructural deficiencies is presented in Chapter 12. Preliminary evaluation conclusions should consider the nature and extent of nonstructural deficiencies.

Normally, life safety concerns associated with nonstructural components are of greatest concern (see Table 5-5). Besides the potential risk of components falling directly on building occupants, some components (e.g. hazardous materials, pressurized piping) pose secondary hazards. Life safety also depends on the ease of egress from an affected building under emergency conditions. Both high accelerations and large relative displacements can damage nonstructural components. Chapter 12 provides further guidance on the acceptability of nonstructural damage depending on the overall performance goals for the building.

Commentary: If the scope of nonstructural deficiencies is extensive, retrofitting may be the most cost effective approach. If architectural finishes must be removed in large areas in order to mitigate nonstructural deficiencies, a significant portion of the cost of modifying an underlying structural component will already be incurred.

5.8.5.3 Additional Testing

When preliminary evaluation conclusions rely on assumptions or incomplete information, additional testing may be warranted before a course of action is determined. Additional testing can also be used to rule out potential deficiencies.

Table 5-5. General Characteristics of Nonstructural Component (adapted from ATC, 1996a)

Component		***Potential Life Safety Hazards***	***Damage Sensitivity***	
			Acceleration	***Drift***
Architectural				
Exterior skin	Adhered veneer	✓	✓	✓
	Glass blocks	✓	✓	✓
	Prefabricated panels	✓	✓	✓
	Glazing systems	✓	✓	✓
Partitions	Heavy	✓	✓	✓
	Light		✓	✓
Interior veneers	Stone, including marble	✓	✓	✓
	Ceramic tile	✓	✓	✓
Ceilings	Directly applied to structure			✓
	Dropped, furred gyp.bd		✓	
	Suspended lath & plaster	✓	✓	✓
	Suspended integrated		✓	✓
Parapets and appendages		✓	✓	
Canopies and marquees		✓	✓	
Chimneys and stacks		✓	✓	
Stairs		✓	✓	✓
Mechanical				
Equipment	Boilers and furnaces	✓	✓	
	Mfg. and process mach.		✓	
	HVAC equip.(isolated)		✓	
	HVAC equip. (nonisolated)		✓	
Storage vessels	Structural supported		✓	
	Flat bottomed		✓	
High pressure piping		✓	✓	✓
Fire suppression piping		✓	✓	✓
Fluid piping	Hazardous materials	✓	✓	✓
	Non-hazardous		✓	✓
Ductwork			✓	✓
Electrical /Communications				
Equipment			✓	
Distribution			✓	✓

Table 5-5. (Continued) General Characteristics of Nonstructural Component (adapted from ATC, 1996a)

		Damage Sensitivity		
Component		**Potential Life Safety Hazards**	**Acceleration**	**Drift**
Light fixtures	Recessed		✓	
	Surface mounted		✓	
	Integrated ceiling	✓	✓	✓
	Pendant		✓	
Furnishings and Equipment				
Storage racks		✓	✓	
Bookcases		✓	✓	
Computer floors			✓	✓
Hazardous materials storage		✓	✓	
Computer/comm. racks			✓	✓
Elevators		✓	✓	✓
Conveyors			✓	✓

Chapter 6

Retrofit Strategies

Audience Interest Spectrum

Owner	Architect	Bldg. Official	Engineer	Analyst
▓	░	░	▓	░

6.1 Introduction

6.1.1 General

This chapter presents an overview of the process used to develop a retrofit strategy and preliminary retrofit design for a building once an evaluation has been conducted in accordance with Chapter 5 and the presence of unacceptable seismic deficiencies has been detected. It includes discussion of alternative retrofit strategies, evaluation of their applicability given the identified seismic deficiencies and the various design constraints, selection of the most appropriate strategy in light of these constraints, and within the context of the selected strategy, selection of an appropriate retrofit system and development of a preliminary retrofit design.

For most buildings and performance objectives, a number of alternative strategies and systems may result in acceptable design solutions. Prior to adopting a particular strategy, the engineer should evaluate a number of alternatives for feasibility and applicability and, together with the owner, should select the strategy or combination of strategies that appears to provide the most favorable overall solution. It is not possible to do this until the owner's performance objectives for the building have been identified and an evaluation of the building has been performed. This evaluation will determine whether the building is capable, in its existing configuration, of providing this performance and, if not, the extent of the any existing deficiencies.

Commentary: *Traditionally, engineers performing seismic evaluations and retrofit designs have used a code-based approach. The process was initiated by evaluating the adequacy of the building to resist lateral forces with a specified "base shear," typically taken to be in the range of 75 percent to 100 percent of the base shear specified by the building code for design of new structures. Typical deficiencies identified in such an evaluation would be that certain elements of the building's lateral force resisting system were significantly "overstressed" or that lateral drifts were excessive. Based on these findings, engineers would then design new supplemental lateral force resisting elements, such as shear walls or braced frames, in order to eliminate the calculated overstress conditions and reduce lateral drifts to acceptable levels. In some extreme cases, engineers would design the new elements to provide the entire lateral force resistance for the structure, negating their need, if not the building's, to rely on the existing element's behavior.*

This traditional approach is both straight-forward and simple to apply. However, it frequently leads to design solutions that are less than optimal given considerations of cost, effect on building appearance, and other pertinent factors. Further, in many cases this traditional approach *will not lead to a technically acceptable design solution. The building code provisions for new construction have closely related requirements for both lateral strength and detailing. It is typically impossible to upgrade an existing building to conform with the detailing requirements of the current building codes. Upgrades that address only the code strength requirements without also addressing the detailing requirements can not be expected to provide the same performance as that intended for new buildings.*

Finally, there is the issue of performance itself. The performance objectives inherent in the building code provisions for new construction are based on consensus judgment as to an appropriate balance between initial construction costs, occupant safety and life time costs. This same balance may not be appropriate for many existing buildings, suggesting that other performance objectives should be selected as a basis for design. Unfortunately, the building code provisions do not currently provide a means to design for alternative performance objectives.

The purpose of this chapter is to provide the engineer with a systematic approach for evaluating the various strategies and systems that may be applicable to a seismic upgrade project and for selecting the strategy and system that provide an optimal design solution, given the desired performance objectives, building characteristics and technical features of the existing structure. In addition to the traditional retrofit approaches described above, this chapter also includes consideration of a number of strategies that are more technically complex.

The methodology indicated for the evaluation of these various strategies to determine their applicability is also somewhat complex compared with traditional code-based approaches of structural evaluation. Often, the effort involved in evaluating multiple upgrade strategies and selecting an optimal one can by itself be quite time consuming and costly. For simple structures, with deficiencies that can be simply and economically mitigated with the introduction of additional strength and stiffness to the lateral force resisting system, detailed evaluation of many alternative strategies is neither warranted nor necessary. Nevertheless, for large structures or for buildings with complex structural systems, the additional effort involved in the evaluation of alternative retrofit strategies, as suggested in this chapter, can result in significant reductions in retrofit cost and lead to design solutions that are more appropriate than those that would otherwise be obtained.

Prior to embarking on a program that includes consideration of many design alternatives, the engineer should exercise his or her individual judgment to determine whether such effort is warranted, given the configuration, deficiencies, and performance objectives intended for the building.

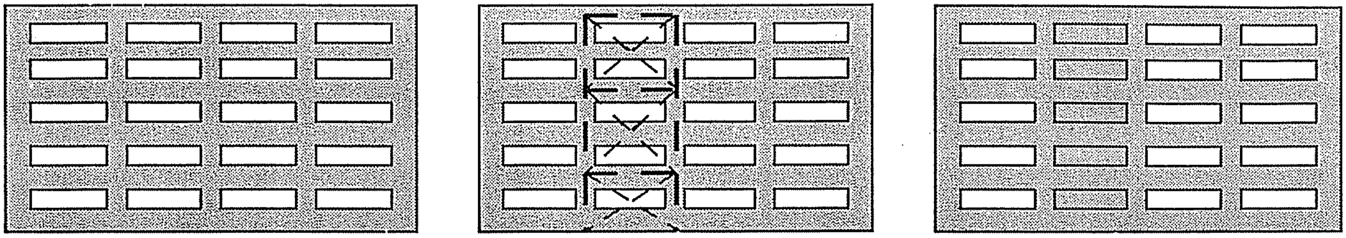
Following selection of the retrofit strategy, it is necessary to select a specific retrofit system and perform a preliminary design. The retrofit system and preliminary design are selected within the context of the selected strategy. Generally, the same factors considered in the strategy selection must also be kept in mind during system selection and preliminary design.

Once a preliminary design is obtained, verification of the adequacy of this design must be performed in accordance with the procedures of Chapter 8 and using the acceptance criteria of Chapter 11. If the preliminary design is found to be inadequate, then it must be modified and the process repeated. When the verification analysis indicates that acceptable performance can be obtained, final detailing of the design may be performed.

6.1.2 Definitions

6.1.2.1 Retrofit Strategies

A retrofit strategy is a basic approach adopted to improve the probable seismic performance of the building or otherwise reduce the existing risk to an acceptable level. Both technical strategies and management strategies can be employed to obtain seismic risk reduction. Technical strategies



a. Unstrengthened building b. Building with retrofit braced frame c. Building with retrofit infill windows

Figure 6-1. Building with Perimeter Concrete Walls

include such approaches as increasing building strength, correcting critical deficiencies, altering stiffness, and reducing demand. Management strategies include such approaches as change of occupancy, incremental improvement, and phased construction.

Commentary: *Engineers sometimes confuse systems and strategies. Strategies relate to modification or control of the basic parameters that affect a building's earthquake performance. These include the building's stiffness, strength, deformation capacity, and ability to dissipate energy, as well as the strength and character of ground motion transmitted to the building and the occupant and contents exposure within the building. Seismic risk reduction strategies include such approaches as increasing strength, increasing stiffness, increasing deformability, increasing damping, reducing occupancy exposure, and modifying the character of the ground motion transmitted to the building. Strategies can also include combinations of these approaches. Retrofit systems are specific methods used to implement the strategy such as, for example, the addition of shear walls or braced frames to increase stiffness and strength, the use of confinement jackets to enhance deformability.*

6.1.2.2 Retrofit System

A retrofit system is the specific method used to achieve the selected strategy. For example, if the basic strategy is to increase building strength, then alternative systems that may be used to accomplish this strategy could include addition of new shear walls, thickening of existing shear walls, and addition of braced frames. While the retrofit systems are closely tied to the strategies, it is not necessary to select a specific system in order to evaluate the applicability of a given strategy. However, it is necessary to select a specific system in order to complete a design.

Commentary: *One of the most important considerations in the selection and design of a retrofit system is its deflection compatibility with the existing structure. Some existing buildings are both weak and brittle, while also being quite stiff. A common strategy for upgrading such a building is to provide supplemental lateral strength. If such strengthening is to be effective, the supplemental elements provided to add this strength must be sufficiently stiff that the added strength can be mobilized prior to the onset of unacceptable damage to the existing elements.*

As an illustrative example, consider the concrete shear wall building illustrated in Figure 6-1a. This building's seismic behavior is controlled by the wall piers, present between the windows. These wall piers have low shear strength and could experience brittle shear failure if the building experiences strong ground motion. Although relatively weak, these piers are also quite stiff. For the purpose of this example, it is presumed that it has been decided to adopt a strategy of structural strengthening.

Structural strengthening can be achieved by a number of alternative systems, two of which are shown in the figure. Figure 6-1b illustrates a system consisting of the addition of supplemental braced steel frames. Figure 6-1c shows a system consisting of reinforced concrete infills within existing window spaces. The braced frame system may be incompatible with the existing building. That is, although the braced frame could add substantial strength to the building, it may not be practical to make this frame as rigid as the existing wall piers, which would therefore still experience shear failure before the braced frame became effective. The window infills, shown in Figure 6-1c, however, having stiffness in excess of that of the original building piers, would be able to adequately protect the building against such damage.

The methodology presented in this document has been specifically developed to ensure appropriate consideration of the deflection compatibility of retrofit systems with the existing structure.

6.1.2.3 Design Constraints

Design constraints are factors other than the building's structural characteristics that affect the ability of a retrofit strategy or system to be effectively implemented. Design constraints that may affect the applicability of a given retrofit strategy include the intended performance objectives, design and construction cost limits, the project schedule, historic preservation requirements, the effects on building appearance and floor space layout, the effects on building occupancy both during and after project construction, and issues of project risk.

Commentary: *Design constraints are typically set by persons other than the design team. These can include political figures, who determine the prioritization and scope of programs affecting state-owned buildings; the client agency, which determines project cost and schedule constraints; the building occupants, who determine the functions that must occur in the building, the extent to which the building will be occupied during construction, the systems that must remain operable during and after construction, the extent to which floor layouts and visual effects may change, and the types of earthquake damage that may be acceptable; the building official, and fire marshal, who determine the extent to which collateral (i.e., disabled access and fire/life safety) upgrades must be performed; historic preservation boards, which determine the permissible extent of alteration to historic fabrics and spaces; and various special interest groups within the community, which may have a significant ability to affect the way in which a project is executed or even prevent its execution.*

6.2 Alternative Retrofit Strategies

A wide range of technical and management strategies are available for reducing the seismic risk inherent in an existing building. Technical strategies are approaches to modifying the basic demand and response parameters of the building for the Design Earthquake. These strategies include system completion, system strengthening, system stiffening, enhancing deformation capacity, enhancing energy dissipation capacity, and reducing building demand.

Commentary: *In the past, many engineers have adopted a strategy of retrofitting the building to "meet current code" or to provide a specified fraction of the base shear capacity contained in the current code. This is not really a strategy at all, but rather a design criterion, often inappropriate, for use with one of the aforementioned strategies. Basic approaches such as strengthening a building, or stiffening it, or adding energy dissipation capacity, are all strategies. Design criteria pertain to the amount of strength, stiffness, energy dissipation capacity, or other attributes that are required in order to meet given performance objectives. In this methodology, the basic design criterion is to maintain damage to critical elements of the building within levels appropriate to the design performance objective by* *controlling lateral drift. Chapters 9 and 10 provide design criteria that may be used respectively in judging the acceptability of likely damage to structural and nonstructural components and elements of the building under the Design Earthquake. In general, these are not directly tied to the building code.*

In addition to technical strategies, a number of alternative management strategies should also be considered. Management strategies could include decisions to implement the retrofit while the building remained occupied; to vacate the building until the retrofit could be performed; to accept the existing risk and not retrofit; to change building occupancy so that the risk is acceptable; to demolish the building and replace it with an alternative facility; to implement the technical strategy on a phased basis over a number of years; or to retrofit on a temporary basis until replacement facilities can be obtained. Additional management strategies could include performing all work on the building exterior, possibly to minimize the impact on building occupants, and performing all work on the building interior, in order to preserve the building's exterior appearance.

Commentary: Management strategies have often been regarded as beyond the design engineer's scope of concern. Nevertheless, it is very important that the engineer consider these alternative management strategies and assist the client in selecting an appropriate strategy. The management strategy selected can have a great effect on the feasibility and cost of implementing the various technical strategies. Further, the best solution for a building is often one that involves management rather than technical action, such as changing the building occupancy or constructing a replacement facility.

6.2.1 Technical Strategies

As a building responds to earthquake ground motion, it experiences lateral displacements and, in turn, deformations of its individual elements. At low levels of response, the element deformations will be within their elastic (linear) range and no damage will occur. At higher levels of response, element deformations will exceed their linear elastic capacities and the building will experience damage. In order to provide reliable seismic performance, a building must have a complete lateral force resisting system, capable of limiting earthquake-induced lateral displacements to levels at which the damage sustained by the building's elements will be within acceptable levels for the intended performance objective. The basic factors that affect the lateral force resisting system's ability to do this include the building's mass, stiffness, damping, and configuration; the deformation capacity of its elements; and the strength and character of the ground motion it must resist.

Commentary: *The technical strategies described in this methodology provide for improved seismic performance by directly operating on these basic response factors, either individually or in concert. The traditional approaches to seismic retrofit – the addition of braced frames and shear walls – operate on building stiffness and strength. Energy dissipation systems operate on the structure's damping capability. Base isolation operates on the character and strength of ground motion transmitted to the structure.*

There are a number of analysis procedures (see Chapter 8) that compare measures of seismic capacity and seismic demand to evaluate existing structures. These methods are also useful to investigate or verify the effectiveness of technical retrofit strategies. The following discussion uses the Capacity Spectrum Method to illustrate the basic process.

The capacity spectrum is derived from an approximate nonlinear, incremental static analysis for the structure. In the process of performing this incremental nonlinear static analysis, a capacity curve is developed for the building. This capacity curve is simply a plot of the total lateral seismic shear demand, "V," on the structure, at various

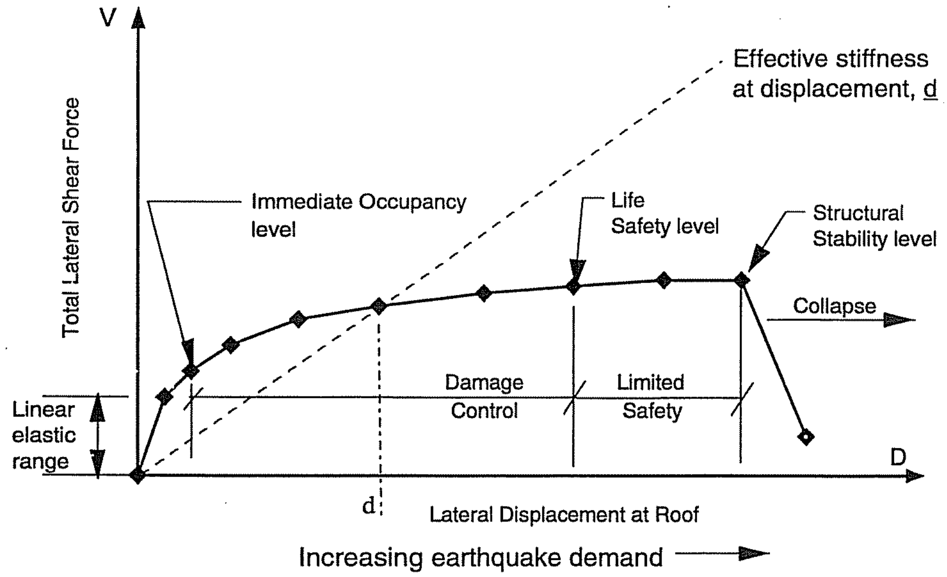


Figure 6-2. Typical Capacity Curve

increments of loading, against the lateral deflection of the building at the roof level, under that applied lateral force. If a building had infinite linear elastic capacity, this capacity curve would be a straight line with a slope equal to the global stiffness of the structure. Since real buildings do not have infinite linear elastic capacities, the capacity curve typically consists of a series of straight-line segments with decreasing slope, representing the progressive degradation in structural stiffness that occurs as the building is subjected to increased lateral displacement, yielding, and damage. The slope of a straight line drawn from the origin of the plot for this curve to a point on the curve at any lateral displacement, "*d*," represents the secant or "effective" stiffness of the structure when pushed laterally to that displacement. A typical capacity curve is shown in Figure 6-2.

In Figure 6-2, the discrete points indicated by the symbol ◆ represent the occurrence of important events in the lateral response history of the structure. Such an event may be the initiation of yield in a particular structural element or a particular type of damage, such as spalling of cover concrete on a column or shear failure of a spandrel element. Each point is determined by a different analysis sequence described in Section 8.2.1. Then, by evaluating the cumulative effects of damage sustained at each of the individual events, and the overall behavior of the structure at increasing lateral displacements, it is possible to determine and indicate on the capacity curve those total structural lateral displacements that represent limits on the various structural performance levels, as has been done in Figure 6-2. Chapter 11 provides specific guidance on acceptance criteria that may be used to determine the lateral deformations for a specific structure that correspond to these performance limits based on story drift as well as element and component strength and deformation capacities.

Commentary: *The process of defining lateral deformation points on the capacity curve at which specific structural performance levels may be said to have occurred requires the exercise of considerable judgment on the part of the engineer. For* *each of the several structural performance levels defined in Chapter 3, Chapter 11 of this methodology defines global system response limits as well as acceptance criteria for the individual structural elements that make up typical buildings. These acceptance criteria generally consist of limiting values of element deformation parameters, such as the plastic chord rotation of a beam or shear angle of a wall. These limiting values have been selected as reasonable approximate estimates of the average deformations at which certain types of element behavior such as cracking, yielding, spalling, or crushing, may be expected to occur. As the incremental static nonlinear analyses are performed, the engineer must monitor the cumulative deformations of all important structural elements and evaluate them against the acceptance criteria contained in Chapter 11.*

The point on the capacity curve at which the first element exceeds the permissible deformation parameter for a structural performance level does not necessarily represent the point at which the structure as a whole reaches that structural performance level. Most structures contain many elements and have considerable redundancy. Consequently, the onset of unacceptable damage to a small percentage of these elements may not represent an unacceptable condition with regard to the overall performance of the building. When determining the points along the capacity curve for a structure at which the various structural performance levels may be said to be reached, the engineer must view the performance of the building as a whole and consider the importance of damage predicted for the various elements on the overall behavior of the building.

This methodology incorporates the concept of "primary" and "secondary" elements to assist the engineer in making these judgments. Primary elements are those that are required as part of the lateral force resisting system for the structure. All other elements are designated as secondary elements. For a given performance level, secondary elements are generally permitted to sustain more damage than primary elements since degradation of secondary elements does not have a significant effect on the lateral load resisting capability of the building. If in the development of the capacity curve it is determined that a few elements fail to meet the acceptance criteria for a given performance level at an increment of lateral loading and displacement, the engineer has the ability to designate these "nonconforming" elements as secondary, enabling the use of more liberal acceptance criteria for these few elements. Care must be exercised not to designate an excessive number of elements that are effective in resisting lateral forces as secondary. Chapter 9 provides further guidance on this issue.

The capacity spectrum curve for the structure is obtained by transforming the capacity curve from lateral force (V) vs. lateral displacement (d) coordinates to spectral acceleration (S_a) vs. spectral displacement (S_d) coordinates using the modal shape vectors, participation factors and modal masses obtained from a modal analysis of the structure (see "Conversion to ADRS Spectra" sidebar).

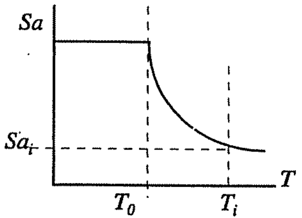
When the capacity curve is plotted in S_a vs. S_d coordinates, radial lines drawn from the origin of the plot through the curve at various spectral displacements have a slope $(\omega')^2$, where ω' is the radial frequency of the effective (or secant) first-mode response of the structure if pushed by an earthquake to that spectral displacement.

Using the relationship $T' = 2\pi/\omega'$, it is possible to calculate, for each of these radial lines, the effective period of the structure if it is pushed to given spectral displacements. Figure 6-3 is a capacity spectrum plot obtained from the capacity curve shown in Figure 6-2 and plotted with the effective modal periods shown. The particular structure represented by this plot would have an elastic period of approximately 1/2 second. At is pushed progressively farther by stronger ground motion, this period lengthens. The building represented in Figures 6-2 and 6-3 would experience collapse before having its stiffness degraded enough enough to produce an effective period of 2 seconds.

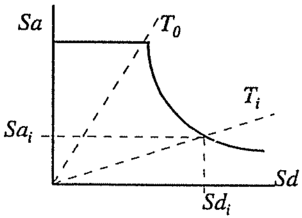
Conversion to ADRS Spectra

Application of the Capacity-Spectrum technique requires that both the demand response spectra and structural capacity (or pushover) curves 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., 1993.

Response Spectrum Conversion



Standard Format (Sa vs T)



ADRS Format (Sa vs Sd)

Every point on a response spectrum curve has associated with it a unique spectral acceleration, S_a, spectral velocity, S_v, spectral displacement, S_d and period, T. To convert a spectrum from the standard S_a vs T format found in the building code to ADRS format, it is necessary to determine the value of S_{di} for each point on the curve, S_{ai}, T_i. This can be done with the equation:

$$S_{d_i} = \frac{T_i^2}{4\pi^2} S_{a_i} g$$

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:

$$S_{a_i} g = \frac{2\pi}{T_i} S_v \qquad S_{di} = \frac{T_i}{2\pi} S_v$$

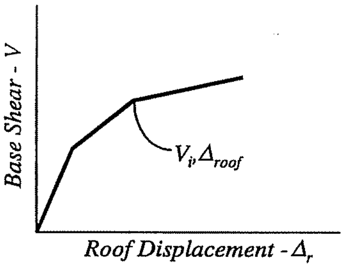
Capacity Spectrum Conversion

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 equations:

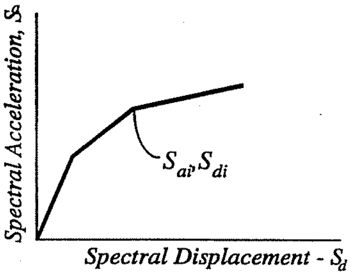
$$S_{a_i} = \frac{\mathrm{V_i} / W}{\alpha_1}$$

$$S_{d_i} = \frac{\Delta_{roof}}{\left(PF_1 \times \phi_{1,roof}\right)}$$

where α_1 and PF_1 are respectively the modal mass coefficient and participation factors for the first natural mode of the structure and $\phi_{1,roof}$ is the roof level amplitude of the first mode. See also Section 8.5, Basics of Structural Dynamics.



Capacity Curve



Capacity Spectrum

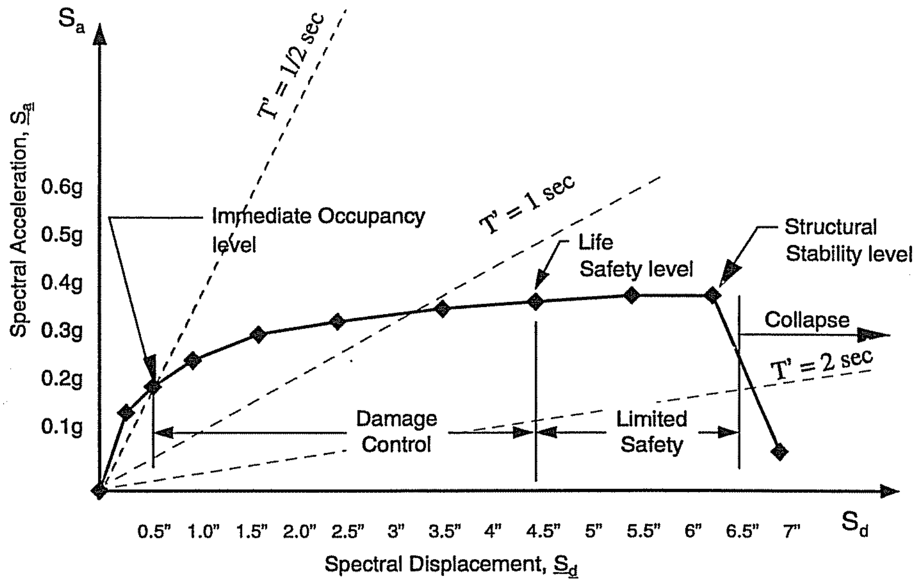


Figure 6-3. Typical Capacity Spectrum

The capacity of a particular building and the demand imposed upon it by a given earthquake motion are not independent. One source of this mutual dependence is evident from the capacity curve itself. As the demand increases, the structure eventually yields and, as its stiffness decreases, its period lengthens. Conversion of the capacity curve to spectral ordinates (ADRS) makes this concept easier to visualize. Since the seismic accelerations depend on period, demand also changes as the structure yields. Another source of mutual dependence between capacity and demand is effective damping. As a building yields in response to seismic demand it dissipates energy with hysteretic damping. Buildings that have large, stable hysteresis loops during cyclic yielding dissipate more energy than those with pinched loops caused by degradation of strength and stiffness. Since the energy that is dissipated need not be stored in the structure, the effective damping diminishes displacement demand.

The capacity spectrum method initially characterizes seismic demand using an elastic response spectrum described in Chapter 4. This spectrum is plotted in spectral ordinates (ADRS) format showing the spectral acceleration as a function of spectral displacement. This format allows the demand spectrum to be "overlaid" on the capacity spectrum for the building. The intersection of the demand and capacity spectra, if located in the linear range of the capacity, would define the actual displacement for the structure; however this is not normally the case as most analyses include some inelastic nonlinear behavior. To find the point where demand and capacity are equal, the engineer selects a point on the capacity spectrum as an initial estimate. Using the spectral acceleration and displacement defined by this point, the engineer then can calculate reduction factors to apply to the 5% elastic spectrum to account for the hysteretic energy dissipation, or effective damping, associated with the specific point. If the reduced demand spectrum intersects the capacity spectrum at or near the initial assumed point, then it is the solution for the unique point where capacity equals demand. If the intersection

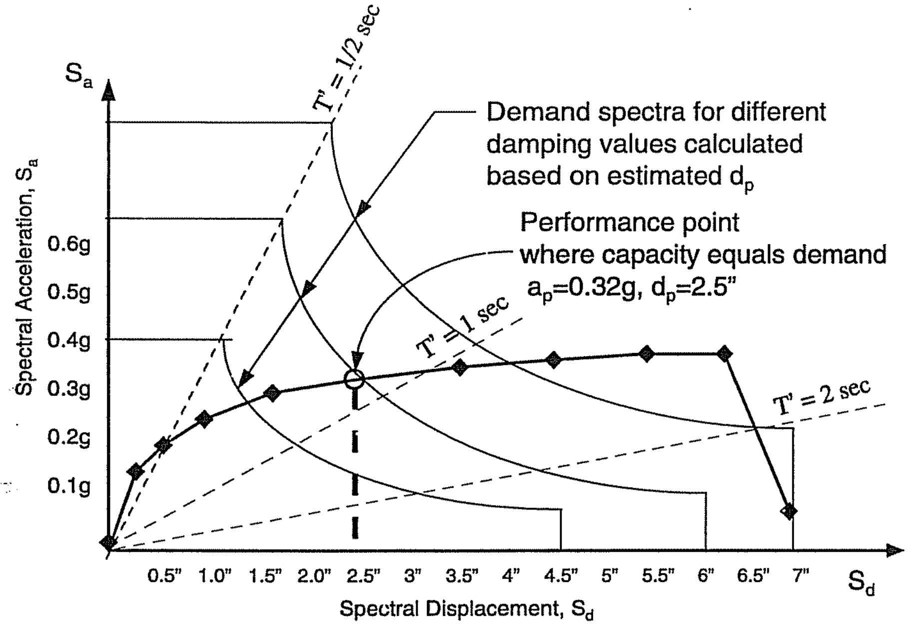


Figure 6-4. Determination of Performance Point

is not reasonably close to the initial point, then the engineer can assume a new point somewhere between and repeat the process until a solution is reached. This is the performance point where the capacity of the structure matches the demand for the specific earthquake.

Once the performance point has been determined, the acceptability of a rehabilitation design to meet the project performance objectives can be judged by evaluating where the performance point falls on the capacity curve. For the structure and earthquake represented by the overlay indicated in Figure 6-4, the performance point occurs within the central portion of the damage control performance range as shown in Figure 6-3, indicating that for this earthquake this structure would have less damage than permitted for the Life Safety level and more than would be permitted for the Immediate Occupancy level. With this information, the effectiveness of the particular rehabilitation strategy to achieve the project performance objectives can be judged. This same technique is used throughout the balance of this chapter to illustrate the way in which the various alternative retrofit strategies may be used to design for project performance objectives.

Commentary: The methodology described here for the determination of the performance point is an approximate approach to determining the nonlinear response of a building to a given ground motion. It should not be considered to be an exact solution with regard to the estimates of displacement response it predicts. For a given ground motion time history, a dynamic nonlinear analysis of the structure may result in somewhat different predictions of maximum structural displacement than does this method. However, such dynamic nonlinear analyses will result in different predictions of displacement even for multiple time histories enveloped by the same response spectrum but having different records. Recent studies have indicated that for structures dominated by first mode response, the capacity spectrum methodology described here provides a *good average estimate of the displacements predicted by multiple time history analyses using different records with the same enveloping spectrum. The method may not be quite as effective for structures with significant higher mode participation in their earthquake response.*

6.2.1.1 System Completion

System completion strategies are applicable to structures that have the basic components of an adequate lateral force resisting system, including diaphragms and walls or frames, but that lack some details required to make the system complete or to ensure that the system behaves as intended. The capacity spectrum for such a structure would typically intersect the demand spectra at an acceptable performance point; before reaching that point, however, some local failure events would occur. Common deficiencies that may lead to such local failures include a lack of adequate chord and collector elements at diaphragms, inadequate bearing length at precast element supports, and inadequate anchorage or bracing of structural or nonstructural components. Correction of these deficiencies through provision of the missing elements would enable the structure to behave in the desired manner. Often this strategy must be implemented together with other strategies to obtain a building with the desired seismic performance capabilities.

Commentary: *Precast concrete tilt-up buildings are an example of a class of building for which system completion strategies are often appropriate. These buildings commonly have adequate shear walls and diaphragms. If a capacity spectrum curve for such a building were to be constructed, it would be found capable of meeting acceptable structural performance levels. However, a common deficiency of these buildings is a lack of adequate anchorage between the precast wall panels and the diaphragms. An appropriate strategy for such a building is to complete the existing structural system through provision of adequate out-of-plane anchorage between the walls and diaphragms.*

6.2.1.1.1 Chords, Collectors, and Drags. Diaphragm chords, collectors, and drags may be constructed of new reinforced concrete beams/struts or of flush-mounted steel plates or members with drilled-in anchors. Where there are existing beams, these may be converted to collector elements by enhancing their capacity or providing strengthening at end connections. This approach is very common for timber diaphragms, a common element of some older concrete construction.

Commentary: *The provision of adequate attachments between new elements and the existing ones is an important consideration for this strategy. Drilled-in anchors, commonly used to attach new elements to existing concrete, often have limited ductility. If inelastic behavior is expected of the new element, the new anchors must have adequate strength to transfer the real demands on the element without brittle failure. Attachments between retrofit steel elements and existing concrete often consist of bolted connections. These bolted connections inherently include some "slop," resulting from the oversize holes in the steel members. Before these elements can become effective, sufficient displacement of the structure must occur to take up this slop. The result could be the onset of significant damage to the structure before the retrofit elements become effective.*

6.2.1.1.2 Element Connectivity. Most concrete structures are monolithically constructed and have adequate nominal interconnection between elements. Buildings that incorporate precast elements may require some supplemental interconnection of elements. This is typically achieved by adding steel hardware between elements at their end connections.

6.2.1.1.3 Anchorage and Bracing of Components. To achieve some performance objectives, the architectural, mechanical, and electrical components of the building must be adequately braced and anchored to resist inertial forces and the drifts the building is expected to

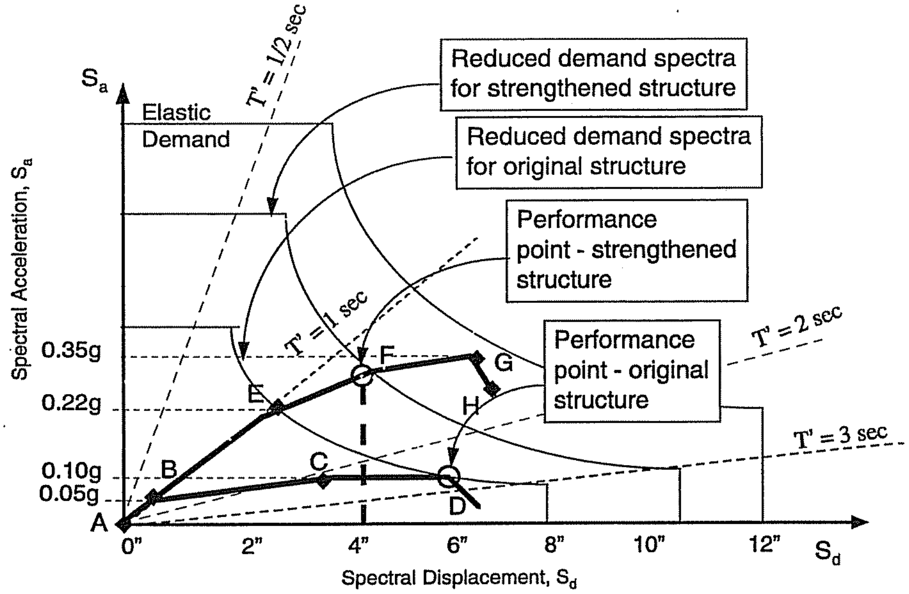


Figure 6-5. Effect of System Strengthening on Performance

experience in response to earthquake ground motion.

Commentary: The adequacy of attachment of nonstructural components to a building can not generally be determined by using the capacity spectrum technique. The more traditional equivalent lateral force technique is typically used to evaluate the effectiveness of existing anchorage and bracing systems and to design retrofit anchorage and bracing. Chapter 10 outlines the recommended methodology for this aspect of seismic retrofitting. However, the capacity spectrum method does provide an excellent tool for estimating the drift the building may experience in the Design Earthquake and therefore, does provide some valuable information for the seismic upgrade of nonstructural components that are drift sensitive.

6.2.1.2 System Strengthening and Stiffening

System strengthening and stiffening are the most common seismic performance improvement strategies adopted for buildings with inadequate lateral force resisting systems. The two are closely related but different. The effect of strengthening a structure is to increase the amount of total lateral force required to initiate damage events within the structure. If this strengthening is done without stiffening, then the effect is to permit the structure to achieve larger lateral displacements without damage. Figure 6-5 uses a demand/capacity spectrum plot to illustrate the effect of system strengthening on earthquake performance.

In Figure 6-5, the capacity spectrum defined by the curve A-B-C-D represents the performance of an unstrengthened structure. It has an initial elastic period of one second, a spectral acceleration capacity at first significant yield of 0.05g, ultimate spectral acceleration capacity of 0.10g and ultimate spectral deformation capacity of approximately six inches. The performance point for this unstrengthened structure occurs at point "D" indicating that the structure just meets the Structural Stability performance level for this earthquake. The curve A-B-E-F-G-H is one possible capacity curve for the structure after it has been strengthened. Note that the initial elastic

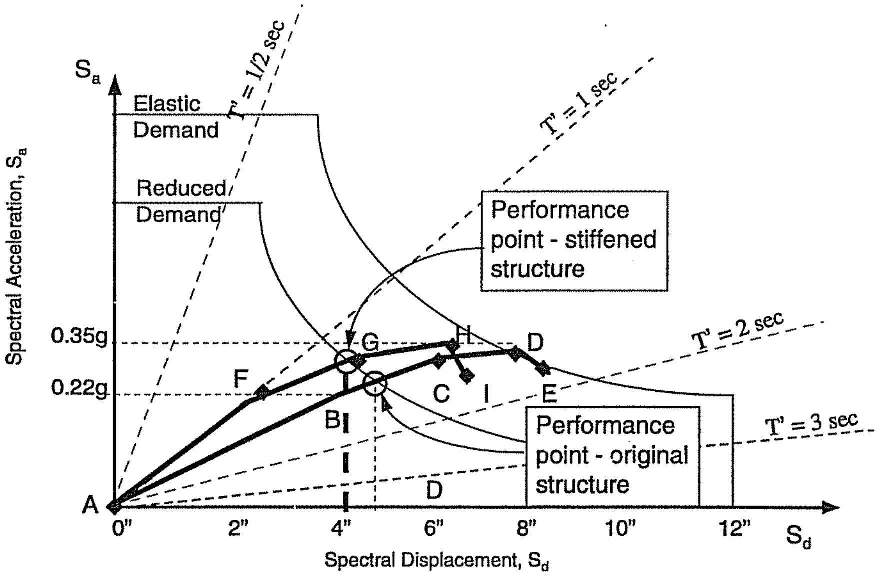


Figure 6-6. Effect of System Stiffening on Performance

period for the structure remains at one second, indicating that the structure has not been stiffened. Also, the ultimate spectral displacement capacity for the structure remains at approximately six inches, indicating it has not been provided with additional deformation capacity. However, the structure has been strengthened resulting in an increase in its spectral acceleration capacity at first significant yield to 0.22g and an ultimate spectral acceleration capacity of 0.35g. This "strengthened" curve has a new performance point, at about 4 inches displacement, well within the requirements for the Structural Stability structural performance level.

Commentary: *Figure 6-5 illustrates the effect of a retrofit system that includes pure strengthening without affecting either the original stiffness of the structure, as illustrated by the slope of the initial segment of the capacity spectra, or the deformation capacity of the structure, as illustrated by the fact that the capacity spectra shown for both the original and the strengthened structure reach their ultimate strength at the same deformation level. In reality, most retrofit systems that increase structural strength, such as the addition of walls or frames, will also increase structural stiffness. Exceptions to this are relatively local retrofit measures that strengthen existing elements without greatly altering their stiffness. For example, a common deficiency of older concrete frames is that the longitudinal reinforcing in these frames has inadequate lap splice lengths, resulting in a low flexural strength. Provision of confinement around the splices can improve their performance and allow the frame to develop greater strength without substantially affecting its stiffness. Such measures are also likely to enhance the frame's deformation capacity, however. The example indicated in Figure 6-5 should therefore be regarded as an abstract, intended to illustrate the way structural strengthening affects the behavior of a structure.*

The effect of stiffening a structure is illustrated in Figure 6-6. Curve A-B-C-D-E in the figure is the performance curve for an unstiffened structure. It has an initial elastic period of approximately 1.5 seconds, a spectral acceleration capacity at first significant yield of 0.22g and an ultimate spectral acceleration capacity of 0.35g. Curve A-F-G-H-I is the performance curve for the same structure after it has been stiffened. The initial elastic period of the stiffened structure is 1 second, while the spectral acceleration capacities at first yield and ultimate remain unchanged. The effect of this stiffening modification is to shift the performance point from a deflection of approximately 4-1/2 inches to a deflection of approximately 4 inches. This does not result in significant change in the structural performance of the lateral force resisting system. However, the performance of elements of the structure that do not participate to a significant extent in resisting lateral forces but that are sensitive to induced lateral deformations can be significant. Such elements could include non-ductile gravity load bearing columns, flat slab systems and architectural partitions and cladding.

System strengthening and stiffening are nearly always performed as concurrent strategies, since most systems that will strengthen a structure also simultaneously stiffen it; similarly, stiffening techniques also usually result in a strength increase.

Typical systems employed for stiffening and strengthening include the addition of new vertical elements, including shear walls, braced frames, buttresses, or moment resisting frames. Diaphragms may need to be added as well.

6.2.1.2.1 Shear Walls. The introduction of shear walls into an existing concrete structure is one of the most commonly employed approaches to seismic upgrading. It is an extremely effective method of increasing both building strength and stiffness. A shear wall system is often economical and tends to be readily compatible with most existing concrete structures.

Commentary: *The addition of shear walls to an existing structure can have some adverse impacts that the engineer should be aware of. If a large number of shear walls are added to a building, they can result in a significant increase in building mass and therefore increase seismic forces and strength requirements. Shear walls can often result in significant architectural impact through the loss of windows and the introduction of barriers within areas of floor space. They also tend to produce large overturning forces at their bases, that may require supplemental foundation work, which is often expensive.*

6.2.1.2.2 Braced Frames. Braced steel frames are another common method of enhancing an existing building's stiffness and strength. Typically, braced frames provide lower levels of stiffness and strength than do shear walls, but they add far less mass to the structure than do shear walls, can be constructed with less disruption of the building, result in less loss of light, and have a smaller effect on traffic patterns within the building.

Commentary: *It is often difficult to effectively attach braced frames to a concrete building because relatively large forces must be transferred between the structure and the braced frames. Typical attachment methods employ drilled-in anchors, which have relatively low strength. The solution is often to use long drag or collector elements that attach the new steel frame to the existing concrete structure by means of a large number of fasteners. If these drag elements are not adequately rigid and strong local deformation of the drag can result in excessive demand on the connectors. This can cause an "unzipping" effect, in which connectors along the length of the drag are sequentially overstressed and fail, shedding the excessive loads to the next group of connectors. Some failures have occurred in seismically retrofitted structures in the past as a result of this effect.*

6.2.1.2.3 Buttresses. Buttresses are braced frames or shear walls installed perpendicular to an exterior wall of the structure to provide supplemental stiffness and strength. This system is often a convenient one to use when a building must remain occupied during construction, as most of the construction work can be performed on the building exterior, minimizing the inconvenience to building occupants. Sometimes a building addition intended to provide additional floor space can be used to buttress the original structure for added seismic resistance.

Commentary: *Braced frames and shear walls placed within a building can often be designed to* *mobilize the weight of the structure to resist overturning demands. Buttresses, situated outside the structure cannot do this and typically require the construction of foundations to provide the necessary overturning resistance. Because the work is performed outside the building line, however, the cost of buttresses may be substantially lower than that for interior shear walls or braced frames, even given the added foundation requirement. Because they are located at the building exterior, buttresses can have significant aesthetic impact; they are seldom appropriate for buildings that are considered historically important.*

6.2.1.2.4 Moment Resisting Frames. Moment-resisting frames can be an effective system to add strength to a building without substantially increasing the building's stiffness. Moment frames have the advantage of being relatively open and therefore can be installed with relatively minimal impact on floor space.

Commentary: *Deflection compatibility is often a problem for this system, since moment-resisting frames must experience relatively large lateral deflections before their strength can be mobilized. Many concrete buildings have relatively limited deformation capacity, and therefore the usefulness of this system in the retrofit of concrete structures is somewhat limited.*

6.2.1.2.5 Diaphragm Strengthening. Most concrete buildings have adequate diaphragms. Exceptions occur where large openings are present or where offsets in the vertical elements of the system produce locally high demands. Methods of enhancing diaphragms include the provision of topping slabs, metal plates laminated onto the top surface of the slab, or horizontal braced diaphragms beneath the concrete slabs. For buildings with timber diaphragms, diaphragm strengthening can be achieved by increasing the existing nailing in the sheathing, replacing the sheathing with stronger material, or overlaying the existing sheathing with plywood.

Commentary: *It is important to consider the behavior of diaphragms when evaluating the performance of a building. If significant diaphragm deformation or inelastic behavior is expected in the building, then this behavior should be modeled in the building analysis and the resulting deformations included in the construction of the capacity curve for the building.*

6.2.1.3 Enhancing Deformation Capacity

Improvement in building seismic performance through enhancement of the ability of individual elements within the building to resist deformations induced by the building response is a relatively new method of seismic upgrading for concrete buildings. Figure 6-7 illustrates the effect of enhancing deformation capacity on building performance.

As illustrated in the figure the capacity curve for the original building (A-B-C-D-E) is unable to intersect the demand curve at a performance point because critical failure of an element occurs prior to this intersection. If this critical element is modified, such that its deformation capacity is increased, then the capacity curve for the building can extend (A-B-C-D-F-G) to larger spectral displacements, allowing the performance point intersection to occur at an acceptable structural performance level.

Systematic methods of enhancing deformation capacity include adding confinement to existing elements, making local reductions in stiffness, modifying columns to alter mechanisms, and providing supplemental support at areas subject to deformation-induced failure. These methods often have significantly less architectural impact than do approaches that involve structural strengthening and/or stiffening.

Commentary: *This strategy is typically most effective when the necessary enhanced deformation capacity can be obtained by modifying only a few existing elements. If the required modifications must be placed throughout the structure, this strategy can become quite costly and disruptive of occupancy during construction.*

6.2.1.3.1 Adding Confinement. The deformation capacity of nonductile concrete

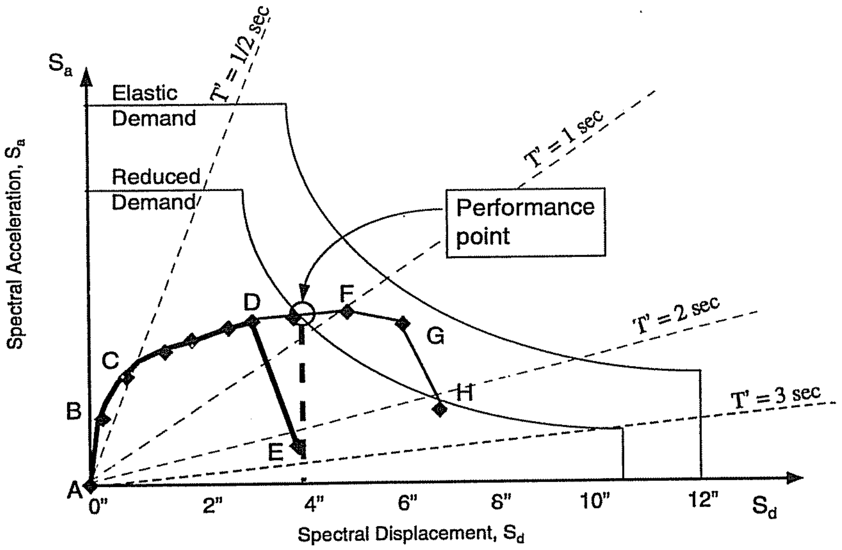


Figure 6-7. Effect of Deformation Enhancement on Structural Performance

columns can be enhanced through provision of exterior confinement jacketing. Jacketing may consist of continuous steel plates encasing the existing element, reinforced concrete annuluses, and fiber-reinforced plastic fabrics.

Commentary: Confinement jacketing can improve the deformation capacity of concrete elements in much the same way that closely spaced hoops in ductile concrete elements do. To be effective, the jacketing material must resist the bursting pressure exerted by the existing concrete element (under the influence of compressive stresses,) in a rigid manner. Circular or oval jackets can provide the necessary confinement in an efficient manner through the development of hoop stresses. Rectangular jackets tend to be less effective and require cross ties in order to develop the required stiffness.

6.2.1.3.2 Column Strengthening. Many older concrete frames incorporate a strong beam-weak column configuration. Buildings with this configuration tend to develop single-story mechanisms in which all of the inelastic deformation demand produced by the earthquake develops within the story in which the mechanism occurs. The concentration of these displacement demands within the single story results in very large local inelastic deformation demands on the columns at relatively low levels of total structure lateral displacement. If the columns can be strengthened such that the beams become the weaker elements, this will inhibit the formation of story mechanisms and will permit much larger overall structural drifts to be attained.

Commentary: Strengthening columns to prevent the formation of story mechanisms may be difficult to accomplish effectively. In addition to strengthening the columns, it is necessary to strengthen the connection between the beams and columns to allow development of the larger moments. In addition, many concrete frames are not reinforced to permit reversing flexural yielding of the beams. Thus the ductility of a frame in which the columns have been strengthened may still be quite limited.

6.2.1.3.3 Local Stiffness Reductions. Local reductions in stiffness can be an effective way to prevent undesirable damage modes as well as to minimize damage to a few scattered elements that are not essential to the building's overall performance. Many older concrete structures are subject to short-column failures at perimeter walls, resulting from the presence of deep spandrels. These effects can often be reduced by introducing joints between the face of the column and adjacent architectural elements, such as spandrel panels or infills, that create the condition. Some buildings may have one or more walls that are present for architectural rather than structural reasons. These walls may be quite stiff and either attract more lateral force than they can resist or introduce torsional response or discontinuous load paths into the structure. Local demolition of these elements, or modification of them to reduce their stiffness, can result in a cost-effective performance improvement for the structure.

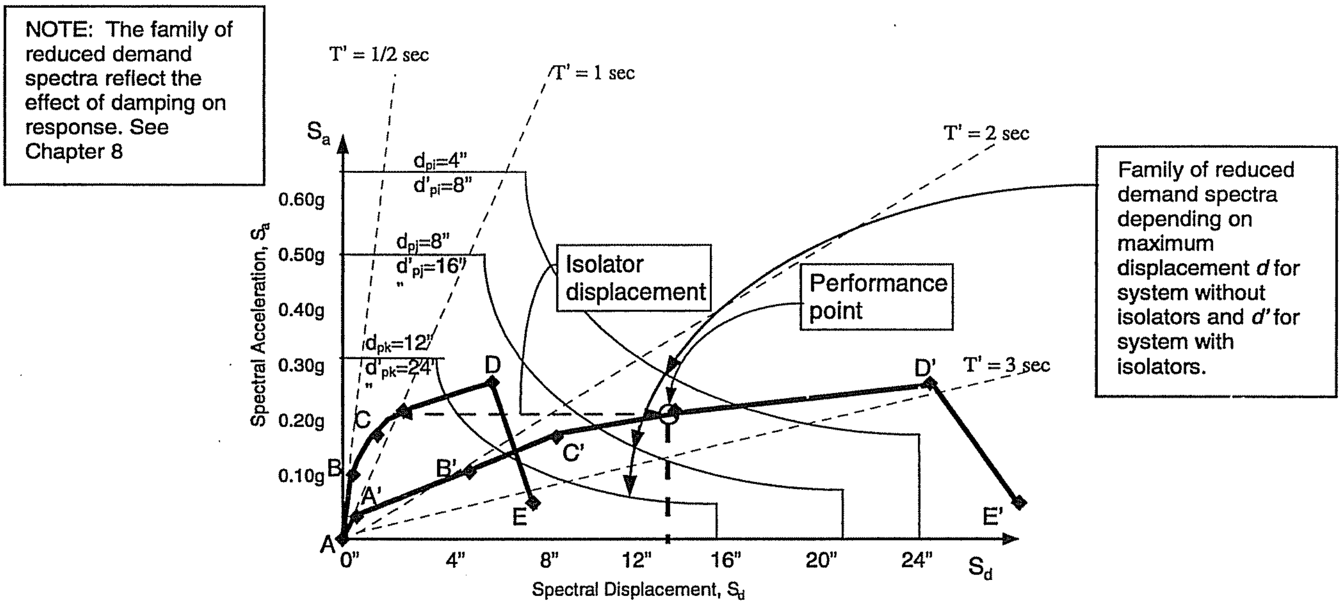
6.2.1.3.4 Supplemental Support. This approach can be effective for the mitigation of deficient gravity load bearing elements that are not significant to the lateral force resistance of the structure but whose support can be jeopardized by large lateral building deflections. For example, flat slabs that may be subject to punching shear failures due to induced lateral building deformations could be provided with supplemental bearing supports at columns. Similarly, precast beams with inadequate bearing length could be provided with supplemental bearing supports.

6.2.1.4 Reducing Earthquake Demands

Rather than modifying the capacity of the building to withstand earthquake-induced forces and deformations, this strategy involves modification of the response of the structure such that the demand forces and deformations are reduced. In effect, the demand spectrum for the structure, rather than the capacity spectrum, is modified. Methods for achieving this strategy include reductions in the building's mass and the installation of systems for base isolation and/or energy dissipation. The installation of these special protective systems within a building typically entails a significantly larger investment than do more-conventional approaches. However, these special systems do have the added benefit of providing for reduced demands on building contents. Consequently, these approaches are often appropriate for buildings housing critical occupancies with sensitive equipment or a need to attain rapid postearthquake functionality. They may also be attractive for the retrofitting of historic structures because they may make it possible to retrofit the structure to be retrofitted without extensive invasive construction within the historic spaces. This benefit can sometimes be over estimated, however, and many structures employing a strategy of reducing earthquake demands also require supplemental strengthening and stiffening.

Commentary: *Seismic retrofits that incorporate base isolation and energy dissipation systems have drawn the structural engineering industry's attention in recent years, and many engineers are eager to apply these technologies in retrofit projects. The practical applicability of both of these technologies is somewhat limited, however. Further, since these technologies are complex, a significant amount of engineering effort is often required in order to develop a preliminary design incorporating them. Engineers should be cautious in committing to detailed evaluations of the applicability of either of these systems unless the added cost appears to be warranted. Further information on the conditions most likely to result in favorable application of these technologies is provided below.*

6.2.1.4.1 Base Isolation. This approach requires the insertion of compliant bearings within a single level of the building's vertical load carrying system, typically near its base. The bearings are designed to have relatively low stiffness, extensive lateral deformation capacity and may also have superior energy dissipation characteristics. Installation of an isolation system results in a substantial increase in the building's fundamental response period and, potentially, its effective damping. Since the isolation bearings have much greater lateral compliance than does the


NOTE: The family of reduced demand spectra reflect the effect of damping on response. See Chapter 8

Family of reduced demand spectra depending on maximum displacement *d* for system without isolators and *d'* for system with isolators.

Figure 6-8. Typical Effect of Base Isolation on Demands and Capacities

structure itself, lateral deformation demands produced by the earthquake tend to concentrate in the bearings themselves. Together these effects result in greatly reduced lateral demands on the portion of the building located above the isolation bearings. Figure 6-8 illustrates the combined effect on the demand and capacity spectra of introducing base isolation into a building.

In the figure, Curve A-B-C-D-E represents the capacity spectrum for the original, unretrofitted structure. This structure has an initial elastic period of approximately 1/2 second, and an ultimate deformation capacity of about 6 inches. First significant yielding occurs at point "B" and the ultimate strength is developed at point D. Since the capacity spectrum does not intersect a demand curve from the d_p family at a corresponding max-imum displacement, the structure would not survive the design earthquake. Curve A-A'-B'-C'-D'-E' is the capacity curve for the structure with base isolation installed. The yield and ultimate strengths of the structure remain unchanged; however, the displacements at which yielding and ultimate behavior occur are greatly increased by the displacement contribution of the isolation bearings. The effective elastic period of the base isolated structure is lengthened to approx-imately 2.5 seconds. Initial yielding of the fixed-base structure (point B) occurs at a spectral displacement of approximately 1/2 inch. For the isolated structure, this same yield behavior (point B') occurs at a spectral displacement of approximately 4 inches. Similarly, ultimate spectral displacement capacity of the isolated structure increases from six inches (point D) to approximately 24 inches (point D').

When a structure is base isolated, the energy dissipation that occurs at a given displacement is significantly different from that which occurs for the same structure in a fixed-base condition. Consequently, in order to overlay the demand and capacity spectra for a base isolated structure, it becomes necessary to recompute the effective damping of the family of demand curves at various maximum structural displacements. This is denoted in the figure by the d'_p values indicated for each of the demand spectra. As can be seen, because the base isolation bearings introduce significant reductions in overall structural stiffness, it takes greater displacements to achieve the same effective damping. However, because the isolated system is capable of safety accommodating far greater displacements than the fixed base structure, it is also capable of mobilizing much larger effective damping once it is isolated.

The performance point for the base isolated structure, as shown in Figure 6-8, occurs at a spectral displacement of approximately 14 inches. Of this 14-inch displacement, approximately 12 inches, the distance along the displacement axis of the plot between the performance point and the corresponding point on the fixed-base curve, is accommodated by the isolator. The structure itself displaces only the residual amount, approximately two inches. It should be noted that all of these displacements are in spectral coordinates. In order to determine the actual displacements, it would be necessary to transform the isolated capacity spectrum back into a capacity curve (base shear vs. roof displacement coordinates). Chapter 8 provides guidance on the procedure for doing this.

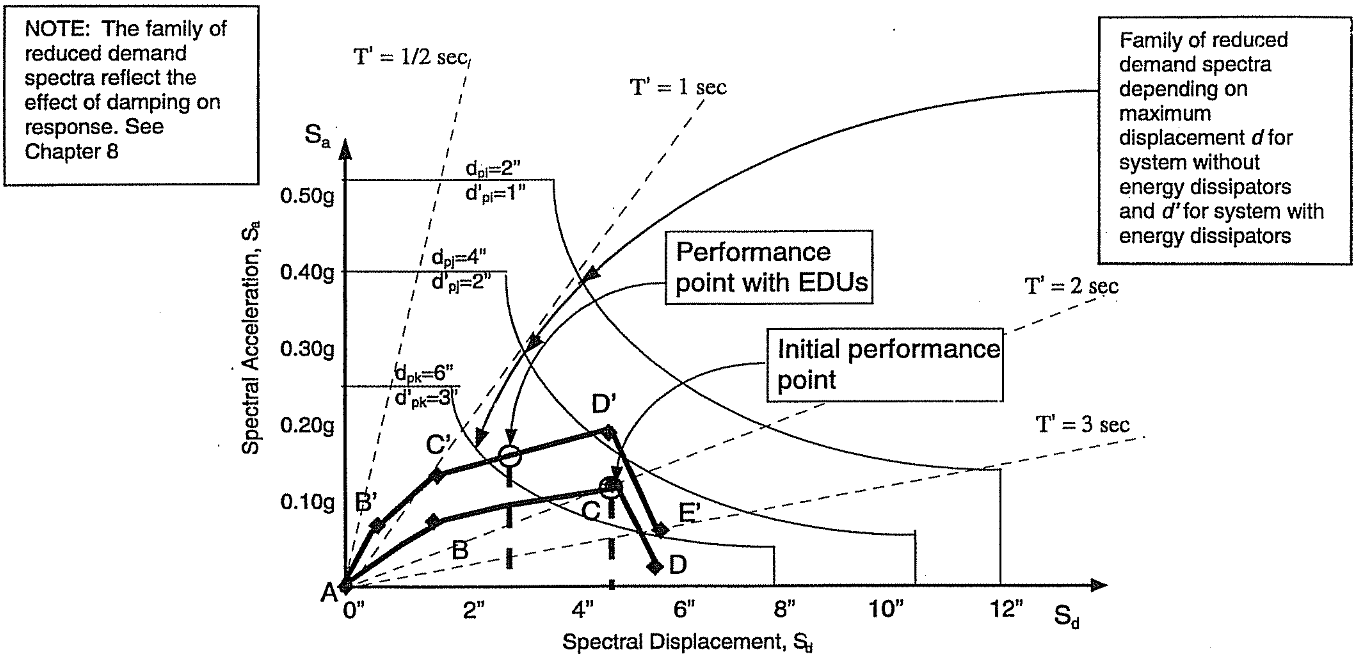
Commentary: *For base isolation to be effective, most of the displacement induced into the isolated structural system must occur within the isolators. In order for this to occur, the structure above the isolation system must have an effective stiffness that is significantly in excess of that of the isolation bearings. The effective stiffness of the superstructure is a function of both its elastic stiffness and the amount of inelastic behavior it exhibits under the residual demands transmitted by the isolators. Base isolation works best for structures that have an initial, unmodified elastic stiffness of 1 second or less. Further, isolation is most effective if the superstructure can remain nearly elastic under the residual demands delivered by the isolators. In order to achieve these two conditions, the installation of a base isolation retrofit frequently also requires stiffening and strengthening of the structure as well.*

Base isolation has most commonly been used in the past as a method of retrofitting historic structures. Initially the decision to use base isolation for this purpose is often based on a belief that the upgrade can be accomplished without performing substantial modifications to the superstructure, thus sparing the important historic fabric of the building from damage during the construction period. Often, however, substantial modification to the superstructure has been necessary in order to obtain the required stiffness and strength above the isolators. When this occurs, it has resulted in some very large project costs as well as a loss of the presumed benefits for historic preservation.

Base isolation may be most effective as a retrofit system when applied in buildings for which there are enhanced performance objectives. The significant reduction in displacement response and accelerations that occur within the superstructure of an isolated building results in much better performance of equipment, systems, and other nonstructural elements than is attainable with most other retrofit systems.

6.2.1.4.2 Energy Dissipation Systems. Energy dissipation systems directly increase the ability of the structure to dampen earthquake response in a benign manner, through either viscous or hysteretic damping. This approach requires the installation of energy dissipation units (EDUs) within the lateral force resisting system. The EDUs dissipate energy and in the process reduce the displacement demands on the structure. The installation of EDUs often requires the installation of vertical braced frames to serve as a mounting platform for the units and therefore, typically results in a simultaneous increase in system stiffness. Energy dissipation systems typically have a greater cost than conventional systems for stiffening and strengthening a building but have the potential to provide enhanced performance.

Figure 6-9 illustrates the effect of energy dissipation on the capacity and demand curves for a retrofitted structure. Curve A-B-C-D is the capacity spectrum for the structure without energy dissipation units. A performance point occurs for this unretrofitted structure at a spectral displacement of approximately 5 inches resulting



NOTE: The family of reduced demand spectra reflect the effect of damping on response. See Chapter 8

Family of reduced demand spectra depending on maximum displacement *d* for system without energy dissipators and *d'* for system with energy dissipators

Figure 6-9. Effect of Enhanced Damping on Building Performance

in a Structural Stability structural performance level. Curve A-B'-C'-D'-E' is the capacity spectrum for the structure after installation of the EDUs. This curve indicates a structure that has both added stiffness, the initial elastic period has shifted from approximately 1.5 seconds to approximately 0.75 second, and also somewhat greater strength. The most important effect however is on the demand spectra. The efficiency of the EDUs in dissipating energy results in much greater effective damping at any displacement. This is evidenced by the lower d'_p values compared to d_p values for the demand spectra. The result is that the performance point for the retrofitted structure shifts to a spectral displacement that is slightly less than three inches, resulting in attainment of the Life Safety structural performance level.

Commentary: *Energy dissipation systems are most effective when installed in structures that have a significant lateral deformation capacity. The amount of energy dissipated by these systems is directly proportional to the force developed by the individual EDUs and the displacement across the EDUs. If a building is relatively rigid, the energy dissipation system will not be able to effectively dampen its response before damage has occurred. Therefore, these systems are most applicable to frame structures.*

An important aspect of buildings retrofitted with energy dissipation systems is that the damping of building response provided by these systems greatly reduces the amount of motion delivered to building contents. As a result, the use of energy dissipation systems should be considered for buildings for which the protection of critical systems or contents is important.

6.2.1.4.3 Mass Reduction. The performance of some buildings can be greatly improved by reducing the building mass. Building mass reductions reduce the building's natural period, the amount of inertial force that develops during its response, and the total displacement demand on the structure. Mass can be reduced by removing heavy nonstructural elements, such as cladding, water tanks, and storage. In the extreme, mass reduction can be attained by removing one or more building stories.

Commentary: *Although mass reduction can be a very effective method of improving a building's seismic performance, it is also a relatively radical technique and consequently is seldom actually implemented. In most cases, the reductions in building mass that can be achieved by removing contents are quite limited in comparison to the overall weight of the structure. The removal of building stories can result in more significant reductions in weight but has obvious detrimental impacts with regard to the amount of floor space available.*

6.2.2 Management Strategies

Management strategies are programmatic in nature and are typically controlled by the building owner rather than the design team. Management strategies tend to be of two types: strategies that affect the acceptability of the building's probable performance and strategies that regulate the way in which a technical strategy is implemented. They include such approaches as occupancy change, demolition, temporary retrofit, phased retrofit, retrofit while occupied, retrofit while vacant, exterior retrofit, and interior retrofit.

Commentary: *While the engineer typically does not have the latitude to select from among management strategies, these are an important consideration in the way a seismic risk reduction project is executed and should be considered by the engineer and discussed with the client. Often, the client for a retrofit project, being unfamiliar with these issues, may not be aware of some of the alternative strategies that are available.*

6.2.2.1 Occupancy Change

Some buildings with inadequate performance capability for the current occupancy may be an acceptable seismic risk if assigned other occupancies. The best risk reduction approach for such buildings may simply be to alter the use of the building. For example, a building capable of meeting the Substantial Life Safety performance level, but not the Immediate Occupancy level would not be an acceptable risk for an acute care facility at a hospital. It might be very adequate, however, for use as a day care center or for medical offices. An appropriate strategy for such a situation may be to use the existing building for one of these latter occupancies and construct a new acute care facility. The desirability of this approach would obviously depend on a number of factors, including a need for the building in the alternative use, the availability of funding to construct a replacement facility, and the availability of land.

6.2.2.2 Demolition

In some cases, the cost of improving a building's seismic performance to the desired level may exceed its economic value. In others, the required structural modifications may render the building undesirable for its intended occupancy. The best approach to improving the seismic risk of such buildings may be demolition. As an example, consider the case of a large warehouse with many large truck loading doors. The doors may render the shear walls incapable of meeting the Life Safety structural performance level. Retrofitting would require a strengthening strategy, consisting either of infilling selected truck doors or placing new braced frames or shear walls within the warehouse space. The building owner might determine that the reduced truck access space or loss of interior space would make the building unrentable as a warehouse. This may trigger a decision to demolish the structure and replace it with a new building.

Commentary: *The decision to demolish rather than seismically retrofit a building is often the result of a cost-benefit study. While nearly all buildings can be successfully retrofitted to provide acceptable performance, the cost of performing such work may be prohibitively high and could approach or even exceed the cost of constructing a replacement facility. Unless a building is an important landmark or contains functions that cannot be taken out of service or relocated, it* *seldom makes sense to invest greater resources in the seismic retrofitting of a building than would be required to replace it.*

6.2.2.3 Temporary Retrofit

In some rare cases it may be desirable to retrofit a building in a highly economical manner for continued short-term service. The technical approach used to retrofit the building under this strategy may be quite unacceptable for normal applications and may include extensive use of exposed structural elements, shoring, bracing, and the like. This approach would make it possible to provide economical protection of the building occupants and contents while plans are developed and financing is obtained for complete building replacement or, possibly, facility phase-out.

6.2.2.4 Phased Retrofit

Building owners may elect to implement a retrofit in phases for a number of reasons, such as an inability to obtain funding for a complete retrofit, a wish not to disturb certain tenants or functions within the building, or a desire to perform the retrofit work concurrently with tenant improvements in various areas of the building. When a phased retrofit is selected because of economic constraints, it is usually desirable to obtain a complete incremental improvement in the building's probable performance with each phase of the work. As an example, consider a frame building with a weak first story as well as inadequate capacity in the stories above. The weak first story could result in collapse in relatively moderate earthquakes, while the deficient upper stories would be judged capable of resisting collapse except in very large earthquakes. A phased approach that might be desirable for this structure would be to strengthen the weak first story in one phase and address the balance of the building deficiencies in later stages.

When performing a phased retrofit, it is important to ensure that the work installed in any particular phase does not unintentionally create a serious seismic deficiency or make an existing one more severe. As an extreme example, consider that an owner wishes to strengthen a frame building in phases, to coincide with the turnover of tenants in leased spaces. It would not be appropriate to add shear walls or braced frames in an upper story before performing the work in a lower story, as this would create a soft/weak story condition. Similarly, it would not be desirable to install the retrofit measures on only one side of the building, as this could create a torsional irregularity. Care should be taken to ensure that at the completion of each stage, the building's probable performance is at least as good as it was prior to the performance of the work.

When performing phased retrofit, it is desirable to perform the design work for all phases concurrently. This ensures that when taken together, the various phases will result in a complete and integrated structure with the desired seismic performance characteristics.

Commentary: *There is not unanimous agreement that when phased retrofit work is performed on a building, no phase should make the building more vulnerable than it was prior to the initiation of the work. Some believe that as long as the increase in risk caused by phased construction is temporary, the short-term additional risk is acceptable. The problem with such an approach is that with phased construction projects, there is always a risk that the project will be terminated before all phases are completed. This could occur as a result of a number of events: change of building ownership, change of occupancy, economic limitations, and similar issues. Thus, a phase of construction that was intended to result in a short-term condition of reduced building capacity and increased risk could result in a permanent high-risk condition.*

6.2.2.5 Retrofit During Occupancy

One of the largest costs of a retrofit project may be the loss of use of building floor space during the construction phase of the project. When a building is vacated to allow retrofit work to be performed, the tenants must find alternative space and bear the moving costs. Upon completion of the work, the owner must find new tenants to occupy the space, which may take many months. Consequently, many owners will desire to perform retrofit work while the building remains occupied. Although it is often possible to do this, a number of challenges are posed to the design team, contractor, and tenants. Often in these situations work must be performed in phases to allow temporary relocation of tenants. Work involving excessive noise or disruption of building utilities usually must be performed during evening or weekend shifts. Precautions must be taken to ensure the security of the construction site as well as the safety of the building tenants. The cost of construction in an occupied building may be as much as 50 percent more than the same work in a vacant building. In addition, construction schedules can be substantially lengthened when the work is performed in occupied spaces.

Commentary: *Prior to making a decision to perform a seismic retrofit project while a building remains occupied, a cost-benefit analysis should be performed to determine the advisability of adopting this approach. Often it may be found that the apparent savings to the tenants and building owner are outweighed by the added cost and schedule of construction and the inconvenience to building operations during the work. Even when a building remains occupied during construction, it is usually necessary to temporarily relocate occupants away from areas of direct construction. This results in cost as well as inefficiency for the occupants.*

6.2.2.6 Retrofit of Vacant Building

This is the typical approach to seismic retrofitting and results in the lowest direct costs of construction as well as the most rapid project execution schedules. As noted in Section 6.2.2.4, however, many building owners will desire not to take this approach. In some cases, particularly when extensive work must be done on the building, this is the only practical approach.

6.2.2.7 Exterior Retrofit

When an owner elects to retrofit a building while it remains occupied, it is often beneficial to perform as much work as possible at the exterior of the building so as to minimize interruption of internal functions and inconvenience to tenants. Technical strategies that can be implemented in this manner include building stiffening and strengthening and demand reduction. Stiffening and strengthening can be accomplished through the addition of exterior buttresses and/or shear walls and braced frames aligned with the existing perimeter wall lines. Demand reduction strategies can be implemented from outside the building by placing damping devices within exterior braced frames or between exterior buttresses and the building. In at least one case, a mid-rise building was base isolated while it remained occupied, by using an "exterior" strategy. In this case, the base isolation system was installed within the building's first story, which was essentially a lobby and was not part of the normally occupied building space. Work on the upper stories was done on the exterior surface of the perimeter walls.

It should be noted that most exterior retrofit projects involve some interior work. Required interior work may include the addition of diaphragm drags and collectors to deliver seismic forces to the new exterior elements and the bracing and anchoring of internal nonstructural components.

Commentary: *Obviously, exterior retrofit approaches are not generally applicable to historically significant buildings.*

6.2.2.8 Interior Retrofit

An important consideration in the retrofit of many buildings is the preservation of exterior appearance. Except in a few cases, the braced frames, shear walls, and buttresses commonly employed in retrofitting are viewed as an unacceptable modification of a building's character. Even the simple wall anchors used to tie exterior walls to diaphragms can be viewed by some as an unacceptable alteration of building appearance. These concerns are particularly important for historic structures and architectural landmarks.

In such cases, the owner may direct that all (or nearly all) retrofit work be performed from inside the building. In historic buildings, interior spaces and fabric may be as significant as exterior features. Cases of this type usually require that work be performed in spaces normally hidden from public view or that new vertical elements of the lateral force resisting system are placed in the same location as historic elements and are finished to have the same appearance as the original historic materials. As an example, an existing hollow clay tile wall could be replaced with a new reinforced concrete shear wall with plaster finishes matching those of the original construction.

6.3 Design Constraints and Considerations

The selection of an appropriate retrofit strategy requires evaluation of the important design constraints. These typically include code requirements, performance objectives, available budget and schedule, aesthetics, construction period occupancy disruptions, and permanent impacts on occupancy and function. In addition, project risk can be an important design constraint in the selection of an appropriate design alternative.

6.3.1 Code Requirements

The legal constraints on a retrofit project should be established prior to embarking on the development of particular retrofit strategies. Code requirements include both structural and nonstructural considerations. The primary structural code considerations include limitations on the design criteria employed and potential restrictions with regard to partial or phased retrofits. Some municipalities require that when substantive work is done on a building, the entire building be upgraded to provide lateral force resistance equivalent to that specified by current code for new buildings or to some fraction of that resistance. In such cases the viability of phased or partial retrofit strategies may be affected.

Nonstructural code considerations can have a significant impact on project cost. Seismic retrofit work commonly triggers collateral upgrades for disabled access and fire life safety systems. Often these collateral upgrades are limited to those spaces in which actual structural retrofit work is performed and along the "path of travel" to the work area. As an example, if retrofit work is performed in the second story of a two-story building, collateral upgrades may have to be performed in the actual work areas as well as at the building entrance, along corridors, in elevators, and in stairways leading to the second-floor work areas, since these are within the path of travel. Retrofit projects in which work is performed on relatively limited locations in a building will generally trigger fewer collateral upgrades than projects that entail work throughout the structure.

Commentary: *Qualified historic structures may not be subject to the same code restrictions as other buildings. In California, the modification, alteration, and repair of historic structures is governed by the state's Historic Building Code (SHBCB 1993). This code encourages alternative solutions that would not normally be permitted in most other buildings.*

Another important consideration is the presence of hazardous materials. Worker safety laws and environmental protection regulations result in increasing restrictions on the way retrofit projects are performed. Work involving removal or disturbance of asbestos-containing materials and lead-based paint can result in significant project cost impacts, as can excavation for foundations on a site with subsoils contaminated by materials deemed to be hazardous. The selection of strategies that minimize the potential for such impacts can be beneficial. An appropriate level of hazardous materials investigation during the project development phase can help to identify these potential impacts. For many buildings there are likely to be existing survey reports on asbestos-containing materials. Reports on lead-based paint are less likely to exist. If a building has a substantial quantity of structural steel, the presence of lead-based paint should be suspected and an evaluation obtained. Typically, if soil borings are obtained in order to develop foundation design recommendations, the geotechnical engineer will also evaluate soil samples for the presence of hazardous materials.

6.3.2 Performance Objectives

The project performance objectives are perhaps the most important design constraint. Performance objectives can range from avoidance of collapse under a specified design earthquake, to protection of building contents, to rapid postearthquake recovery of function. A given retrofit strategy may be appropriate to some objectives but not to others. Strategies that involve strengthening and stiffening of a structure are highly effective at achieving Structural Stability and Substantial Life Safety performance objectives. However, such strategies may not be appropriate to the protection of building contents. In fact, stiffening and strengthening a building will often deliver more-severe forces to the building's contents, potentially resulting in damage. A demand reduction strategy, such as the introduction of a base isolation or energy dissipation system is often more appropriate for obtaining performance objectives involving protection of contents or immediate postearthquake occupancy.

6.3.3 Project Budget

Cost is often the overriding factor in determining the project performance objectives, the retrofit strategy employed, and even whether a retrofit will be performed. Different strategies can have widely different costs. When evaluating the costs related to a particular strategy, it is important to include all cost components. These include design fees, construction costs, operating and maintenance costs, tenant relocation costs, costs relating to the loss of use of floor space both during and after construction, and the owner's project management and supervision costs. Generalizations with regard to the likely costs of different strategies are difficult to make, as the cost of a particular strategy is directly dependent on the characteristics of the individual building and the project performance objectives. Projects that call for Immediate Occupancy or Damage Control will, however, be significantly more costly on the average than projects with lower performance objectives. Although costs are extremely important to the selection of a strategy, they are also very difficult to assess without a well-defined design plan. It is usually necessary to develop a schematic-level design for each strategy considered in order to assess the probable cost impacts with any reliability.

6.3.4 Aesthetics

Aesthetics are often an important factor in selecting a retrofit strategy. Retrofit elements placed at the exterior of a building, including infill walls, new walls, buttresses, and braced frames, are typically perceived as having a negative impact on building appearance. To the extent that these elements are viewed as degrading the building's appearance, they detract from its value. In some cases, architectural redesign of the building exterior can mask the new structural elements or incorporate them into the building's design. The cost of such remodeling is typically high, however.

Aesthetic impacts must also be considered within the interior spaces of buildings. Retrofit measures that result in decreased ceiling heights or narrow corridors are often viewed as undesirable, even if the spaces are within the legally specified requirements. In many occupancies, exposed structural elements are perceived as undesirable aesthetically unless special precautions are made to provide a "finished" look to the elements. As an example, braced frames that incorporate bolted connections will often be viewed negatively by building occupants. Similar bracing with welded connections that have been ground smooth may provide an acceptable appearance, however.

The issue of aesthetics is most important in historic landmark structures. The retrofit strategies selected for these buildings should result in minimal impact on the building configuration or appearance and, to the extent possible, minimal alteration of the actual materials and fabric that make up the building. Base isolation may often be an appropriate strategy for such structures. Although base isolation is highly disruptive of a building's base story, using this approach may make it possible to minimize the work that must be performed at levels above the base, where important historic features may be situated. Another technique that can be useful in historic buildings is to place new shear walls behind existing walls, using the existing wall as a veneer.

6.3.5 Construction Period Occupancy Disruption

The ability of tenants to continue to occupy a building during retrofit construction can have a significant benefit with regard to overall project cost. This can make strategies that permit such construction period occupancy more attractive. As previously stated, strategies that make it possible to perform retrofit work from the building exterior are the most conducive to continued occupancy during the construction period. But, even strategies that entail substantial work within the building interior can be implemented in partially occupied buildings. If it is possible to temporarily relocate a portion of a building's occupancy during the construction period, then it is possible to rotate other occupants from their normal locations to the empty space, giving access to discrete areas of the building for construction operations.

Most retrofit work entails a substantial amount of noise and dust, which the occupants of a building must be prepared to live with if they are to remain in residence during the construction period. The most noisy retrofit construction activities include selective demolition by means of chip hammers and roto-hammer drilling for the placement of dowels and anchors. It is advisable to perform such operations in off-hours, such as evenings and weekends, to minimize the disruption to building occupants. Although such off-hour construction activities increase direct labor costs, they may actually result in a decrease in total project costs through avoided costs related to tenant relocation.

Other considerations related to work on occupied buildings include the need to keep utilities in service and the need to ensure a secure work area and maintain a safe working environment for the occupants. Construction sites are potentially dangerous, and precautions must be taken to ensure that building occupants do not have free access to areas where work is actively being performed. In addition, protection of occupant safety requires that construction operations not block exits or corridors. Additional care must also be taken to contain potentially hazardous materials used or exposed in the construction process, since building occupants will not in general be wearing protective clothing.

6.3.6 Permanent Occupancy Impacts

Many retrofit strategies will result in some permanent impairment of the future occupancy and use of the building. As an example, the installation of vertical braced frames or shear walls within the interior of a building will limit future traffic patterns within the building as well as limit the possibility for placing partitions in certain areas. The placement of frames or walls at the perimeter of a building may reduce the amount of natural light available and make office space less desirable.

Seismic retrofitting can also confer an occupancy benefit. A building that has been retrofitted to provide immediate postearthquake occupancy should have significantly more value to tenants than a building which must be closed for repairs following an earthquake. The extent of this value will be related to the cost of finding replacement space and the cost of relocating equipment and contents within the building.

6.3.7 Project Risk

Typically the selection of a retrofit strategy and system occurs early in the design process and must be based on only very limited study of the various options. Consequently, at the time the evaluation of the alternatives is made there are likely to be a number of poorly defined factors that could affect the cost and even the feasibility of one or more of the design alternatives evaluated. These factors could include such things as unknown materials strengths, undefined foundation conditions, incomplete structural analyses, and grossly estimated element sizes. The level of risk associated with these undefined design conditions should be evaluated independently for each strategy and system. In general it will not be the same for all of the alternatives.

Another source of project risk that should be considered is construction risk. Although seismic retrofit design projects typically include some exploration of the existing condition and configuration of a building, it is not usually feasible to verify all conditions that will be encountered during construction. One of the largest sources of project delays and cost overruns on retrofit construction projects is the discovery of unexpected conditions when selective demolition is performed during construction. Retrofit systems that require frequent attachment to the existing structure generally have a greater risk in this regard.

Commentary: *The risk associated with discovering unanticipated conditions during the construction phase can not be overemphasized. The unanticipated conditions can include such things as framing that does not conform to original construction documents, materials that are substantially different than expected, and similar features that could affect the feasibility of constructing the design as intended. In order to minimize such risks, it is extremely important that there be an appropriate program of investigation of the building during the design phase.*

There are several ways to include potential project risk in an evaluation of alternative retrofit approaches. One way is to include project risk as an actual design constraint, evaluating it along with the other design constraints when selecting a strategy. A more common method of treating project risk in the evaluation is to assign a unique cost contingency allowance, based on the perceived uncertainty, to the estimated project cost. A system that seems to have a high risk could be assigned a contingency allowance on the order of 30 percent or more, while approaches that appear to be relatively free of risk could be assigned lower contingencies, perhaps on the order of 10 percent.

6.4 Strategy Selection

In order to select a retrofit strategy, it is necessary to establish the basic performance objectives desired for the building and the existing deficiencies relative to those performance objective. Once these have been determined, it is possible to evaluate each of the various strategies to determine whether they are technically and practically capable of mitigating the identified deficiencies. If the engineer has an adequate understanding of the various design constraints before starting this process, it may often be possible to eliminate many of the available strategies without detailed study. It is important, therefore, to meet with the building owner prior to starting this process to define these design constraints and their relative importance. Table 6-1 is a checklist that may be useful in obtaining an understanding of these issues.

Table 6-1 lists several of the more important design constraints, discussed previously in Section 6.3. Each of these constraints will apply in some degree to every project. Their relative importance on a given project will not be the same, however, and it will vary from one project to another. It is important to understand the relative importance of each for the specific project being considered. Project cost limitations should be established as early as possible. Unless a study of retrofit alternatives has been performed, the

Table 6-1. Checklist of Retrofit Design Considerations

Constraint	Importance (1-10)	Limitation
Performance objective	____	Structural Level ___ for ___ % in 50 years Nonstructural Level ___for ___ % in 50 years
Project cost	____	Constr: $________ Other: $________
Project schedule	____	________ months
Construction occupancy	____	___ Building vacant ___ Partial occupancy ___ Full occupancy
Hazardous materials	____	___ none known ___ known present ___ Intended remediation
Building appearance	____	___ May be altered ___ Must be preserved
Floor space impact	____	___ No obstruction ___ Obstruction allowed

building owner will typically have no idea as to how much the project could or should cost. However, most owners will be able to determine a maximum cost beyond which the project would not proceed. An understanding of this limit will assist in eliminating some strategies from consideration.

Construction schedule may be nearly as important as cost. Often retrofit projects are conducted simultaneously with other improvements to a building, such as modifications to accommodate a new tenant. Some strategies may require a significantly longer design and construction duration than others and may therefore be eliminated quickly through consideration of this constraint.

Building appearance is nearly always important. However, alteration of a building's appearance does not always result in a less attractive building. If there are specific reasons for preserving the appearance of a building (for example, if the structure is a landmark), these should be understood before a strategy is selected.

Many buildings are constructed for a given occupancy and use and are expected to remain in that use throughout their lives. As an example, a classroom building on a college campus can be expected to house individual classrooms throughout its life. In such a building, the replacement of non-structural partitions with structural walls would probably be acceptable. In an office building however, it is highly desirable

Table 6-2. Sample Strategy Evaluation Matrix

	Desirability						
Strategy	*Cost*	*Schedule*	*Architectural Impact*	*Occupancy Disruption*	*Seismic Performance*	*Total Score*	*Comments*
Importance (1-10; 10 very important)	10	3	7	3	10		
System completion	0	0	0	0	0	0	Not viable
System strengthening and stiffening	9	10	7	10	6	259	
Exterior shear walls	9	10	7	10	6	228	
Exterior braced frames	10	9	3	10	5	228	
Interior shear walls	7	8	10	0	6	224	
Interior braced frames	8	7	10	0	5	221	
Exterior buttresses	9	10	3	10	6	231	
Demand reduction							
Base isolation	3	5	10	0	10	215	
Enhanced damping	0	0	0	0	0	0	Not viable
Mass reduction	0	0	0	0	0	0	Not viable

to retain flexibility in the layout of offices to accommodate the needs of different tenants over the years. The installation of permanent walls within such a structure would be less desirable.

Once the deficiencies and design constraints are properly understood, the evaluation of alternative strategies can begin. In general, it will be necessary to generate a design for each viable strategy to at least a schematic level so that the alternative strategies can be compared. Capacity and demand spectra should be drawn for the structure in the unmodified state in order to determine the types and extent of retrofitting required, as described in Section 6.2 and below.

The viability of and approximate requirements for various strategies can be judged directly from the demand-capacity plots, as described in the Section on Preliminary Design. Once a schematic design has been developed, cost estimates can be generated and the impacts on building appearance, and function judged.

For each strategy investigated and found to be viable, an evaluation should be made of the extent to which the strategy can meet the design constraints. This evaluation should include development of preliminary cost estimates, project schedules, schematic floor plans, and elevations for the retrofitted building and any other materials that may be helpful to the building owner and occupants in understanding the potential impacts of the proposed work. A matrix format similar to that presented in Table 6-2 may be useful in evaluating the relative merits of each strategy and in selecting a final strategy for development of the design.

In the matrix shown in Table 6-2, alternative strategies for retrofit of a particular building have been evaluated. A relative weight, presumably based on discussions with the owner, has been assigned to each of the design constraints, based presumably on discussions with the Owner. Then for each strategy that is viable, an evaluation rating is assigned for each design constraints. Ratings range from 1 to 10, with 10 representing little impact or the most desirable effect and 1 representing the least desirable impacts. A weighted score is then obtained for each strategy, consisting of the sum of the individual evaluation scores factored by the weight for each design constraint. In this particular case, a system strengthening strategy consisting of the installation of exterior shear walls was rated the most favorably. For other projects, however, each design constraint would be assigned a different weight and individual strategies would have different scores.

Commentary: *The specific design constraints that are contained in the evaluation matrix will be unique to each project, as will the relative weights that are assigned to them. The specific evaluation criteria used, and their relative assigned weights, should be based on discussions with the client agency, the building occupants, and other stakeholders.*

6.5 Preliminary Design

Once a retrofit strategy has been selected, it is necessary to develop a preliminary retrofit design. A preliminary design should include the approximate sizes and preferred locations for all major elements of the retrofit strategy including braced frames, shear walls, buttresses, base isolators, and energy dissipation units. The preliminary design may be used for several purposes including coordination between the various design disciplines, for example architectural, mechanical, and electrical; development of preliminary cost estimates; and, development of the strategy evaluation matrices (described in the previous Section). The preliminary design also provides a basis upon which to perform a structural verification analysis used to ensure that the design approach is capable of meeting the project performance objective and for determining forces used in the final detailing of the retrofit structural elements.

The first step in the preliminary design process is to prepare and overlay demand and capacity spectra for the unretrofitted structure, using the procedures described in Chapters 4 and 8. The performance point for the unretrofitted structure should be determined and the behavior of the structure at this performance point understood. If the structure is incapable of meeting the desired performance objectives at the performance point, than the specific deficiencies should be identified. Structural performance is most closely tied to the lateral deflection induced in the building by the earthquake ground motions. If the performance predicted for the unretrofitted structure is unacceptable, the capacity curve for the structure should be evaluated in order to determine a target lateral displacement at which acceptable performance can be attained. Data required for use in performing a preliminary design of retrofit elements can then be obtained by adjusting the capacity and demand spectra in an appropriate manner relative to the retrofit strategy, as explained in previous Sections of this Chapter, and by then obtaining data on the required supplemental strength, stiffness, period or effective damping from the adjusted curves. The following sections describe the application of this technique to specific retrofit strategies.

6.5.1 Stiffening and Strengthening

Stiffening and strengthening of a structure will typically be accomplished by the addition of braced frames, shear walls, buttresses and similar elements that add both strength and stiffness to the structure. The effect of this will be to shift the performance point for the structure to a lower spectral displacement (or lateral deflection). In order to perform a preliminary design for this strategy, it is necessary to decide what target spectral displacement is desired for the retrofitted structure, and then, to determine the approximate additional strength and stiffness that will shift the performance point to this spectral displacement.

Figure 6-10 illustrates the way this information may be obtained for a representative structure. In the figure, the capacity spectrum for the unretrofitted structure, shown as a bold curve, indicates an initial period for the structure of approximately 1.5 seconds and an ultimate spectral acceleration capacity of approximately 0.1g. The first step is to determine what spectral displacement would result in acceptable performance

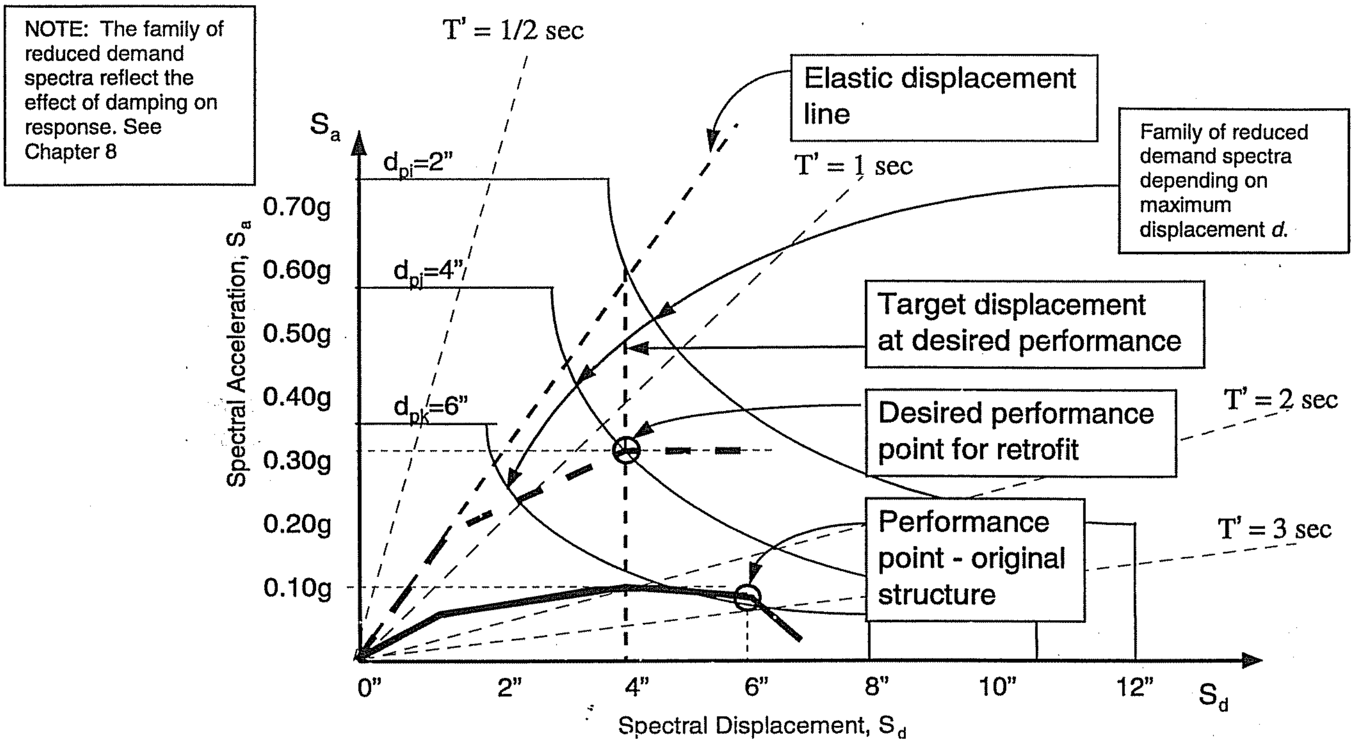

NOTE: The family of reduced demand spectra reflect the effect of damping on response. See Chapter 8

Figure 6-10. Preliminary Calculation for Retrofit Using Strengthening and Stiffening

for the retrofitted structure. For this example, assume that examination of the damage state that exists at each of the various points on the capacity curve indicates that acceptable performance can be obtained at a spectral displacement of 4 inches. This is illustrated in the figure by the vertical dashed line drawn at a spectral displacement of 4 inches and labeled as the target displacement for the desired performance. Note that this target displacement will be different for every structure and every structural performance level. The displacement of 4 inches used in this example has simply been selected for illustrative purposes only. Similarly, the ultimate shear capacity for the structure of 0.1g has been arbitrarily selected for the purposes of this example. The actual ultimate shear capacity for a given structure should be determined directly from the capacity curve for that structure.

For the purpose of developing a preliminary design for a strengthening and stiffening strategy it is appropriate to make several approximate simplifying assumptions. The first of these assumptions is that the demand spectra for the structure will not be significantly affected by the retrofit and that therefore, the same demand spectra used in finding the initial performance point can be used on a preliminary basis to solve for the retrofitted performance point. In reality, strengthening and stiffening the structure will result in somewhat altered demand spectra. However, this assumption will typically lead to a conservative solution for the preliminary design. Therefore, the approximate solution for the retrofitted performance point is obtained by extending the vertical line at the target spectral displacement, 4 inches in the case of the example illustrated by the figure, until it intersects the demand curve calculated using a value of d_p equal to this target displacement. This point is annotated on the figure as the "desired performance point." A horizontal line extending from the desired performance point to the y axis indicates the minimum spectral acceleration capacity desired for the retrofitted structure. With this information known, the required ultimate base shear capacity for the retrofitted structure can be obtained from the following equation:

$$V_{required} = \frac{S'_{au}}{S_{au}} V_u \qquad (6\text{-}1)$$

where $V_{required}$ is the desired ultimate shear capacity for the retrofitted structure, S'_{au} is the spectral acceleration at the desired performance point, S_{au} is the ultimate spectral acceleration for the original, unretrofitted structure and V_u is the ultimate base shear capacity for the original structure. In the case of the example illustrated in figure, S'_{au} is 0.3g and S_{au} is 0.1g, indicating that the retrofitted structure should have a lateral shear strength that is three times (0.3g/0.1g) times that of the original structure, or, that the retrofit elements should have twice the lateral shear strength of the existing structure.

The next step is to determine an appropriate initial stiffness for the retrofitted structure. As an approximation, an estimate of the initial period required for the retrofitted structure can be obtained by extending the vertical line through the desired performance point until it intersects with the elastic response spectrum (demand spectrum for 5 percent viscous damping). In the example shown in the figure, this demand curve is the one labeled as having a d_p value of two inches. A radial line, drawn from the origin of the demand/capacity spectrum plot through the intersection of the vertical target displacement line with the elastic response spectrum defines the desired initial stiffness for the retrofitted structure, expressed as a period with units of seconds. This period can be calculated from the equation:

$$T' = 0.32\sqrt{\frac{S_{d5\%}}{S_{a5\%}}} \qquad (6\text{-}2)$$

where *T'* is the target initial period for the retrofitted structure, $S_{d5\%}$ is the target displacement and, $S_{a5\%}$ is the spectral acceleration corresponding to the intersection of the target displacement line with the elastic response spectrum, expressed in units of the acceleration due to gravity, g. For the case illustrated in the figure, $S_{d5\%}$ is 4 inches, $S_{a5\%}$ is 0.6g and *T'* is calculated as 0.8 seconds.

The target stiffness for the retrofitted structure can than be calculated from the equation:

$$K_r = K_i\left(\frac{T_i}{T'}\right)^2 \qquad (6\text{-}3)$$

where K_r is the stiffness required of the retrofitted structure, K_i is the initial stiffness of the unretrofitted structure and T_i and *T'* are respectively the initial periods for the unretrofitted structure and the retrofitted structure.

Once the required stiffness and strength for the retrofit elements has been determined, as indicated above, it is possible to develop preliminary sizes for shear walls, braced frames or other elements to provide these properties. While this approach is suitably accurate to lead to a preliminary design solution, it is extremely important that the actual demand and capacity spectra for the retrofitted structure be formally computed as part of the final design process. A number of simplifying assumptions have been made in the formulation of the above equations and approach and designs derived by the method described may either be excessively conservative or inadequate for individual structures. Design verification, through development of the revised demand/capacity spectra is an essential part of the final design process.

6.5.2 Base Isolation

Most base isolated buildings have a period that ranges between two to three seconds and effective viscous damping ratios that range from fifteen to twenty-five percent. Consequently, in order to

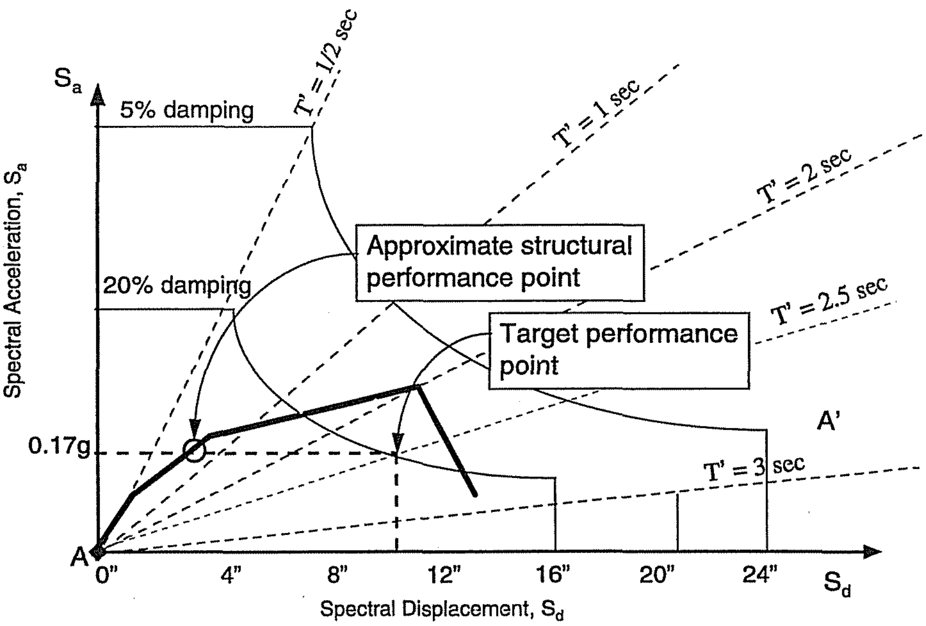


Figure 6-11. Approximate Solution for Base Isolation Preliminary Design

perform a preliminary design for base isolation it is convenient to assume that the base isolated structure will have an effective period of 2-1/2 seconds and an effective damping of twenty percent. By overlaying the capacity spectrum developed for the unretrofitted structure with an appropriate demand curve for 20 percent effective viscous damping, it is possible to evaluate the feasibility and requirements for a base isolated design. This is illustrated in Figure 6-11.

As shown in the figure, the first step is to overlay the capacity spectrum for the unretrofitted structure with the 20 percent damped demand spectrum. The 20 percent damped demand spectrum may be derived by using values of *SRA* and *SRV* of 0.55 and 0.65, respectively, in the Chapter 8 procedures for developing families of demand spectra. The next step is to draw the radial line representing a period of 2-1/2 seconds. This line will have a slope equal to 0.0164 g/in. The intersection of the 2-1/2 second period line with the 20 percent damped spectrum solves for the target performance point for the preliminary design of the base isolated structure. In the case of the example shown in Figure 6-11, this is represented by a spectral acceleration of 0.17g and a spectral displacement of approximately 10.4 inches.

Once the target performance point is known, the structural performance point, for the superstructure should be solved for. This is obtained by drawing a horizontal line from the y axis through the target performance point. The intersection of this line with the capacity curve for the structure indicates the structural performance point for the isolated structure. This will either

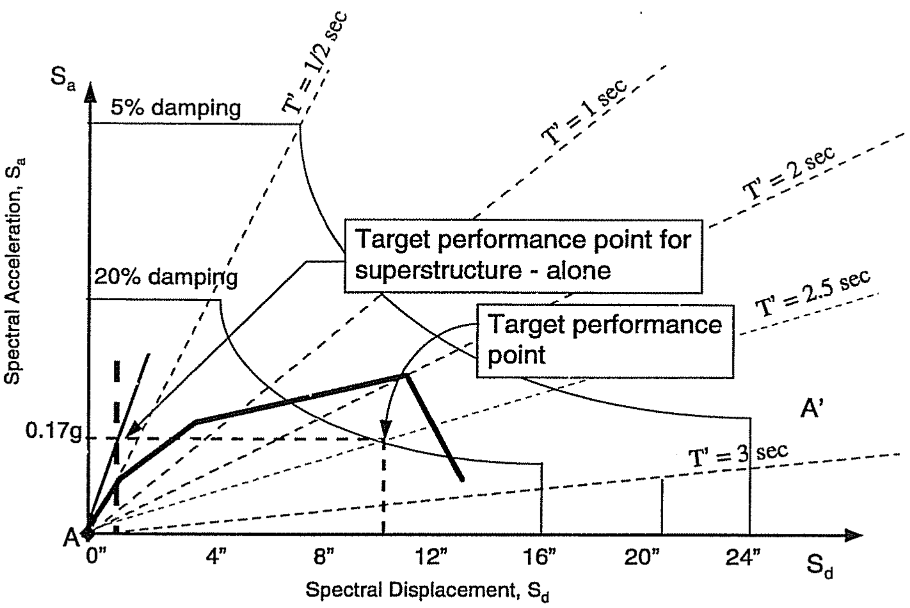


Figure 6-12. Preliminary Determination of Strengthening and Stiffening, Base Isolated Structure

occur at an acceptable performance level, for the project performance objectives, or not. If it does not represent an acceptable performance level then this indicates that for base isolation to be an appropriate strategy, it is necessary to perform supplemental stiffening and strengthening of the structure in addition to base isolation.

In order to determine the amount of stiffening and strengthening required, it is necessary to examine the capacity curve for the unretrofitted structure to determine the maximum spectral displacement associated with an acceptable structural performance. Similar to the procedure indicated in Section 6.5.1, a vertical line should be drawn through this new target displacement for the superstructure. The intersection of the vertical line drawn through the target displacement with the horizontal line drawn through the intersection of the 2-1/2 second period line and the 20 percent damped response spectrum defines the required stiffness and strength of the retrofitted structure if base isolation is to achieve the project performance objectives. Figure 6-12 illustrates this approach.

In Figure 6-12, it has been assumed that the base isolated structure will not be permitted to experience any yielding under the project performance objectives. Consequently, a vertical line has been drawn through the spectral displacement at which this occurs, in this case approximately one inch. The intersection of this vertical line with the horizontal line drawn at 0.17g represents the target performance point for the retrofittted superstructure, independent of the base isolation system. With this information the required supplemental stiffness and strength of the retrofit elements can be determined as previously indicated in Section 6.5.1. The radial line drawn from the origin of the figure to the target performance point for the superstructure alone provides the period T' for the retrofitted structure for use in the formulas of Section 6.5.1. In those formulas, the term $S_{a20\%}$ should be substituted for the term $S_{a5\%}$. The value for $S_{a20\%}$ is simply given by the intersection of the horizontal line through this target performance point and the vertical axis, in this case 0.17g. The term $S_{d20\%}$ should be substituted for the term $S_{d5\%}$. The $S_{d20\%}$ value is obtained as the intersection of the vertical line through this target performance point and the horizontal axis, in this case one inch.

Preliminary design for the base isolators themselves should be done with the assistance of one or more of the several suppliers of these devices who have design tools available to assist in the selection of isolators. The design displacement for the isolators is obtained as the difference in spectral displacements at the target performance point and the structural performance point respectively, converted back to actual displacement (rather than spectral displacement) coordinates using the procedures of Chapter 8. Base isolated structures behave as almost ideal single degree of freedom systems. Consequently, the modal participation factor and modal mass contribution for base isolated structures is very nearly 1. Therefore, for a first order approximation, the spectral displacements from the demand/capacity spectra plots may be taken as equal to the actual displacements, without further conversion. In figure 6-11, the isolation system displacement would be calculated as a spectral displacements of 10.4 inches - 3 inches, or approximately 7.4 inches. If the superstructure is strengthened and stiffened, as indicated in Figure 6-12, then this would be obtained as the difference in spectral displacements between the target performance point and target performance point for the superstructure alone. In the case of the structure represented by figure 6-12, this would be 10.4 inches - 1 inch, or approximately 9.4 inches. This data will be required by the base isolation bearing supplier to perform a preliminary size of the bearings, as well as the estimated weight on each bearing and the estimated design base shear in the structure. The estimated design base shear is obtained from the equation:

$$V = S_{a20\%} W \qquad (6\text{-}4)$$

where W is the effective seismic weight of the structure as defined elsewhere in this methodology.

In addition to determining the size of the base isolators, it is necessary to select a location for the plane of isolation in the structure. This consists of a horizontal plane, cut though the structure, in which the isolation bearings are placed. The isolation system displacement occurs across this plane. It is typically necessary to provide horizontal diaphragms above and below this isolation plane. The diaphragm above the isolation plane must distribute the lateral forces from the superstructure to the individual isolator bearings, in proportion to their relative stiffnesses. Note that it is likely that several different size isolators will be recommended by the isolation system vendor, depending on the dead and live column loads that are supported, each having somewhat different stiffness. The diaphragm below the isolation plane must be capable of distributing the lateral loads from the individual isolators to the foundation system for the structure. If each individual isolator is provided with a foundation capable of resisting the forces from the isolator, this diaphragm may not be required.

Note that the procedures indicated above are not adequate for a final design either of the superstructure retrofit or of the isolation system. The procedures contained in Chapter 8, supplemented as appropriate by the procedures of the building code should be used to perform the final design of base isolation retrofits.

6.5.3 Energy Dissipation

Most retrofits employing energy dissipation units will have an effective viscous damping ranging between 20 percent and 40 percent. In order to determine if a retrofit employing EDUs is appropriate, the first step is to prepare a plot of the capacity curve for the unretrofitted structure, overlaid with damped spectra for 5 percent, 10 percent, 20 percent, 30 percent and 40 percent viscous damping. Such a curve is shown in Figure 6-13. The intersection of the capacity curve with each of these spectra indicates the performance point that would be obtained if

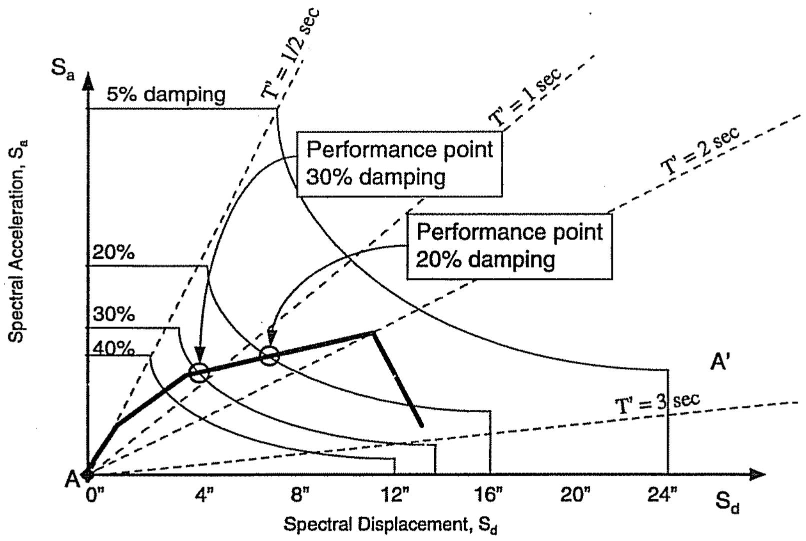


Figure 6-13. Preliminary Design of Retrofit With Energy Dissipation Units

EDUs capable of providing the indicated damping were to be installed.

The demand spectra for the different effective viscous damping ratios can be obtained using the procedures of Chapter 8 and values for the coefficients SR_A and SR_V obtained from Table 6-3.

Table 6-3. Values of the Coefficients SR_A *and* SR_V

Effective Viscous Damping β	SR_A	SR_V
5%	1.0	1.0
10%	0.77	0.82
20%	0.55	0.65
30%	0.42	0.55
40%	0.33	0.48

Once the capacity spectra and various damped demand spectra have been drawn, the performance points for each effective damping should be examined to determine if the indicated damping would result in acceptable performance in accordance with project performance objectives. If performance for a given level of effective damping is determined to be effective, it is necessary to select the particular EDUs that will be used, on a preliminary basis, and the method of installation of these units in the structure.

There are a number of different EDU systems available in the market, each of which have significantly different force-displacement and force-velocity relationships. Consequently, there are no readily available rules of thumb that can be used to select a generic EDU for the basis of preliminary design. Preliminary selection of EDUs should be done with the assistance of a supplier of these devices. The supplier will need to know the characteristics of the structure, including its mass and existing stiffness; the effective damping desired, represented by the coefficient β, determined from the demand/capacity spectrum plot and the displacement or interstory drift in which this effective damping must be developed. With this information, the vendor will be able to make a preliminary determination of the required damper size and also provide an estimate of the forces that will develop in the EDUs. These forces must be developed into the structure. EDUs are typically installed in structures as part of lateral

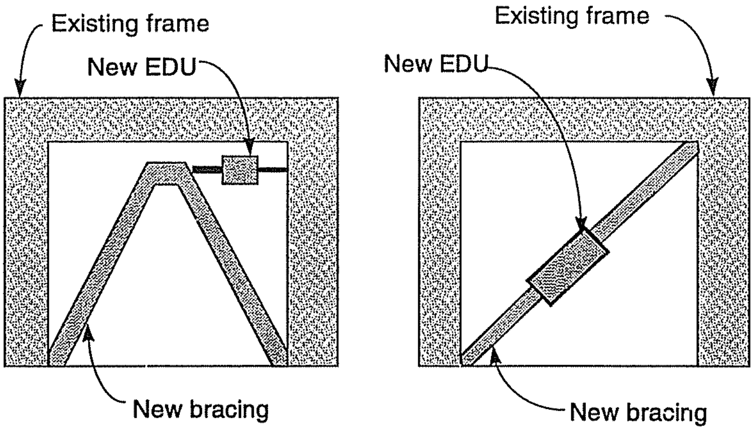


Figure 6-14. Typical Installation of Energy Dissipation Units

bracing systems. Figure 6-14 indicates two ways in which such dampers can be installed. Once the forces in the dampers have been estimated by the damper supplier, it is possible to use these forces directly in the preliminary design of the bracing elements as well as to check the adequacy of the existing structure and foundations to resist these forces.

As with other retrofit strategies, final design of retrofits employing EDUs will require that the capacity and demand spectra be redeveloped using the procedures of Chapter 8 and that the performance point be evaluated for adequacy. It may not be possible to characterize the behavior of some types of EDUs using the capacity spectrum approach. For such systems, it will be necessary to perform a nonlinear response history analysis.

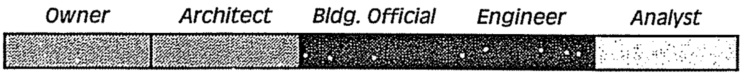
6.5.4 Other Strategies

Preliminary designs for retrofits employing strategies other than stiffening and strengthening, base isolation and energy dissipation must be performed using a trial and error process. In this process, a retrofit design is assumed, the resulting capacity and demand spectra developed and the performance point determined. If the assumed design results in an acceptable performance point for the project performance objectives, then the design is adequate. If not, then the design must be revised and the process repeated or an alternative strategy employed.

Chapter 7

Quality Assurance Procedures

Audience Interest Spectrum



7.1 General

To ensure the appropriate application of the methodology during the evaluation, design and construction phases, a quality assurance program should be required. Since performance-based design and the methodologies presented in this document are relatively new and may be somewhat unfamiliar to the broad engineering community, procedures for assuring the appropriate criteria and application of the methodologies are critical. Even though the procedures in this document do not explicitly extend into the construction phase, the importance of construction quality on building performance in general and the likelihood of encountering unforeseen conditions in retrofit construction in particular warrant special attention to construction monitoring and quality assurance.

This chapter presents the major features of such a program consisting of the following three processes: peer review, plan check, and construction quality assurance. Peer review provides an independent second opinion regarding the building evaluation and retrofit design criteria, retrofit strategies, and issues involving engineering judgment. Plan check provides a more detailed review of the construction documents for conformance to the established criteria and strategies. Construction quality assurance provides a measure of confidence that the retrofit design intent is properly implemented during construction and includes site observations on the part of the design team, a testing and inspection program, and a contractor's quality control plan.

Comprehensive programs for each of these processes are presented; however, it is likely that the circumstances surrounding a particular building project will require varying levels of the scope outlined below. The owner should require some form of a quality assurance program on all projects, and this program should be developed and initialized during the preliminary evaluation/design phase of the project. It is the responsibility of owner and/or building official, in consultation with the design team, to ensure that the program implemented satisfies the needs of the particular project.

Though the extent and formality of the quality assurance program should be tailored to the particular project, such a program should encompass the major phases of the project and address, within the above framework, the following issues:

- The adequacy of the information generated by the field investigations (see Chapter 5)
- The adequacy and appropriate application of the analytical methods used to identify and quantify the building response and vulnerabilities
- The validity of the proposed retrofit concepts
- The completeness and accuracy of the design calculations

- The adequacy and proper execution of the testing and inspection program during construction
- The conformance of construction materials and execution with design requirements

7.2 Peer Review

Because of the complexity and uncertainty inherently involved in the seismic evaluation and retrofit of concrete buildings, an independent second opinion in the form of a project peer review can enhance the quality and reliability of the design. Moreover, since the procedures and methodologies outlined in this document require the thoughtful exercise of engineering judgment supported by relatively new and unfamiliar and sometimes complex analysis procedures, a project peer review should be implemented in order to assure the owner that these procedures and methodologies have been appropriately applied and followed.

Commentary: *Though peer reviews have been performed for some time, formalized peer review standards and requirements are less established and, in some cases, still in development. This section should serve as a guide towards establishing a project peer review process for a specific building project. There are several standards that are currently in use for peer review including ASCE's* Standard for Independent Peer Review *(ASCE 1995),* Recommended Guidelines for the Practice of Structural Engineering in California *(SEAOC 1995a), and* Guidelines for the Seismic Retrofit of State Buildings *(DSA 1995).*

When project peer review is implemented, the owner, or in some cases the building official, will contract directly with the project peer review team. The structural engineer of record will be responsible for meeting with the team at regular intervals, providing them with information (which may include a preliminary investigation plan, a proposed analysis methodology, conceptual design schemes, progress sets of design documents and calculations, and a testing and inspection plan), and responding to the team's review comments. The services associated with peer review are additional items of scope which should be reflected in the engineering contract.

7.2.1 Purpose

The purpose of the project peer review is to improve public safety and to enhance the quality, reliability, and performance of the retrofitted building. The project peer review team fulfills this purpose by confirming the overall criteria, concepts, strategies, and execution of concepts; providing a second opinion on issues involving engineering judgment; and providing added assurance that new and/or complex analysis methodologies or retrofit strategies are applied appropriately. The owner, with input from the design team, will determine appropriate objectives for the project peer review.

Commentary: *Performance based design and the analysis techniques presented in this document are relatively new and advanced methodologies. There will be a certain amount of time before the engineering community becomes familiar and comfortable with these techniques, and there will be an associated learning curve leading to the effective use of them. Additionally there are often differences of opinion on the applicability and use of the methodologies for a specific building project. It is for these reasons that a project peer review should be viewed as not only beneficial but essential when these methodologies are employed.*

7.2.2 Objectives

Among the objectives that may be appropriate for a given project are the following:

- To stimulate thought and discussion of a collaborative nature by examining the basis for the evaluations and judgments made and offering alternative interpretations and solutions for consideration by the design team
- To provide confirmation of the appropriateness of performance objectives, basic assumptions, and design criteria

- To review the proposed analysis methodology for appropriateness
- To provide independent validation of engineering judgments that contribute to the evaluation, retrofit design, and assessment of seismic performance
- To provide review and comment regarding the reliability, constructibility, and economy of the retrofit design solution
- To track the progress of the evaluation and design process to ensure a comprehensive vulnerability assessment and retrofit solution
- To review the final design documents to ensure that the concepts developed during evaluation and preliminary design are implemented in the actual retrofit
- To provide review and comment regarding the performance of non-structural components

Commentary: *In most cases the following three functions are provided solely by the structural engineer of record. They are included here for the rare instances when the complexity of the construction process or unforeseen conditions might warrant the objective oversight function as an added quality assurance measure.*

- *To provide review and comment regarding special construction such as shoring or special sequences that may be required*
- *To provide review and comment regarding the completeness and appropriateness of the testing and inspection plan*
- *To monitor the progress of the construction process to ensure that the quality assurance plan is implemented*

The peer review process is not intended to be a value engineering exercise. However, the process often yields a similar economic benefit simply by providing the added assurance that all the deficiencies are discovered and corrected. Furthermore, any suggestions regarding economy and constructibility of the retrofit that result from the peer review are both desirable and appropriate.

Peer review is not intended to be a check for code compliance, although some code issues will be addressed in the process of defining design criteria and objectives. Code compliance is addressed more explicitly in the plan check process.

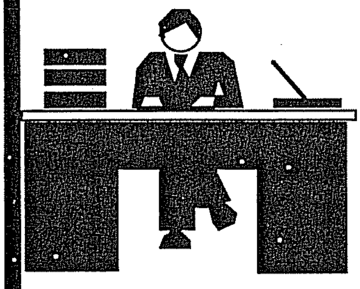
7.2.3 Requirements

Project peer review is recommended on all projects unless the owner (and in some cases the building official), in consultation with the structural engineer of record, determines that the project qualifies as a "simple" project. In order for the project to qualify as simple, the following conditions should be present:

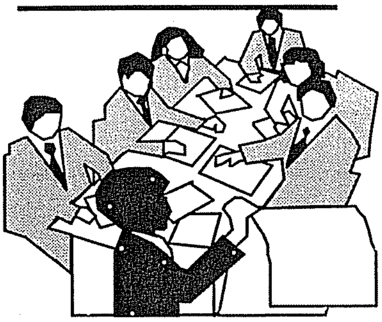
- The building is one or two stories.
- Complete as-built drawings are available.
- Observed existing conditions substantially conform to as-built drawings.
- Any existing structural irregularities do not specifically affect the building's dynamic response.
- Conventional (linear elastic) analysis procedures are used.

Commentary: *The above list is by no means comprehensive. Other considerations include the size of the building (relatively small), the regularity and redundancy of the lateral force resisting system, the occupancy (no special functions or contents), and the performance required (no special damage control or lower seismic hazard). The judgment as to whether eliminating the peer review is warranted must be made by the owner or building official with the input of the design team.*

Plan Check



Peer Review



Drawings/calcs submitted; reviewed by individual; written comments returned	**Basic Process**	Panel or individual tracks progress of engineer- of -record
Familiarity with code requirements	**Qualifications of Reviewer(s)**	Equal to engineer-of-record
Not required	**Strategy for Evaluation and Retrofit**	Should be reviewed
Not reviewed	**Conceptual Design**	Consensus required
Detailed review	**Construction Documents**	Check for conformance with concept
Primary focus	**Code Compliance**	Secondary focus
Not explicitly considered	**Seismic Performance**	Primary focus
Not considered	**Construction Cost Control**	Potential forum for discussion
Included in permit fees	**Cost of Review**	Additional but normally reasonable
Building official/permit requirements prevail	**Conflict Resolution**	consensus desirable/ owner option

7.2.4 Qualifications of Peer Review Team

The project peer review team should consist of one or more engineers whose collective experience spans the technical issues anticipated in the design project, especially a strong background in performance-based design and the nonlinear static analysis procedures presented in this document. In order to be effective, the review team should possess qualifications at least equal to those of the design team in analysis methods, a thorough understanding of materials and system behavior, and familiarity with construction means and methods. The lead peer review engineer should be a California registered structural engineer, familiar with the regulations governing the retrofit work being reviewed, and experienced in the seismic retrofit of similar concrete structures.

The project peer review team must not have involvement in the project other than the tasks associated with the peer review.

Commentary: *One possible exception to this prohibition exists. In the case where the owner is not subject to plan check review by an independent agency, the owner may also retain the project peer review team (or a member of the team) to perform the detailed plan check. However, there is not universal consensus on this exception: the alternative viewpoint that is sometimes encountered is that the project peer review team, having been intimately involved in the concept and strategy development, would be in the position of reviewing its own work when performing plan check services.*

It is essential that there be good communication between the structural engineer of record and the project peer review team and that the review team be objective, unbiased, and fair.

Commentary: *It is desirable that the review team and the structural engineer of record have mutual respect.*

7.2.5 Schedule for Peer Review

The project peer review team should be selected, and the peer review initiated, early in the evaluation or preliminary analysis and design phase and should serve through the review of essentially complete (90-100 percent) construction documents. Although covering most of the project design schedule, the peer review process will usually be completed as a series of discrete reviews at defined project milestones, such as the following:

- Conceptual phase—focusing on evaluation criteria and performance design basis
- Preliminary evaluation and conceptual design phase—focusing on results of preliminary investigation, identification of deficiencies, assessment of expected performance, decision to retrofit, and proposed detailed analysis
- Design development phase—focusing on analysis and preliminary retrofit design strategies and reconfirmation of criteria and performance basis.
- Construction document phase—focusing on the design of systems used to implement the retrofit strategy and conformance to established design and performance criteria

7.2.6 Scope

The scope of the peer review must be sufficient to serve the purposes and desired objectives described above. The owner (and in some cases the building official), in consultation with the structural engineer of record, should determine an appropriate scope for the project peer review. The level of peer review effort required will vary depending on the individual project; and may range from no review to a comprehensive independent check similar to that required for the design of new hospitals. Factors influencing the scope of the peer review include the complexity of the proposed evaluation procedures and retrofit schemes and the capability and experience of the design team. The scope should be sufficiently broad to enable the project peer review team to gain a thorough understanding of the project constraints and context.

The following list contains possible scope items. It is intended to be comprehensive and to encompass a wide range of items that could be included in the scope of the peer review depending on the requirements of any specific project.

Commentary: *This list may be far too comprehensive for all items to be required or included for most individual retrofit projects. However, they are included as a menu to assist the owner and design team in determining the appropriate scope for a project based on the specific conditions and desired level of assurance for that project. The published peer review guidelines referenced above contain addition information and specific scope items that may be useful in determining the appropriate scope for peer review.*

- Review quality assurance program
- Review as-built information and the results of the preliminary investigation
- Review preliminary evaluation and analysis approach and results
- Review strategy for final analysis and design at the end of the design development phase
- Review the design work plan to ensure that all pertinent elements of the evaluation and analysis procedures are addressed:
  - Performance and seismic hazard criteria
  - Assigned capacity of structural systems and elements and load paths
  - Degree of redundancy, ductility, and compatibility
  - Acceptance limits
  - Nonstructural systems
  - Special conditions including base isolation and damping systems and devices
- Monitor design progress, review changes, verify execution of design work plan for potential effects on achievements of desired performance objectives
- Review the contract documents at 90 percent completion for consistency, completeness, conformance with design work plan
- Review the proposed construction phase testing and inspection plan developed by the structural engineer of record
- Review plan check scope, meet with the structural engineer of record and the plan check engineering team to review and discuss the plan check methodology and to assist in identifying conditions that warrant special attention.

Commentary: *In order to get the maximum benefit out of the plan check process, it is desirable for the structural engineer of record (and as deemed appropriate the peer reviewer) to provide the plan checker with an introduction to the project including the identification of primary objectives, criteria, and assumptions, complex portions of the design and/or construction, and critical elements of the load path.*

In most cases the peer review services will terminate with the review of the 90 percent to 100 percent construction documents. However, it may be desirable to consider some involvement of the project peer review team beyond that point in cases where there will be issues of construction sequence and/or means and methods that should be defined during design or where it is anticipated that significant engineering decisions will have to made after destructive testing and/or partial demolition. An example would be temporary shoring and bracing requirements where the retrofit strategy involves temporarily interrupting load paths to facilitate construction.

Such scope items could include: reviewing of construction plan and assisting in identifying conditions that require special construction procedures or sequences, reviewing additional information discovered during construction, and reviewing changes as a result of hidden conditions or constructibility constraints.

These items are not intended to address the general construction plan, but only should be *considered in situations which deviate from methods and sequences of standard construction. In these situations it is often prudent to have a second opinion, and the owner may wish to include additional items of scope in the peer review.*

Items of scope selected from the above list or other items of scope, as deemed appropriate by the structural engineer of record and the owner and/or building official, should be identified in the written agreement for peer review services.

7.2.7 Reporting

Specific form and timing of report(s) are to be determined by mutual agreement in coordination with the design team, the project peer review team, and the owner. It may be desirable for the review team to produce progress reports at major milestones in the design process.

Each report should include the following:

- Scope of review
- Statement of status of project documents at each review stage
- Statement of findings, comments, and any recommendations
- Statement of items requiring further review

A comprehensive report summarizing findings, comments, and recommendations should be produced after the review of the 90 percent complete construction documents. The 90 percent report should address explicitly each of the items that have been included in the scope of the peer review. The report should distinguish between comments that involve life safety issues, comments that have to do with constructibility or economy, and comments that have to do with the clarity of the information presented in the documents.

7.2.8 Conflict Resolution and Responsibility

The structural engineer of record should respond explicitly to each comment or recommendation made by the project peer review team. Where there are differences of opinion between the structural engineer of record and the review team, they should attempt to resolve the differences through direct interaction. The structural engineer of record has the ultimate responsibility for the structural design, and, therefore, must have the control of the design solution. However, differences of opinion between the project peer review team and the structural engineer of record should be resolved to the satisfaction of the owner and/or building official. All issues raised by the review team should be addressed by the structural engineer of record and, if possible, resolved by consensus. However, the structural engineer of record should not be required to defer to the judgment of the review team and is not obligated to incorporate all of their recommendations.

Commentary: *One means to provide for conflict resolution is to incorporate provisions for a peer review board, separate from the project peer review team, that can fill the role of arbiter of disputes. Such a resource provides an ideal forum in which to resolve professional differences of opinion. Unfortunately, such a review board may not be available to the general building design and construction community.*

In the case of a total impasse, the owner and/or building official must either accept the recommendation of the structural engineer of record or replace him/her. It should be emphasized that such a step may have grave implications for the project schedule and budget. Under no circumstances should the project peer review team be retained to replace the structural engineer of record.

7.2.9 Peer Review Agreement

There should be a written agreement between the owner and the project peer review team defining the terms and conditions under which the peer review will be performed. Such an agreement should address the following:

- The specific items of scope to be addressed by the peer review
- The schedule for the peer review

- The issue of liability

Commentary: Typically, peer review guidelines (ASCE 1995, SEAOC 1995a) recommend that the agreement include an indemnity clause in recognition of the fact that the reviewer does not have control of the design. The intent of the peer review process is to enhance the quality—not distribute the liability.

- The process for implementing the peer review including lines of communication
- The methods of reporting and the format of reports
- The process of reconciliation of differing opinions between the project peer review team and the structural engineer of record
- Compensation
- Methods of addressing additional items of scope that may become necessary or desirable as the design progresses

7.3 Plan Check

While the peer review process provides the owner with quality assurances related to criteria, evaluation, the analysis and design process, and achievement of desired performance, the plan check is typically performed under the auspices of the building official and consists mainly of a detailed review of the construction documents and calculations for correctness, completeness and compliance with applicable codes, standards, or guidelines. It may be performed by the building official or an independent engineer retained by the building official. The plan check should generally not be performed by the same team that performed the project peer review.

***Commentary:** In most cases the plan check is performed by agencies that are independent of the owner, e.g., building departments or the OSHPD. Where this is not the case, i.e., where the owner, agency, or institution is not subject to plan check review by building officials of an independent agency, the owner may wish to consider having the peer reviewer continue in the role of plan checker, since having in-depth knowledge of the structural system is an advantage in performing the plan check.*

7.3.1 Purpose

The plan check process provides the building official (and the general public) a measure of confidence in the implementation and compliance of the retrofit design, as represented by the drawings, specifications, and calculations with applicable codes, standards, or guidelines. Although the plan checker may identify conceptual analysis and design issues, this is not the intent and should not be a common occurrence.

The purposes of the plan check are as follows:

- To ensure that the set of calculations prepared by the structural engineer of record is comprehensive and addresses all systems, subsystems, elements, and details required as part of the retrofit
- To provide a degree of confidence that the calculations performed are substantially accurate
- To provide a degree of confidence that the drawings are complete and address all elements deemed pertinent to the performance criteria for the project
- To provide a degree of confidence that the details on the drawings are compatible with the analyses performed and are internally consistent

***Commentary:** Many of the activities in a typical detailed plan check duplicate activities that the structural engineer of record must perform in house. The plan check is not intended to replace the internal quality control procedures of the structural engineer of record.*

7.3.2 Requirements

A plan check should be performed on all projects. The level of the plan check will inevitably vary depending on the requirements of the specific building department.

7.3.3 Qualifications of the Plan Checker

The qualifications of plan checker should be similar to those of the peer reviewer. The plan checker should be familiar with the analysis procedures employed and experienced in the production of contract drawings and specifications for construction projects similar in scope and nature to the project being checked.

Commentary: *The plan check must address the critical issue of communication of the design to the construction team through the drawings and specifications. In order to do this properly the plan check team should have experience in producing contract documents.*

7.3.4 Schedule for Plan Check

The formal detailed plan check should commence when the construction drawings and calculations are no less than 90 percent complete. However, the building official responsible for performing the plan check should be introduced to the project at an earlier phase, i.e. during preliminary design, in order to become familiar with the specifics of the particular project and be able to determine if the assistance of an outside plan checker will be needed.

7.3.5 Scope

As a minimum, the plan check should include the following elements:

- Review of seismic performance, capacity, and demand criteria
- Review of all elements of the lateral system and verification that calculations for each element have been prepared
- Review of all elements of the gravity system affected by the retrofit construction and verification that calculations for each affected element have been performed
- Preparation of independent calculations for a representative sample of each type of element and cross check against the calculations prepared by the structural engineer of record, including:
  - Verification of lateral analysis performed by the structural engineer of record
  - Diaphragm shears and moments (may be based on verified lateral analysis) and verification of collector forces and connection designs
  - Cross check of details against plans including a visual check of local load paths in details and, where appropriate, calculations for local load paths
  - Detailed check of typical connections and verification of applicability of typical details, i.e., verification that typical details cover all conditions intended
  - Detailed check of several of non-typical connection details, beams, columns, braces, foundations
- Review of the construction phase testing and inspection program prepared by the structural engineer of record, including suggestions of modifications deemed appropriate based on the detailed review of the design

7.3.6 Reporting and Correction Process

The plan checker should prepare a written report summarizing the scope of the review, the specific elements checked, and the results of the check. A red-lined check set should accompany the report. The structural engineer of record should respond to all items noted by the plan checker, either by revising the calculations and affected drawings or, where appropriate, clarifying and/or verifying the intent of the original design. Corrections must be resubmitted to the plan checker for back checking.

Commentary: *It is imperative that there be good communication between the structural engineer of record and the plan check team. To the extent possible, questions that are raised during* *the plan check should be discussed prior to red-lining in order to minimize misunderstandings.*

7.3.7 Conflict Resolution and Responsibility

Where possible, differences of opinion between the plan check team and the structural engineer of record should be resolved through direct communication. If direct communication fails to resolve the conflict, the project peer review team should serve as mediator. If the review team is also the plan check team, or if mediation fails to resolve the conflict, then the building official should serve as the mediator and all conflicts must be resolved to his/her satisfaction. Nevertheless, the control of, and the responsibility for, the structural design will remain fully and solely with the structural engineer of record.

Commentary: *The potential for conflicts should be recognized and addressed before the conflicts arise—as they inevitably will. A process for resolving disputes expeditiously will benefit all parties.*

7.4 Construction Quality Assurance

In order to ensure that the constructed project conforms substantially with all design requirements a three-tiered quality assurance program should be utilized during the construction phase of all projects. The three tiers are as follows:

- Design team construction services and site visits
- The owner's testing and inspection program, as outlined in the testing and inspection plan prepared by the structural engineer of record
- The contractor's quality control program

7.4.1 Design Team Construction Services and Site Visits

The primary means of providing construction quality assurance rests with the involvement of the structural engineer of record in the construction progress. Site observations and construction monitoring are always critical to the achievement of a certain level of structural performance, but this is even more the case in concrete retrofit construction. No matter how extensive the evaluation and design phase exploration, there always exists the potential for unforeseen conditions to be discovered during construction, some of which can have significant implications for performance and/or assumptions made during evaluation and design and may not be recognized by the contractor. Additionally, because the intent of certain details may not be clear to the contractor, there exists the potential for misinterpretation, which could lead to seemingly innocuous yet significant deviations. Therefore, for the procedures and methodologies of this document to be effective in providing acceptable building performance, the structural engineer of record must play an active roll in assuring that the retrofitted building conforms to the criteria and intent of the retrofit analysis and design.

A construction quality assurance program should identify the extent to which the structural engineer of record will perform visual observations and should define his/her role in monitoring the performance of the independent testing and inspection agency and the contractor's quality control personnel.

7.4.1.1 Site Visits

The structural engineer of record should make site observations at appropriate stages of construction to verify the conformance of the existing construction with the assumptions of the analysis and retrofit design, and to provide reasonable assurance that the new construction is in general compliance with the design documents. As a minimum, the number and duration of the site visits should be sufficient to allow for observation of the following:

- Foundation reinforcing details
- Hidden conditions that have been exposed for the first time during construction

- Reinforcement placement for elements of the lateral system
- Critical details that will be obscured by subsequent construction
- Other conditions or elements critical to the reliable performance of the retrofitted structure

7.4.1.2 Reporting

The structural engineer of record should provide documentation in the form of written field reports of all site visits, including the items observed or discussed with the construction team and any conditions requiring corrective measures. The contractor's quality control personnel should be responsible for notifying the independent testing and inspection agency when the corrective measures are completed. The agency should be responsible for verifying and reporting the corrective measures.

7.4.2 Construction Testing and Inspection Program

The structural engineer of record should develop scope of a testing and inspection program to be implemented by an independent testing and inspection agency.

7.4.2.1 Inspector Qualifications

The owner should retain an independent testing and inspection agency qualified to perform the testing and special inspection required for the project. As a minimum, the agency should provide inspectors that have been qualified in accordance with ICBO procedures for the type of test or inspection they are performing. The structural engineer of record may require more rigorous qualifications where deemed appropriate based on the need for special or unusual testing procedures.

7.4.2.2 Scope

The program must address the type and frequency of testing required for all construction materials that influence the structural integrity of the retrofit construction. These materials should be tested and/or inspected in accordance with conventional methods. Examples of items for field testing and/or inspection include:

- Reinforcing size, spacing, and cover
- Concrete materials and placement
- Structural steel welding and bolting

The testing and inspection program should also outline a specific testing procedure and frequency for all special conditions including, but not limited to, the following:

- The drilling and grouting of reinforcing steel or bolts
- The installation of expansion bolts
- The installation of isolators or dampers

7.4.2.3 Reporting

The testing and inspection agency should submit formal reports using standard forms to facilitate subsequent retrieval of archived construction information. All reports should be submitted to the owner and the structural engineer of record in a timely fashion to facilitate any corrective measures that are required.

7.4.2.4 Coordination

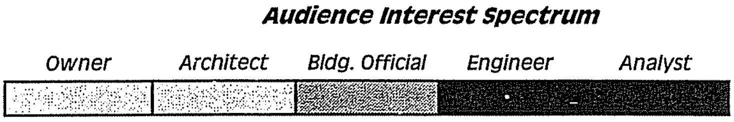
The procedures contained in the testing and inspection program should include provisions for coordinating with the contractor's quality control program. For example, where the contractor's quality control program requires the use of pour checklists, the independent testing and inspection agency should collect and maintain record copies of the checklists.

7.4.3 Contractor's Quality Control Program

The contractor should be required to institute and follow formal quality control monitoring and reporting procedures independent of the owner's testing and inspection program. The contractor should identify individuals who are responsible for ensuring quality control on the job site. There should be an established formal means of monitoring and verifying the contractor's quality control procedures such as concrete pour sign-off sheets and checklists. (Such monitoring may be part of the scope of work of the owner's independent testing and inspection agency.) The contractor's quality control personnel should be responsible for coordinating with the owner's independent testing and inspection agency to assure that testing and inspections proceed in a timely manner in accordance with the testing and inspection program.

Chapter 8

Nonlinear Static Analysis Procedures



8.1 Introduction

This chapter presents analytical procedures for evaluating the performance of existing buildings and verifying the design of seismic retrofits. The organization of the chapter is as follows:

Section 8.1	Introduction
Section 8.2	Methods to perform simplified nonlinear analysis
Section 8.2.1	Step-by-step procedures to determine capacity (pushover)
Section 8.2.2	Step-by-step procedures to determine demand (displacement)
Section 8.2.3	Step-by-step procedures for checking performance
Section 8.2.4	Other considerations
Section 8.3	Example of procedures
Section 8.4	Other analysis methods
Section 8.5	Basics of structural dynamics

Various analysis methods, both elastic (linear) and inelastic (nonlinear), are available for the analysis of existing concrete buildings. Elastic analysis methods available 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, which at this time is considered overly complex and impractical for general use. Available 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 (e.g., FEMA-273 (ATC 1996a)) that uses pushover analysis and a modified version of the equal displacement approximation to estimate maximum displacement; and the secant method (e.g., City of Los Angeles, Division 95 (COLA 1995)) that uses a substitute structure and secant stiffnesses.

This document emphasizes the use of nonlinear static procedures in general and focuses on the capacity spectrum method. This method has not been developed in detail previously. It provides a particularly rigorous treatment of the reduction of seismic demand for increasing displacement.

Analytical Procedures

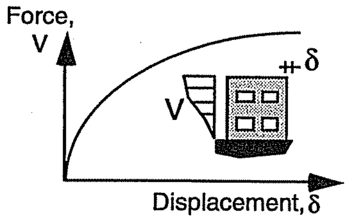
Elastic
(8.4.1)
- Code Procedures
- Demand Capacity Ratios

Simplified Nonlinear
(see Section 8.2)

Other Nonlinear
(8.4.2)
- Secant Method
- Time History

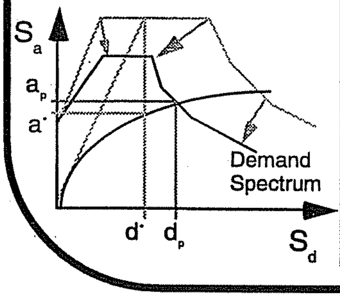
Capacity
(8.2.1)

The central focus of the simplified nonlinear procedure is the generation of the "pushover" or capacity curve. This represents the lateral displacement as a function of the force applied to the structure. This process is independent of the method used to calculate the Demand and provides valuable insight for the engineer.

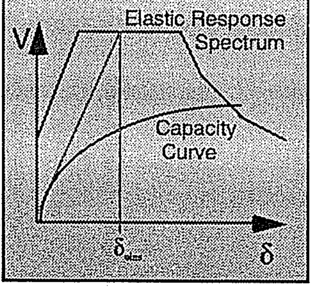


Demand
(8.2.2)

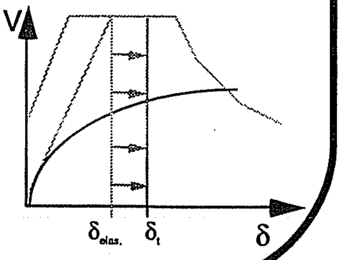
*The **Capacity Spectrum Methods A,B,&C** (8.2.2.1) reduce the elastic spectrum to intersect the Capacity Curve in spectral coordinates to find a Performance Point a_p, d_p. The Equal Displacement Point a^*, d^*, is a good starting point for the iterative process.*



*The **Equal Displacement Approximation** (8.2.2) estimates that the inelastic displacement is the same as that which would occur if the structure remained completely elastic.*

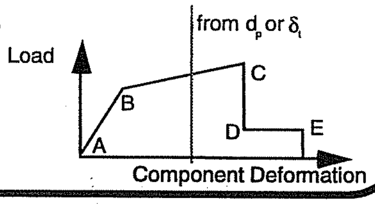


*The **Displacement Coefficient Method** (8.2.2.2) modifies $\delta_{elas.}$ with coefficients to calculate a Target Displacement, δ_t.*



Performance
(8.2.3)

Using the Performance Point or Target Displacement, the global response of the structure (Table 11-2) and individual component deformations (11.4) are compared to limits in light of the specific performance goals for the building.



The displacement coefficient method is an equal alternative and is briefly reviewed. These methods are described in detail in Section 8.2 and an example is given in Section 8.3. Many of the other available methods of analysis are discussed in Section 8.4.

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 mechanisms and account for redistribution of forces during progressive yielding. Inelastic analysis procedures help demonstrate how buildings really work 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 Chapter 6, it provides an immediate and clear picture of how various retrofit strategies, such as adding stiffness or strength, will impact the building's response to earthquake demands.

8.2 Methods to Perform Simplified 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.

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.

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. This process is discussed in more detail in Section 8.2.1. For two dimensional models, computer programs are available that directly model nonlinear behavior and can create a pushover curve directly. The pushover capacity curve approximates how structures behave after exceeding their elastic limit.

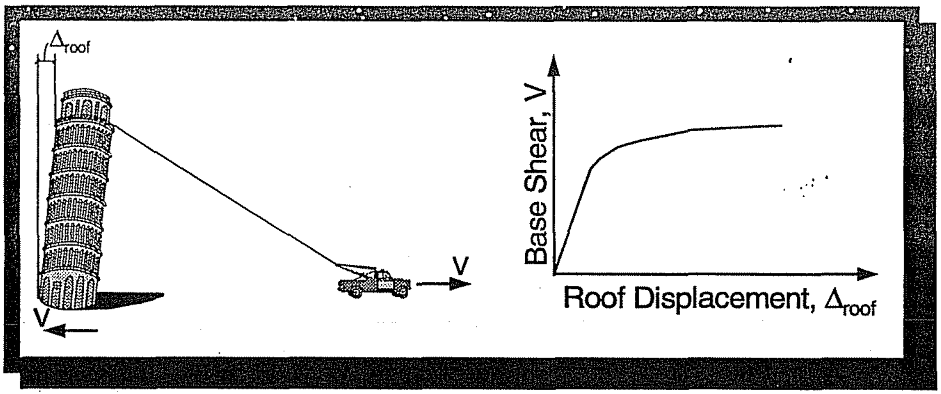
Demand (displacement): Ground motions during an earthquake produce complex horizontal displacement patterns in structures that may vary with time. Tracking this motion at every time-step to determine structural design requirements is judged impractical. Traditional linear analysis methods use lateral forces to represent a design condition. For nonlinear methods it is easier and more direct 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.

Performance: Once a capacity curve and demand displacement are 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.

The next three subsections provide step by step procedures for determining capacity, demand and performance using the capacity spectrum method and the displacement coefficient method. Except for the procedures used to determine the demand displacement, these methods are quite similar.

8.2.1 Step By Step Procedures To Determine Capacity

Structure 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.



Some nonlinear computer programs (e.g., DRAIN-2DX (Powell et. al. 1992)) are able to perform a pushover analysis directly, with no iteration required. The step by step method below does not apply if such programs are used. When a linear computer program (e.g., ETABS (CSI 1995), SAP90 (CSI 1992), RISA (RISA 1993)) is used, the following procedure can be used to construct a pushover curve:

Commentary: 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.

1. Create a computer model of the structure following the modeling rules in Chapter 9, and if the foundation is modeled, following the foundation modeling rules in Chapter 10.
2. Classify each element in the model as either primary or secondary, as defined in Chapter 9.
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.

Commentary: The pushover procedure has been presented in various forms for use in a variety of methodologies (e.g., Seneviratna and Krawinkler 1994, Moehle 1992). As the name implies, it is the process of pushing horizontally, with a prescribed loading pattern, incrementally, until the structure reaches a limit state. There are several levels of sophistication that may be used for the pushover analysis. Five examples are given below. Level 3 is prescribed as the basic method for these guidelines; however, Level 4 may be required for buildings with weak stories and Level 5 may be required for tall buildings or buildings with irregularities that cause significant participation from modes of vibration other than the fundamental mode.

   1. *Simply apply a single concentrated horizontal force at the top of the structure. (Would generally only apply to a one-story building.)*
   2. *Apply lateral forces to each story in proportion to the standard code procedure without the concentrated F_t at the top (i.e., $F_x = [w_x h_x / \Sigma w_x h_x]V$).*
   3. *Apply lateral forces in proportion to the product of story masses and first mode shape of the elastic model of the structure (i.e., $F_x = [w_x \phi_x / \Sigma w_x \phi_x]V$). 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.*
   4. *Same as Level 3 until first yielding. For each increment beyond yielding, adjust the forces to be consistent with the changing deflected shape.*
   5. *Similar to 3 and 4 above, but include the effects of the higher modes of vibration in determining yielding in individual structural elements while plotting the capacity curve for the building in terms of first mode lateral forces and displacements. The higher mode effects may be determined by doing higher mode pushover analyses (i.e., loads may be progressively applied in proportion to a mode shape other than the fundamental mode shape to determine its inelastic behavior.) For the higher modes the structure is being both pushed and pulled concurrently to maintain the mode shape.*

4. Calculate member forces for the required combinations of vertical and lateral load.
5. Adjust the lateral force level so that some element (or group of elements) is stressed to within 10 percent of its member strength.

   Commentary: *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 additional lateral load. For structures 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.

   Commentary: *It is also useful to record member forces and rotations because they will be needed for the performance check.*
7. Revise the model using zero (or very 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.

   Commentary: *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 determine element rotations, it is necessary to add the rotations from 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 effects; distortions considerably beyond the desired performance level; an element (or group of elements) reaching a

lateral deformation level at which significant strength degradation begins, as defined in Section 9.5; or an element (or group of elements) reaching a lateral deformation level at which loss of gravity load carrying capacity occurs, as defined in Section 9.5. See Figure 8-1 for a typical capacity curve.

***Commentary:** Some engineers prefer to continue the construction of the capacity curve beyond the above suggested stopping points to understand the structural behavior assuming that all inadequate elements are retrofitted.*

Exception: In certain cases where elements lose all or a significant portion of their lateral load carrying ability, but could continue to deflect with no other unacceptable affects, continuation of the analysis may be justified. The most notable example of this behavior may be coupling spandrels in shear walls that are not needed for vertical load support. This behavior, which implies considerable redistribution of lateral load, can be explicitly modeled as indicated in step 11. This procedure can also be used to model elements that degrade more gradually, but modeling this behavior requires an estimation of the number of cycles of loading, consideration of the reliability of predicted behavior, and careful review of all aspects of performance of the degrading elements.

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, as indicated in Section 9.5. 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 8-2 illustrates the process, for an example where three different capacity curves are required.

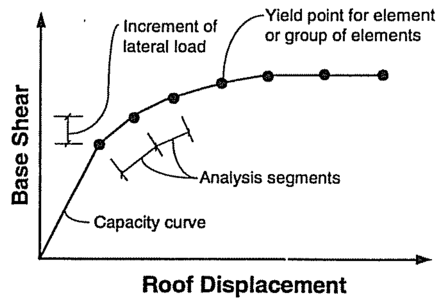


Figure 8-1. Capacity Curve

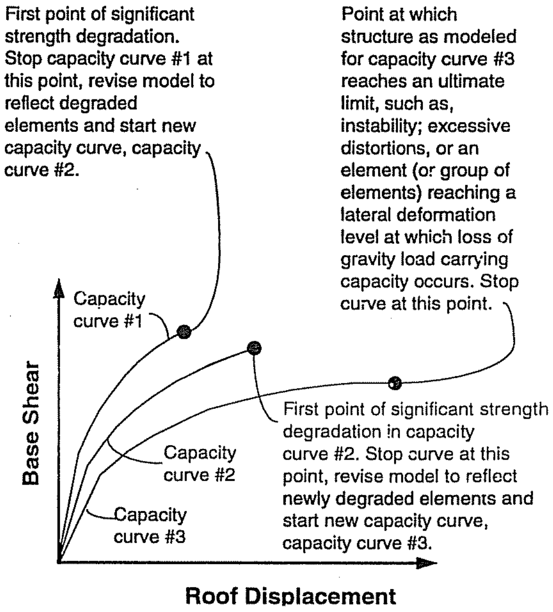


Figure 8-2. Multiple Capacity Curves Required To Model Strength Degradation

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 "sawtooth" shape, as shown in Figure 8-3.

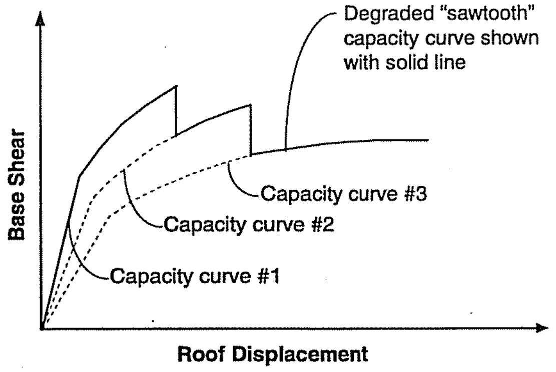


Figure 8-3. Capacity Curve With Global Strength Degradation Modeled

Commentary: 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. In addition, the sensitivity of the estimated demand displacement to the modeling assumptions should be checked by bounding the response with a range of assumptions.

8.2.2 Step By Step Procedures To Determine Demand

***Commentary:** 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 as-is condition or for a retrofit scheme, the probable maximum displacement for the specified ground motion must be estimated. Although a significant amount of effort has been expended in the last few years to develop simplified methods for estimating this displacement (for this project, for FEMA-273 (ATC 1996a), for LA's Division 95 (COLA 1995), for analysis of new isolated and highly damped buildings, and in private universities), no clear consensus has emerged.*

The project team for this document has concentrated on the capacity spectrum method, primarily because the method involves continued and significant use of the capacity curve and because of the potential of the method to assist development of retrofit strategies, as shown in Chapter 6. In addition, recent unpublished studies at the State University of New York, Buffalo (SUNY) indicate that the principles of the capacity spectrum method, when combined with spectral reductions based on estimates of hysteretic damping similar to those used in this chapter, yield displacements generally within 10 percent of the average maximums from multiple time history runs. The method of obtaining a design displacement derived for the FEMA-273 document based on application of coefficients to the elastic displacement is also given as another option. Considering the many possible variations in ground motions, material properties, and mathematical modeling of structural elements, the differences between results from this method and the capacity spectrum method in the majority of cases are insignificant. As in all aspects of structural engineering, particularly when dealing with existing buildings, good engineering judgment must be exercised.

It is expected that in the near future, a consensus will develop on this aspect of simplified nonlinear analysis, which in combination with computerized calculation of the capacity curve, will enable development of far more cost-effective, production-oriented procedures.

In order to determine compliance with a given performance level, a displacement along the capacity curve must be determined that is consistent with the seismic demand. Two methodologies for determining this displacement are presented in this section.

The capacity spectrum method is presented in Section 8.2.2.1. It 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 and terminology with the balance of this document.

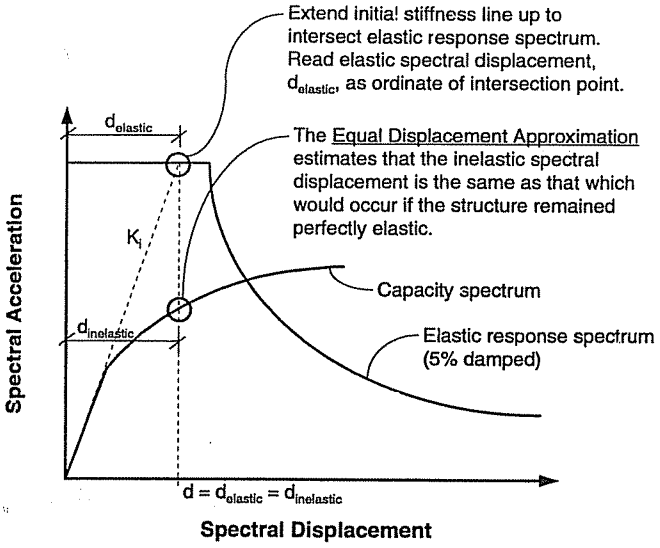


Figure 8-4. Equal Displacement Approximation

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 (ATC 1996a), sometimes called the coefficient method, is presented in Section 8.2.2.2. 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.

Commentary: *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 Figure 8-4, 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 longer period range (T > 1.0 seconds), the simple 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 seconds) the displacements obtained from the simple approximation may be significantly different from (less than) the results obtained using the capacity spectrum and coefficient methods.*

The equal displacement approximation is often a useful tool for estimating an initial trial performance point in the iterative capacity spectrum procedures described in Sections 8.2.2.1.2 and 8.2.2.1.3.

The target displacement obtained using the displacement coefficient method is equal to the displacement obtained using the equal displacement approximation modified by various coefficients.

8.2.2.1 Calculating Demand Using the Capacity Spectrum Method

The location of the Performance Point must satisfy two relationships: 1) the point must 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-damped 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. An approximate effective damping is 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 reductions 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 techniques. This section is organized as follows:

- **Conceptual Development of the Method (Section 8.2.2.1.1).** This section contains the theoretical basis of the method and the derivation of formulae. Considerable, and careful, study by the user is required to digest the background theory presented in this section. This section is not intended to serve as step by step instructions for determination of the performance point. For such step by step instructions, the user should proceed to Procedure A, B, or C , in which only the minimum required mathematical relationships of this section are referenced.
- **Procedure A (Section 8.2.2.1.2).** This is the most direct application of the concepts and relationships described in Section 8.2.2.1.1. Procedure A is truly iterative, but is formula-based and easily can be programmed into a spreadsheet. It is more an analytical method than a graphical method. It may be the best method for beginners because it is the most direct application of the methodology, and consequently is the easiest procedure to understand.
- **Procedure B (Section 8.2.2.1.3).** A simplification is introduced in the bilinear modeling of the capacity curve that enables a relatively direct solution for the performance point with little iteration. Like Procedure A, Procedure B is more an analytical method than a graphical method, and it is probably the most convenient for spreadsheet programming. Procedure B may be a less transparent application of the methodology than Procedure A.
- **Procedure C (Section 8.2.2.1.4).** Procedure C is a pure graphical method to find the performance point, similar to the originally conceived capacity spectrum method, and is consistent with the concepts and mathematical relationships described in Section 8.2.2.1.1. It is the most convenient method for hand analysis. It is not particularly convenient for spreadsheet programming. It is the least transparent application of the methodology.

Commentary: Users who study the three procedures above will have a good understanding of the capacity spectrum method, and will probably have their preferred procedure. However, other mathematical or graphical procedures may also be developed to provide a better interface with a user's other analytical methods.

8.2.2.1.1 Conceptual Development of the Capacity Spectrum Method

Conversion of the Capacity Curve to the Capacity Spectrum

To use the capacity spectrum method it is necessary to convert the capacity curve, which is in terms of base shear and roof displacement to what is called a capacity spectrum, which is a representation of the capacity curve in Acceleration-Displacement Response Spectra (ADRS) format (i.e., S_a versus S_d). The required equations to make the transformation are:

$$PF_1 = \left[\frac{\sum_{i=1}^{N} (w_i \phi_{i1}) / g}{\sum_{i=1}^{N} (w_i \phi_{i1}^2) / g} \right] \tag{8-1}$$

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N} (w_i \phi_{i1}) / g \right]^2}{\left[\sum_{i=1}^{N} w_i / g \right] \left[\sum_{i=1}^{N} (w_i \phi_{i1}^2) / g \right]} \tag{8-2}$$

$$S_a = \frac{V / W}{\alpha_1} \tag{8-3}$$

$$S_d = \frac{\Delta_{roof}}{PF_1 \, \phi_{roof,1}} \tag{8-4}$$

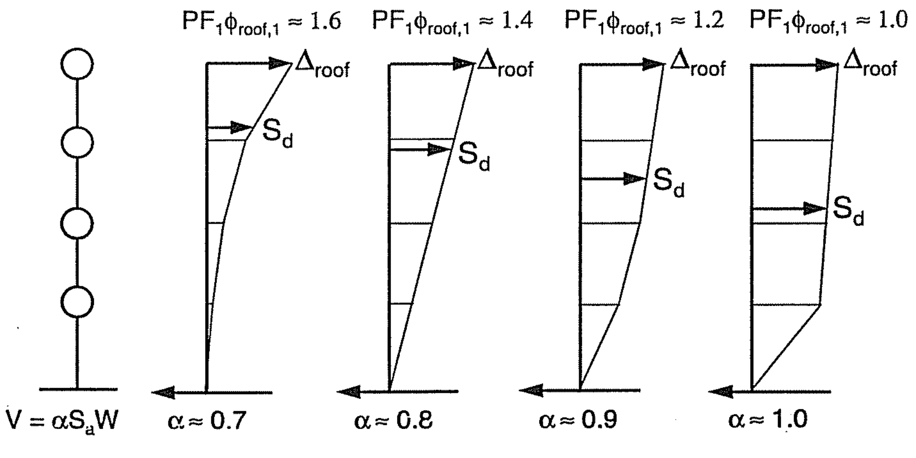


Figure 8-5. Example Modal Participation Factors and Modal Mass Coefficients

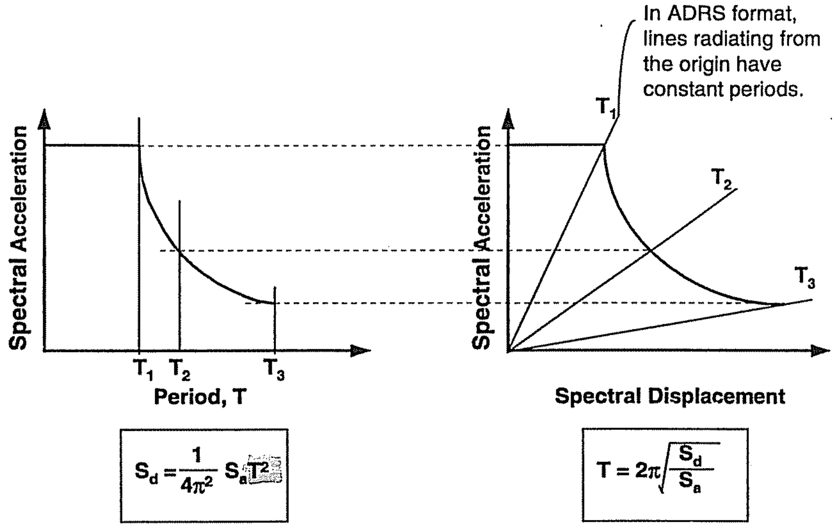
where:

PF_1 = modal participation factor for the first natural mode.
α_1 = modal mass coefficient for the first natural mode.
w_i/g = mass assigned to level i.
ϕ_{i1} = amplitude of mode 1 at level i.
N = level N, the level which is the uppermost in the main portion of the structure.
V = base shear.
W = building dead weight plus likely live loads, see Section 9.2.
Δ_{roof} = roof displacement (V and the associated Δ_{roof} make up points on the capacity curve).
S_a = spectral acceleration.
S_d = spectral displacement (S_a and the associated S_d make up points on the capacity spectrum).

Section 8.5 provides general information on structural dynamics and more information on the derivation of these equations. It is helpful to have some physical understanding of the relationship between the participation factor, the modal mass coefficient, and building displacement. As shown in Figure 8-5, the participation factor and the modal mass coefficient vary according to the relative interstory displacement over the height of the building. For example, for a linear distribution of interstory displacement over the height of the building, $\alpha \approx 0.8$ and $PF_1\phi_{roof,1} \approx 1.4$.

The general process for converting the capacity curve to the capacity spectrum, that is, converting the capacity curve into the ADRS format, is to first calculate the modal participation factor PF_1 and the modal mass coefficient α_1 using equations 8-1 and 8-2. Then for each point on the capacity curve, V, Δ_{roof}, calculate the associated point S_a, S_d on the capacity spectrum using equations 8-3 and 8-4.

Most engineers are familiar with the traditional S_a versus T representation of response spectra; however, they are less familiar with the S_a versus S_d (ADRS) representation. Figure 8-6 shows the same spectrum in each format. In the ADRS format, lines radiating from the origin have constant period. For any point on the ADRS spectrum, the period, T, can be computed using the relationship $T = 2\pi(S_d/S_a)^{1/2}$. Similarly, for any point on the traditional spectrum, the spectral



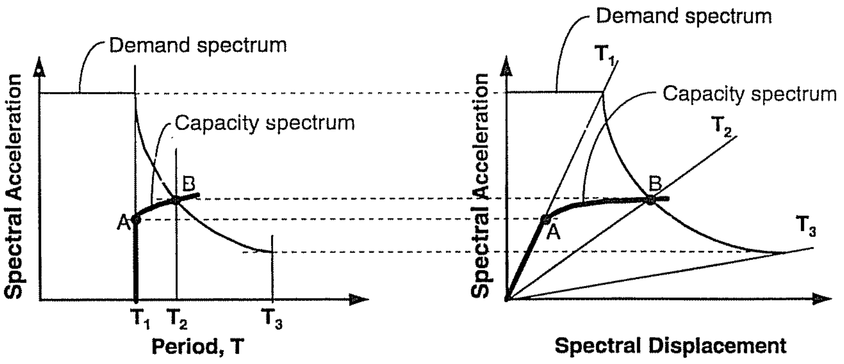
$S_d = \frac{1}{4\pi^2} S_a T^2$

$T = 2\pi\sqrt{\frac{S_d}{S_a}}$

Traditional Spectrum (S_a versus T)

ADRS Spectrum (S_a versus S_d)

Figure 8-6. Response Spectra in Traditional and ADRS Formats



Traditional Spectrum (S_a versus T)

ADRS Spectrum (S_a versus S_d)

Figure 8-7. Capacity Spectrum Superimposed Over Response Spectra in Traditional and ADRS Formats

displacement, S_d, can be computed using the relationship $S_d = S_a T^2 / 4\pi^2$. These two relationships are the same formula arranged in different ways.

Figure 8-7 shows the same capacity spectrum superimposed on each of the response spectra plots shown in Figure 8-6. Following along the capacity spectrum, the period is constant, at T_1, up until point A. When point B is reached, the period is T_2. This indicates that as a structure undergoes inelastic displacement, the period lengthens. The lengthening period is most apparent on the traditional spectrum plot, but it is also clear on the ADRS plot, remembering that lines of constant period radiate from the origin.

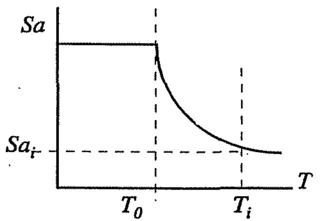
Conversion to ADRS Spectra

Application of the Capacity-Spectrum technique requires that both the demand response spectra and structural capacity (or pushover) curves 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, 1993.

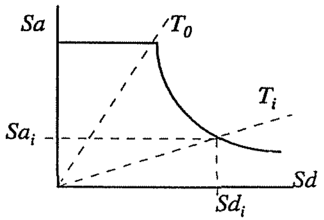
Response Spectrum Conversion

Every point on a response spectrum curve has associated with it a unique spectral acceleration, S_a, spectral velocity, S_v, spectral displacement, S_d and period, T. To convert a spectrum from the standard S_a vs T format found in the building code to ADRS format, it is necessary to determine the value of S_{di} for each point on the curve, S_{ai}, T_i. This can be done with the equation:

$$S_{d_i} = \frac{T_i^2}{4\pi^2} S_{a_i} g$$



Standard Format (Sa vs T)



ADRS Format (Sa vs Sd)

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:

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

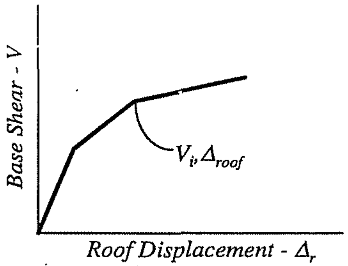
Capacity Spectrum Conversion

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 equations:

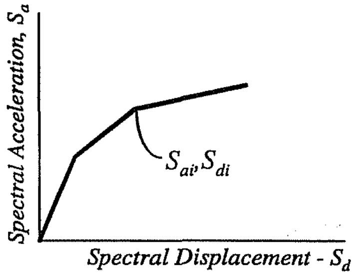
$$S_{ai} = \frac{V_i / W}{\alpha_1}$$

$$S_{di} = \frac{\Delta_{roof}}{\left(PF_1 \times \phi_{1,roof}\right)}$$

where α_1 and PF_1 are respectively the modal mass coefficient and participation factors for the first natural mode of the structure and $\phi_{1,roof}$ is the roof level amplitude of the first mode. See also Section 8.5, Basics of Structural Dynamics.



Capacity Curve



Capacity Spectrum

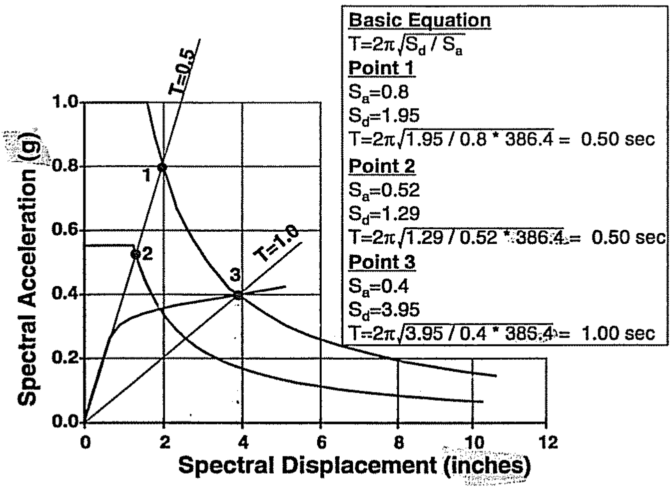


Figure 8-8. Lines of Constant Period and Period Lengthening in ADRS Format

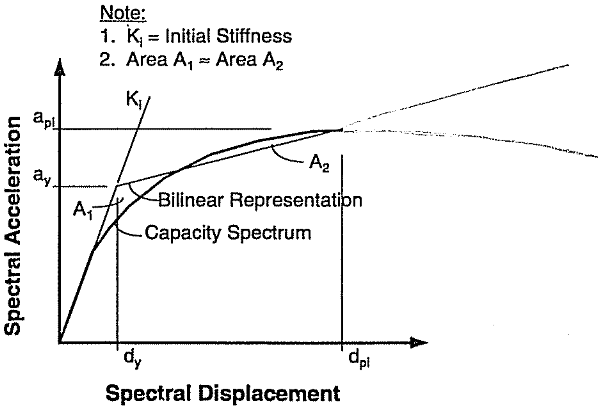


Figure 8-9. Bilinear Representation of Capacity Spectrum for Capacity Spectrum Method

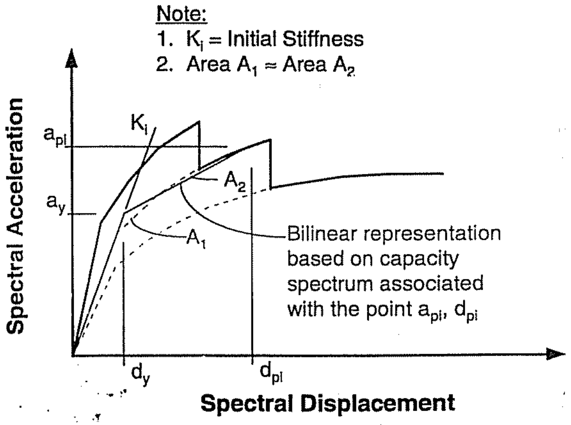


Figure 8-10. Bilinear Representation of Capacity Spectrum for Degrading ("Sawtooth") System

Figure 8-8 helps illustrate that, in the ADRS format, lines radiating from the origin have constant period, and that the period lengthens as the structure undergoes inelastic displacement. Points 1 and 2 in the figure lie on two different response spectra, and on a single line radiating from the origin. As shown by the calculation included in the figure, they both are associated with a period of 0.5 seconds. Point 3 has a 1.0 second period. Thus, for the capacity spectrum (pushover) shown, the elastic period of the structure is 0.5 seconds, and when the structure is pushed to point 3, with a spectral displacement of 3.95 inches, and considerable inelastic displacement, the period has lengthened to 1.0 seconds.

Construction of Bilinear Representation of Capacity Spectrum

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 a_{pi}, d_{pi}. 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. The first estimate of point a_{pi}, d_{pi} is designated a_{p1}, d_{p1}, the second a_{p2}, d_{p2}, and so on. Guidance on a first estimate of point a_{p1}, d_{p1} is given in the step-by-step process for each of the three procedures. Oftentimes, the equal displacement approximation can be used as an estimate of a_{p1}, d_{p1}.

Refer to Figure 8-9 for an example bilinear representation of a capacity spectrum. To

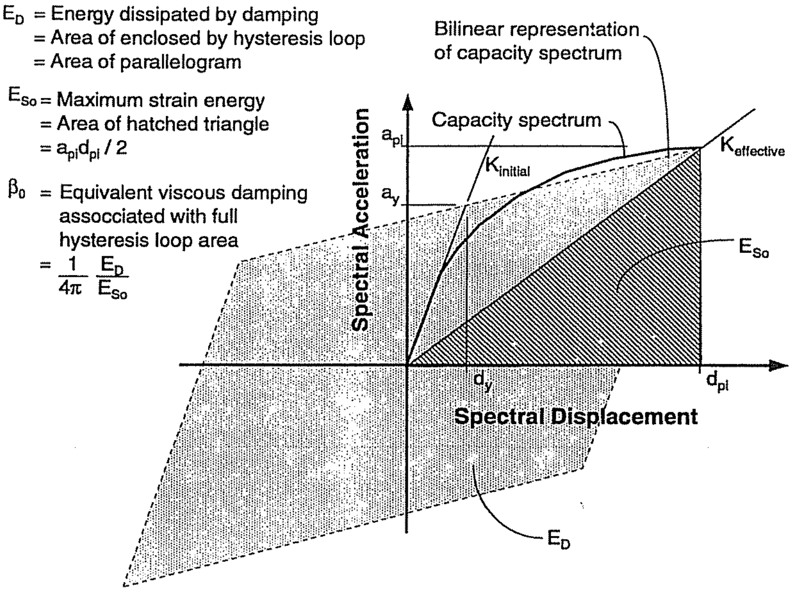


Figure 8-11. Derivation of Damping For Spectral Reduction

construct the bilinear representation draw one line up from the origin at the initial stiffness of the building using element stiffnesses as recommended in Chapter 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 point a_y, d_y, the area designated A_1 in the figure is approximately equal to the area designated A_2, The intent of setting area A_1 equal to area A_2 is to have equal area under the capacity spectrum and its bilinear representation, that is, to have equal energy associated with each curve.

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 Figure 8-10.

Estimation of Damping and Reduction of 5 percent Damped 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. Hysteretic damping can be represented as equivalent viscous damping using equations that are available in the literature.

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

$$\beta_{eq} = \beta_0 + 0.05 \qquad (8\text{-}5)$$

where,

β_0 = hysteretic damping represented as equivalent viscous damping

0.05 = 5% viscous damping inherent in the structure (assumed to be constant)

The term β_0 can be calculated as (Chopra 1995):

$$\beta_0 = \frac{1}{4\pi}\frac{E_D}{E_{So}} \qquad (8\text{-}5a)$$

where,

E_D = energy dissipated by damping

E_{So} = maximum strain energy

The physical significance of the terms E_D and E_{So} in equation 8-5a is illustrated in Figure 8-11.

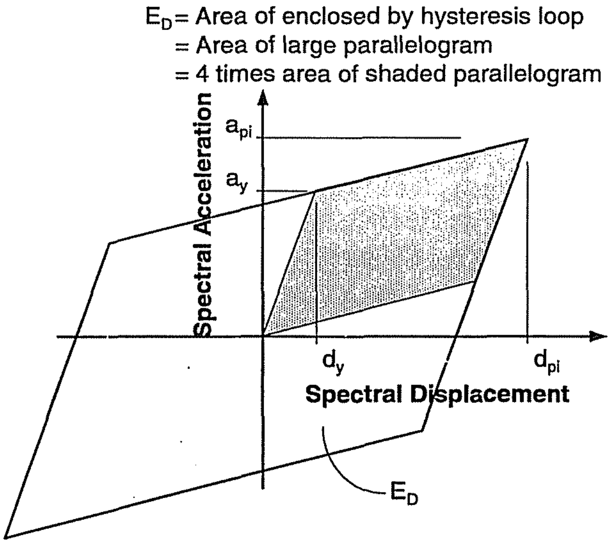


Figure 8-12. Derivation of Energy Dissipated by Damping, E_D

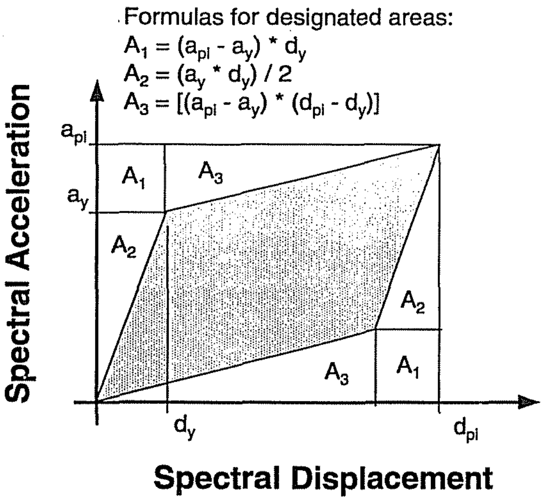


Figure 8-13. Derivation of Energy Dissipated by Damping, E_D

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 hatched triangle.

Referring to Figures 8-11, 8-12 and 8-13, the term E_D can be derived as

$$
\begin{aligned}
E_D &= 4^*(\text{shaded area in Figures 8-12 or 8-13})\\
&= 4(a_{pi}d_{pi} - 2A_1 - 2A_2 - 2A_3)\\
&= 4[a_{pi}d_{pi}-a_yd_y-(d_{pi}-d_y)(a_{pi}-a_y) - 2d_y(a_{pi}-a_y)]\\
&= 4(a_yd_{pi} - d_ya_{pi})
\end{aligned}
$$

Referring to Figure 8-11, the term E_{So} can be derived as

$$E_{So} = a_{pi}d_{pi} /2$$

Commentary: *Note that E_{So} could also be written as $k_{\text{effective}}d_{pi}^2/2$.*

Thus , β_0 can be written as:

$$\beta_0 = \frac{1}{4\pi}\frac{4(a_yd_{pi} - d_ya_{pi})}{a_{pi}d_{pi} / 2} = \frac{2}{\pi}\frac{a_yd_{pi} - d_ya_{pi}}{a_{pi}d_{pi}}$$

$$\beta_0 = \frac{0.637(a_yd_{pi} - d_ya_{pi})}{a_{pi}d_{pi}}$$

and when β_0 is written in terms of percent critical damping, the equation becomes:

$$\beta_0 = \frac{63.7(a_yd_{pi} - d_ya_{pi})}{a_{pi}d_{pi}} \qquad (8\text{-}6)$$

Thus β_{eq} becomes:

$$\beta_{eq} = \beta_0 + 5 = \frac{63.7(a_yd_{pi} - d_ya_{pi})}{a_{pi}d_{pi}} + 5 \qquad (8\text{-}7)$$

The equivalent viscous damping values obtained from equation 8-7 can be used to estimate spectral reduction factors using relationships developed by Newmark and Hall [Newmark and Hall, 1982]. As shown in Figure 8-14, 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 equation 8-7 and Newmark and Hall equations are consistent with similar factors contained in base isolation codes and in the FEMA Guidelines (these factors are presented in these other documents as the damping coefficient, B, which is equal to 1/SR, see the commentary below). The committees who

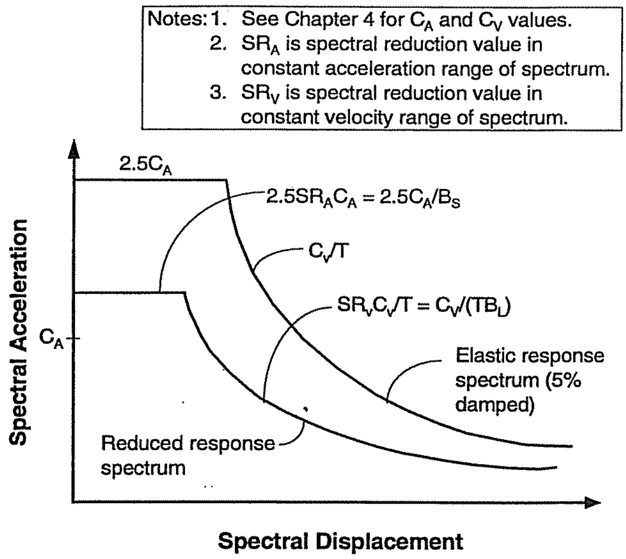


Figure 8-14. Reduced Response Spectrum

developed these damping coefficients concluded that spectra should not be reduced to this extent at higher damping values and judgmentally increased the coefficients starting at about 25 percent 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.

Commentary: *This document is written using the concept of spectral reduction factors, SR. However, both the term spectral reduction (SR) and the term damping coefficient (B=1/SR) are carried through this section. The term spectral reduction factor has been used widely in the literature and previously in this project, but B, damping coefficient, is currently in use in other codes such as the 1991 UBC, 1994 UBC, FEMA Guidelines and 1994 NEHRP Provisions. 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 is derived from a formula which includes the variable, β.*

The idealized hysteresis loop shown in Figure 8-11 is a reasonable approximation for a ductilely 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 Figure 8-11 lead to overestimates of equivalent viscous damping because the actual hysteresis loops are imperfect, that is, they are reduced in area, or pinched.

This document addresses existing reinforced concrete buildings that are not typically ductile structures. For such buildings, calculation of the equivalent viscous damping using equation 8-7 and the idealized hysteresis loop in Figure 8-11 yields results that overestimate realistic levels of damping. In this document, in order to be consistent with these previously developed damping 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, κ, has been introduced. Effective viscous damping, β_{eff}, is defined by:

$$\beta_{eff} = \kappa\beta_0 + 5 = \frac{63.7\kappa(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5 \qquad (8\text{-}8)$$

Note that equation 8-8 is identical to equation 8-7 except that the κ-factor has been introduced to modify the first (β_0) term.

The κ-factor is a measure of the extent to which the actual building hysteresis is well represented by the parallelogram of Figure 8-11, either initially, or after degradation. The κ-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, this document simulates three categories of structural behavior. Structural behavior Type A represents stable, reasonably full hysteresis loops most similar to Figure 8-11, and is assigned a κ of 1.0 (except at higher damping values as discussed above). Type B is assigned a basic κ of 2/3 and represents a moderate reduction of area (κ is also reduced at higher values of β_{eff} to be consistent with the Type A relationships). Type C represents poor hysteretic behavior with a substantial reduction of loop area (severely pinched) and is assigned a κ of 1/3.

The ranges and limits for the values of κ assigned to the three structural behavior types are given in Table 8-1 and illustrated in Figure 8-15. Although arbitrary, they represent the consensus opinion of the product development team. The value of κ for structural behavior Type A (good behavior), is derived from the spectrum reduction factors, B, specified in the Uniform Building Code (ICBO 1994) and the NEHRP Provisions (BSSC 1995) for the design of new base isolated buildings. The values of κ assigned to the other two types are thought to be reasonable for average and poor structural behavior. The numerical derivation of spectral reduction factors used in this methodology, based on these assigned values of κ, follows.

Numerical Derivation of Spectral Reductions

The equations for the reduction factors SR_A (equal to $1/B_S$) and SR_V (equal to $1/B_L$) are given by:

$$SR_A = \frac{1}{B_S} \approx \frac{3.21 - 0.68\ln(\beta_{eff})}{2.12} \tag{8-9}$$

$$= \frac{3.21 - 0.68\ln\left[\dfrac{63.7\kappa(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5\right]}{2.12}$$

$$\geq \text{Value in Table 8-2}$$

$$SR_V = \frac{1}{B_L} \approx \frac{2.31 - 0.41\ln(\beta_{eff})}{1.65} \tag{8-10}$$

$$= \frac{2.31 - 0.41\ln\left[\dfrac{63.7\kappa(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5\right]}{1.65}$$

$$\geq \text{Value in Table 8-2}$$

Note that the values for SR_A and SR_V should be greater than or equal to the values given in Table 8-2.

Table 8-1. Values for Damping Modification Factor, κ

Structural Behavior Type[1]	β_o (percent)	κ
Type A[2]	≤ 16.25	1.0
	>16.25	$1.13 - \dfrac{0.51(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}}$
Type B	≤ 25	0.67
	>25	$0.845 - \dfrac{0.446(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}}$
Type C	Any value	0.33

1. See Table 8-4 for structural behavior types.
2. The formulas are derived from Tables of spectrum reduction factors, B (or BI), specified for the design of base isolated buildings in the 1991 UBC, 1994 UBC and 1994 NEHRP Provisions. The formulas created for this document give the same results as are in the Tables in the other documents.

Table 8-2. Minimum Allowable SR_A and SR_V Values[1]

Structural Behavior Type[2]	SR_A	SR_V
Type A[2]	0.33	0.50
Type B	0.44	0.56
Type C	0.56	0.67

1. Values for SR_A and SR_V shall not be less than those shown in this Table
2. See Table 8-4 for structural behavior types.

To illustrate the effect of the structural behavior types on the spectral reduction factors, Figures 8-15, 8-16, 8-17 and 8-18 respectively show graphical representations of κ, β_{eff}, SR_A and SR_V versus β_o for structural behavior types A, B and C. Note that β_o is the equivalent viscous damping representation of the hysteretic damping associated with the full area of the hysteresis loop formed by the bilinear approximation of the capacity spectrum, as shown in Figure 8-11.

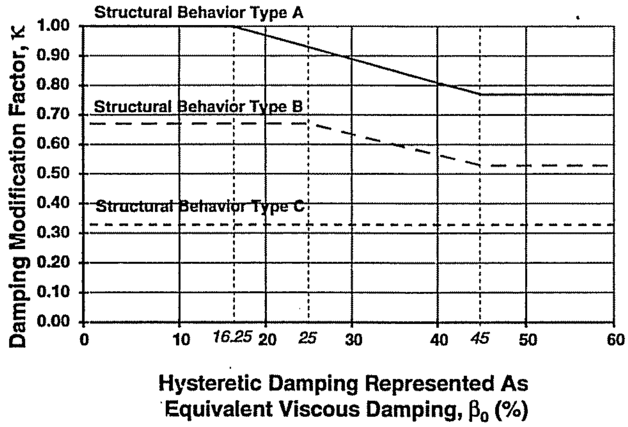


Figure 8-15. Damping Modification Factor, κ, for Structural Behavior Types A, B and C

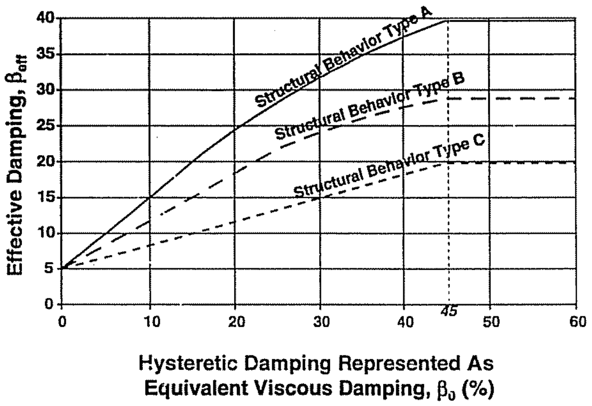


Figure 8-16. Effective Damping, βeff, for Structural Behavior Types A, B and C

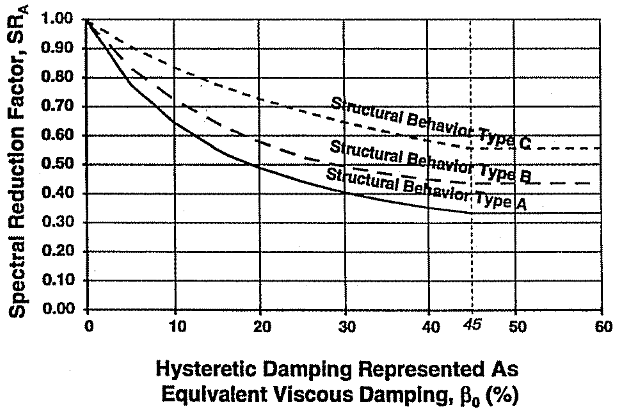


Figure 8-17. Spectral Reduction Factor, SRA, for Structural Behavior Types A, B and C

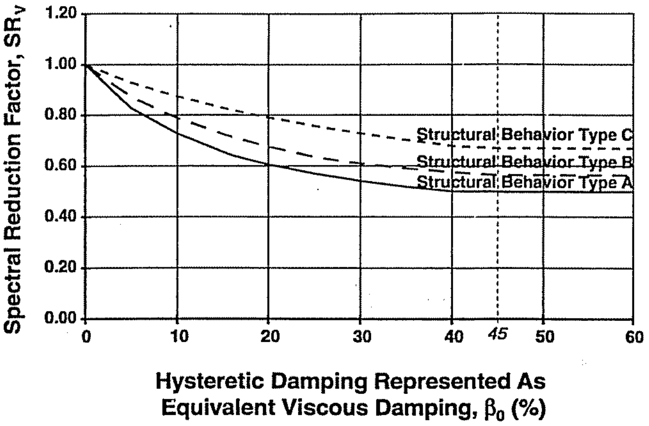


Figure 8-18. Spectral Reduction Factor, SRV, for Structural Behavior Types A, B and C

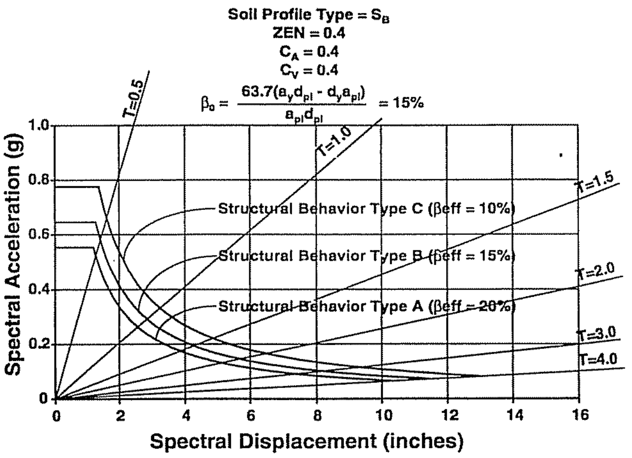


Figure 8-19. Example ADRS Response Spectra for Structural Behavior Types A, B and C

Table 8-3. Spectral Reduction Factors, $SR_A = 1/B_S$ and $SR_V = 1/B_L$

	Behavior Type A[1]			Behavior Type B[1]			Behavior Type C[1]		
β_0 (percent)	β_{eff}	SR_A $(1/B_S)$	SR_V $(1/B_L)$	β_{eff}	SR_A $(1/B_S)$	SR_V $(1/B_L)$	β_{eff}	SR_A $(1/B_S)$	SR_V $(1/B_L)$
0	5	1.00	1.00	5	1.00	1.00	5	1.00	1.00
5	10	0.78	0.83	8	0.83	0.87	7	0.91	0.93
15	20	0.55	0.66	15	0.64	0.73	10	0.78	0.83
25	28	0.44	0.57	22	0.53	0.63	13	0.69	0.76
35	35	0.38	0.52	26	0.47	0.59	17	0.61	0.70
≥45	40	0.33	0.50^2	29	0.44	0.56	20	0.56	0.67^2

1. Structural behavior type, see Table 8-4.
2. Controlled by minimum allowable value for SRv, see Table 8.2

Figure 8-19 shows the difference in the response spectra in ADRS format for structural behavior types A, B and C when β_0 equals 15%.

The spectral reduction factors SR_A (equal to $1/B_S$) and SR_V (equal to $1/B_L$) given in equations 8-9 and 8-10 can be put in tabular form as shown in Table 8-3. Enter the Table with the parameter β_0, the equivalent viscous damping representation of the hysteretic damping associated with the full area of the hysteresis loop formed by the bilinear approximation of the capacity spectrum in Figure 8-11.

***Commentary:** The inverted value of B_L for structural behavior Type A is identical to the damping coefficient used in other codes such as the 1991 UBC, 1994 UBC and 1994 NEHRP Provisions.*

Table 8-4. Structural Behavior Types

Shaking Duration[1]	Essentially New Building[2]	Average Existing Building[3]	Poor Existing Building[4]
Short	Type A	Type B	Type C
Long	Type B	Type C	Type C

1. See Section 4.5.2 for criteria.
2. Buildings whose primary elements make up an essentially new lateral system and little strength or stiffness is contributed by noncomplying elements.
3. Buildings whose primary elements are combinations of existing and new elements, or better than average existing systems.
4. Buildings whose primary elements make up noncomplying lateral force systems with poor or unreliable hysteretic behavior.

The selection of structural behavior type depends on both the quality of the primary elements of the seismic resisting system and the duration of shaking, as shown in Table 8-4. Criteria to determine the appropriate duration category for a given situation are given in Chapter 4 (Section 4.5.2). The first column of structural behavior types, *Essentially New Buildings,* represents buildings with new lateral force resisting systems and detailing complying with current code, in which the existing noncomplying elements contribute little strength or stiffness. The third column of Table 8-4, *Poor Existing Buildings*, represents existing buildings with a lateral force resisting system with unknown or unreliable hysteretic behavior or behavior known to degrade or be severely pinched. The middle column, *Average Existing Buildings*, should be used for other cases, and may be appropriate for the majority of existing retrofit buildings.

***Commentary:** As described in Section 3.4, a seismic performance objective is defined by selecting a desired building performance level for a*

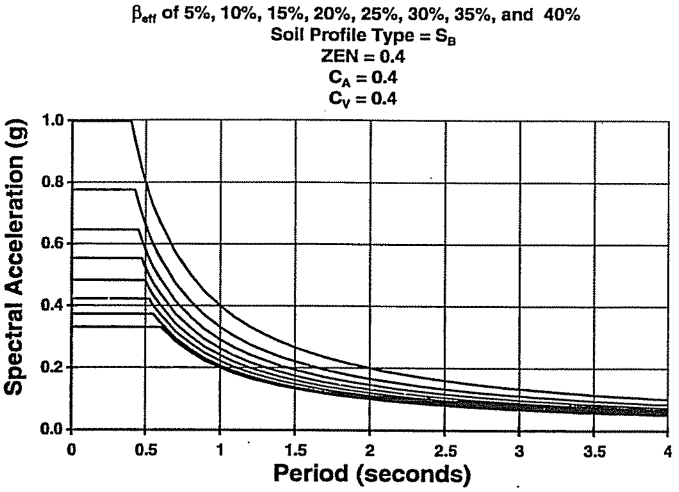


Figure 8-20. Family of Demand Spectra in Traditional Sa versus T Format

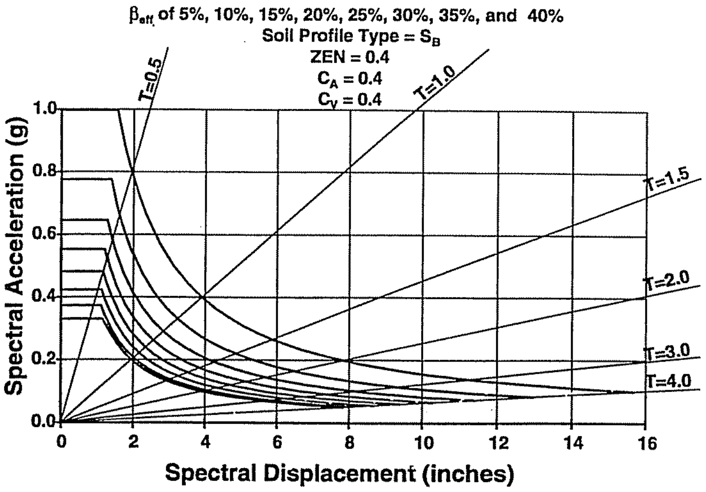


Figure 8-21. Family of Demand Spectra in ADRS Format

site. For example, a building may have short duration shaking for a Serviceability Earthquake (usually 50% chance of being exceeded in 50 years) and long duration shaking for a Design Earthquake (usually 10% chance of being exceededin 50 years). As another example, a building may have short duration shaking for the maximum shaking expected from a single event on a one adjacent fault, and long duration shaking for the maximum shaking expected from a single event on another adjacent fault.

Development of the Demand Spectrum

The 5 percent response spectrum can be developed based on the information in Chapter 4. The reduced 5 percent response spectrum, called the demand spectrum, can be plotted as shown in Figure 8-14. A procedure for converting the response spectrum from the standard S_a versus T format to the S_a versus S_d format (ADRS) is given near the beginning of Section 8.2.2.1.1.

Figure 8-20 depicts an example family of demand spectra, each spectrum representing a different level of effective damping, plotted in the given level of earthquake ground motion. Dual- or multiple-level performance objectives can be created by selecting two or more different performances, each for a different level of ground motion. With multiple-level performance objectives, it is possible, based on the criteria for duration of ground shaking given in Section 4.5.2, to have short earthquake shaking duration for one ground motion and long earthquake shaking duration for another ground motion at the same traditional S_a versus T format. Figure 8-21 shows the same family of demand spectra, each spectrum representing a different level of effective damping, plotted in the ADRS format. Families of demand spectra, such as these, can be plotted for any combination of soil profile type and earthquake shaking intensity using the information provided in Chapter 4. Such families of demand spectra, plotted in the ADRS format, can be quite useful when analyzing the structure using the capacity spectrum method.

Intersection of Capacity Spectrum and Demand Spectrum

When the displacement at the intersection of the demand spectrum and the capacity spectrum, d_i, is within 5 percent ($0.95d_{pi} \le d_i \le 1.05\ d_{pi}$) of the displacement of the trial performance point, a_{pi}, d_{pi}, d_{pi} becomes the performance point. If the intersection of the demand spectrum and the capacity spectrum is not within the acceptable tolerance, then a new a_{pi}, d_{pi} point is selected and the process is repeated. Figure 8-22 illustrates the concept. The performance point represents the maximum structural displacement expected for the demand earthquake ground motion.

When the capacity spectrum is a "sawtooth" curve, that is, the final composite capacity spectrum is constructed from several different capacity spectra which account for strength degradation of elements, special care must be taken in determining the performance point. The bilinear representation of the capacity spectrum, that is used to determine the reduction factors for the 5 percent damped spectrum, is constructed for a single capacity spectrum curve, not the composite curve. For the analysis to be acceptable, the bilinear representation must be for the same single capacity spectrum curve that makes up the portion of the composite capacity spectrum where the intersection point occurs. Figure 8-23 illustrates the concept for a "sawtooth" capacity spectrum.

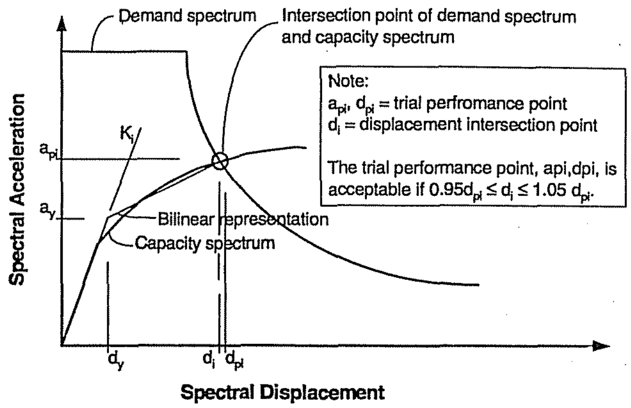
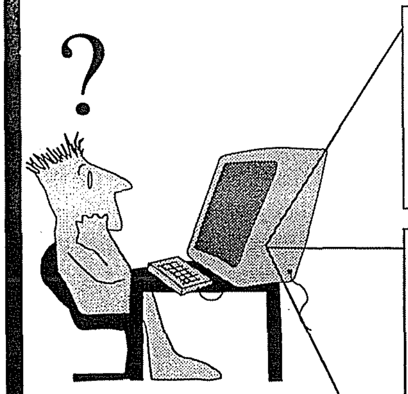


Figure 8-22. Intersection Point of Demand and Capacity Spectrums Within Acceptable Tolerance

Commentary: If the performance point is found to fall near a step in the "sawtooth" capacity spectrum, then the engineer should be aware that, because of the variability of the analysis, the actual building displacement could be on either side of the step. The engineer should consider both points when examining building performance.



Question: Which capacity spectrum procedure should I use?

Answer: It largely depends on personal preference, but these guidelines may help.

Procedure A:
- **Clearest, most transparent and most direct application of the methodology**
- **Analytical method**
- **Convenient for spreadsheet programming**
- **May be the best method for beginners because it is most direct and thus easiest to understand**

Procedure B:
- **Analytical method**
- **Simpler than procedure A because of simplifying assumptions (that may not always be valid)**
- **Most convenient for spreadsheet programming**
- **Reasonably transparent application of methodology**
- **Users of this method should fully understand the inherent assumptions**

Procedure C:
- **Graphical method**
- **Most convenient method for hand analysis**
- **Not as convenient for spreadsheet programming**
- **Least transparent application of methodology**

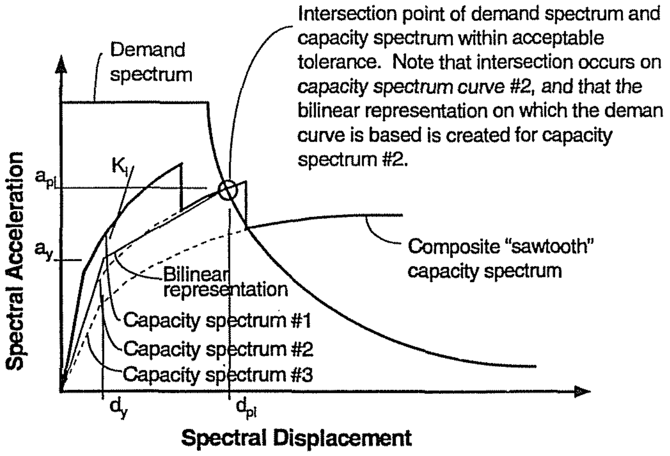


Figure 8-23. Intersection Point of Demand Spectrum and "Sawtooth" Capacity Spectrum

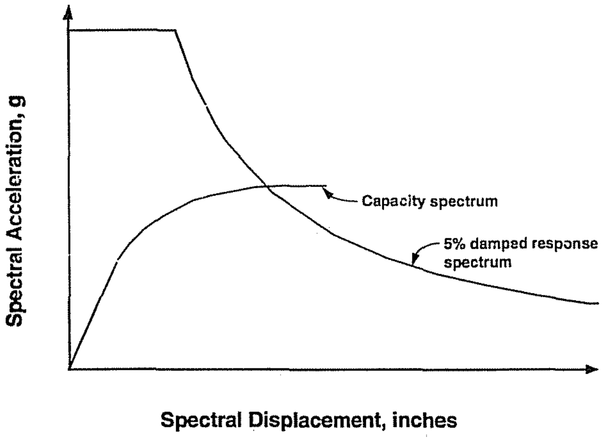


Figure 8-24. Capacity Spectrum Procedure A After Step 2

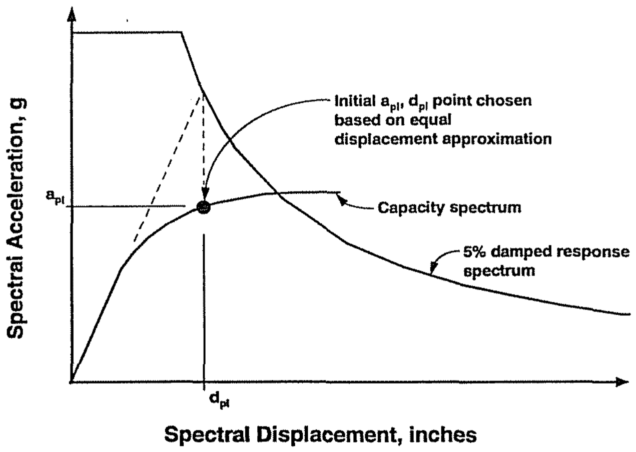


Figure 8-25. Capacity Spectrum Procedure A After Step 3

8.2.2.1.2 Calculating Performance Point Using Procedure A. In this procedure, iteration is done by hand or by spreadsheet methods to converge on the performance point. This procedure is the most direct application of the principles described above. The following steps are involved:

1. Develop the 5 percent damped (elastic) response spectrum appropriate for the site using the procedures provided in Chapter 4.
2. Transform the capacity curve into a capacity spectrum as described in Section 8.2.2.1.1 using equations 8-1, 8-2, 8-3 and 8-4. Plot the capacity curve on the same chart as the 5% damped response spectra as shown in Figure 8-24.
3. Select a trial performance point, a_{pi}, d_{pi} as shown in Figure 8-25.

   Commentary: A first choice of point a_{pi}, d_{pi} could be the displacement obtained using the equal displacement approximation, or, it might be the end point of the capacity spectrum, or, it might be any other point chosen on the basis of engineering judgment.
4. Develop a bilinear representation of the capacity spectrum using the process described in Section 8.2.2.1.1 and illustrated in Figure 8-9. The result of this step is illustrated in Figure 8-26.

   Commentary:** In the case of a composite "sawtooth" capacity spectrum, the bilinear representation should be based on the capacity spectrum that makes up the portion of the composite capacity spectrum where the trial performance point* ***a_{pi}, d_{pi} occurs.
5. Calculate the spectral reduction factors as given in equations 8-9 and 8-10. Develop the

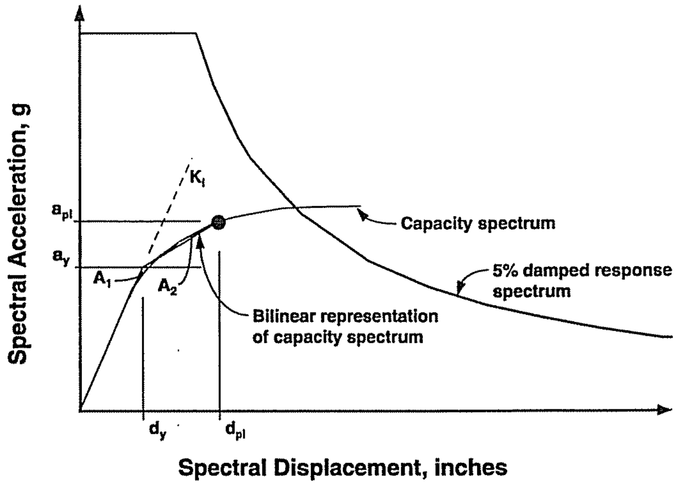


Figure 8-26. Capacity Spectrum Procedure A After Step 4

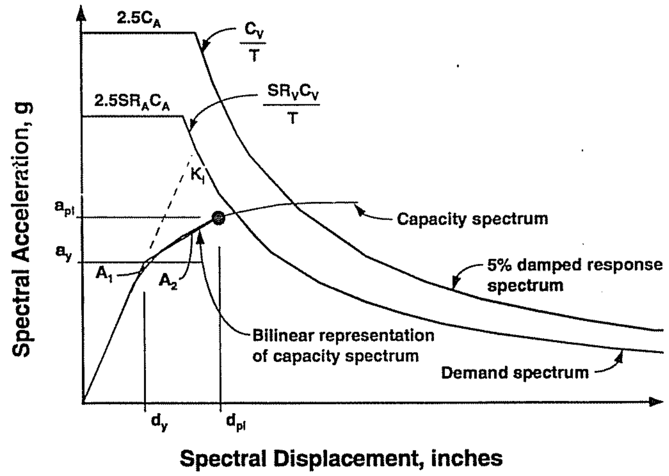
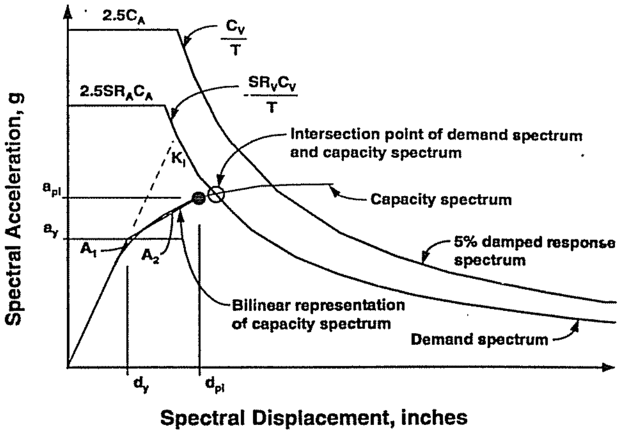


Figure 8-27. Capacity Spectrum Procedure A After Step 5

demand spectrum using the process illustrated in Figure 8-14. Draw the demand spectrum on the same plot as the capacity spectrum as shown in Figure 8-27.

6. Refer to Figure 8-28. Determine if the demand spectrum intersects the capacity spectrum at



1.

Figure 8-28. Capacity Spectrum Procedure A After Step 6

the point, api, dpi. or if the displacement at which the demand spectrum intersects the capacity spectrum, di, is within acceptable tolerance of dpi. The acceptable tolerance is illustrated in Figure 8-22.

7. If the demand spectrum does not intersect the capacity spectrum within acceptable tolerance, then select a new a_{pi}, d_{pi} point and return to step 4.

   Commentary: A new choice of point a_{pi}, d_{pi} might be the intersection point determined in step 6, or any other point chosen on the basis of engineering judgment.

8. If the demand spectrum intersects the capacity spectrum within acceptable tolerance, then the trial performance point, a_{pi}, d_{pi}, is the performance point, a_p, d_p, and the displacement, d_p, represents the maximum structural displacement expected for the demand earthquake.

   Commentary: Calculation of the demand displacement using capacity spectrum procedure A can be done by hand, graphically, or it can be done in a spreadsheet, graphically. In the spreadsheet method, the capacity spectrum would be graphed. Next a trial value of the performance point, a_{pi}, d_{pi}, would be selected. Based on the trial *performance point, trial values of point* a_y, d_y *can be chosen to define the bilinear representation of the capacity spectrum. This bilinear representation can be automatically plotted on the same chart as the capacity spectrum. The values of point* a_y, d_y *can be revised until the bilinear representation meets the requirements of step 4. Once given points* a_{pi}, d_{pi}, *and* a_y, d_y *the spreadsheet can then be set up to automatically calculate the spectral reduction factors and to plot the demand spectrum on the same chart as the capacity spectrum. The chart can then be reviewed to see if the intersection of the capacity spectrum and the demand spectrum is within acceptable tolerance. If it is not within acceptable tolerance, a new point* a_{pi}, d_{pi} *can be selected and the process repeated.*

8.2.2.1.3 Calculating Performance Point Using Procedure B. This procedure makes a simplifying assumption that is not made in the other two procedures. It assumes that not only the initial slope of the bilinear representation of the capacity curve remains constant, but also the point a_y, d_y and the post-yield slope remains constant. This simplifying assumption allows a direct solution without drawing multiple curves because it forces the effective damping , β_{eff}, to depend *only* on d_{pi}. The following steps are involved:

1. Develop the 5 percent damped response spectrum appropriate for the site using the procedures provided in Chapter 4.
2. Draw the 5 percent damped response spectrum and draw a family of reduced spectra on the same chart. It is convenient if the spectra plotted correspond to effective damping values (β_{eff}) ranging from 5 percent to the maximum value allowed for the building's structural behavior type. The maximum β_{eff} for Type A construction is 40 percent, Type B construction is 29 percent and Type C construction is 20 percent. Figure 8-29 shows an example family of demand spectra.

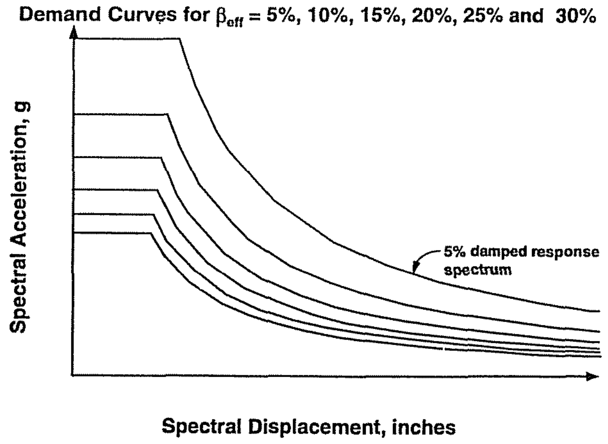


Figure 8-29. Capacity Spectra Procedure "B" After Step 2

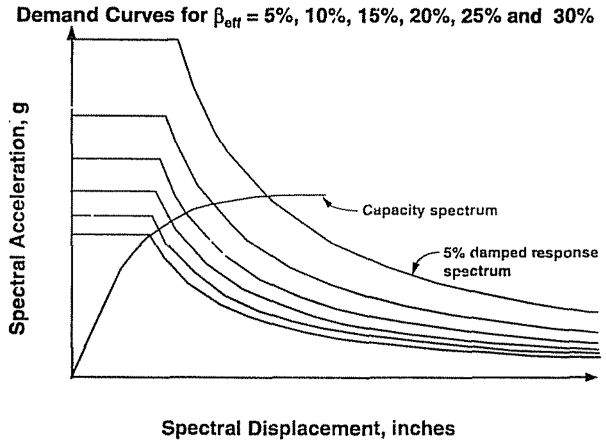


Figure 8-30. Capacity Spectra Procedure "B" After Step 3

3. Transform the capacity curve into a capacity spectrum as described in Section 8.2.2.1.1 using equations 8-1, 8-2, 8-3 and 8-4. Plot the capacity spectrum on the same chart as the family of demand spectra, as shown in Figure 8-30.

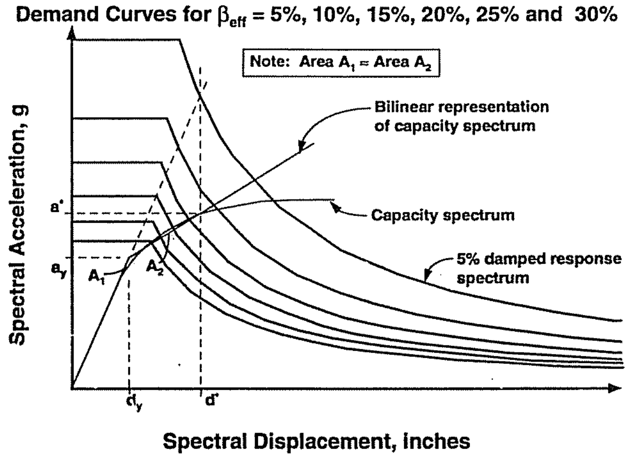


Figure 8-31. Capacity Spectrum Procedure "B" After Step 4

4. Develop a bilinear representation of the capacity spectrum as illustrated in Figure 8-31. The initial slope of the bilinear curve is equal to the initial stiffness of the building (as specified in Chapter 9). The post-yield segment of the bilinear representation should be run through the capacity spectrum at a displacement equal to the spectral displacement of the 5 percent damped spectrum at the initial pre-yield stiffness (equal displacement rule), point a*, d*. The post-yield segment should then be rotated about this point to balance the areas A_1 and A_2 as shown in Figure 8-31.

   Commentary: Step 3 is where the simplifying assumption is made in this procedure. It sets the slope of the post-yield segment of the bilinear representation of the capacity spectrum to a constant value, and therefore allows β_{eff} to be expressed directly in terms of d_{pi}. Requiring the post-yield segment to pass through the capacity spectrum at the point of elastic displacement is intended to assure that the post-yield segment is closely simulating the capacity spectrum in this region. If the performance point does not occur in this region, then the engineer may want to verify the results using procedure A or C.

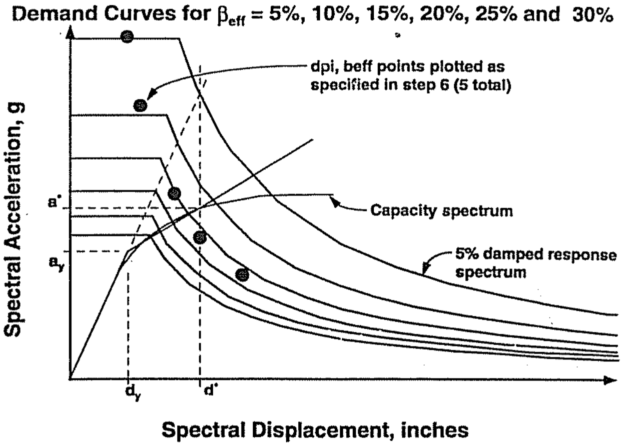


Figure 8-32. Capacity Spectrum Procedure B After Step 6

5. Calculate the effective damping for various displacements near the point a*, d*. The slope of the post-yield segment of the bilinear representation of the capacity spectrum is given by:

$$post\ yield\ slope = \frac{a* - a_y}{d* - d_y} \tag{8-11}$$

For any point a_{pi}, d_{pi}, on the post-yield segment of the bilinear representation, the slope is given by:

$$post\ yield\ slope = \frac{a_{pi} - a_y}{d_{pi} - d_y} \tag{8-12}$$

Since the slope is constant, equations 8-11 and 8-12 can be equated:

$$\frac{a* - a_y}{d* - d_y} = \frac{a_{pi} - a_y}{d_{pi} - d_y} \tag{8-13}$$

Solve equation 8-13 for a_{pi} in terms of d_{pi}. Call a_{pi} solved for in these terms a_{pi}'.

$$a_{pi}' = \frac{(a* - a_y)(d_{pi} - d_y)}{d* - d_y} + a_y \tag{8-14}$$

This value can be substituted for a_{pi} into equation 8-8 to obtain an expression for β_{eff} that is in terms of only one unknown, d_{pi}.

$$\beta_{eff} = \frac{63.7\kappa\left(a_y d_{pi} - d_y a_{pi}'\right)}{a_{pi}'\, d_{pi}} + 5 \tag{8-15}$$

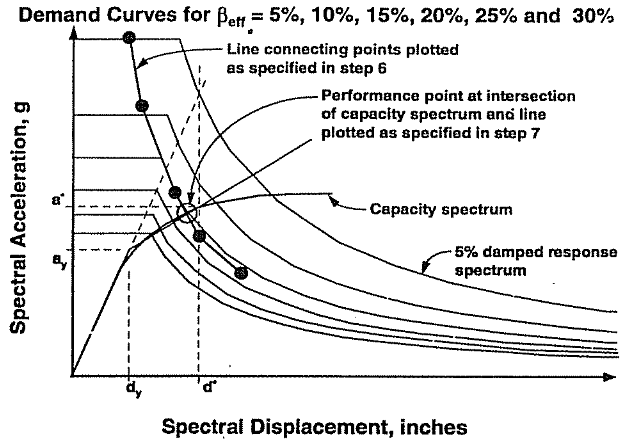


Figure 8-33. Capacity Spectrum Procedure B After Step 7

Solve equation 8-15 for β_{eff} for a series of d_{pi} values. When entering Table 8-1 to find the κ factor, or entering Table 8-3, then substitute the expression $\frac{63.7(a_y d_{pi} - d_y a_{pi}')}{a_{pi}' d_{pi}}$ for β_0, that is, $\frac{63.7(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}}$.

6. For each d_{pi} value considered in step 5, plot the resulting d_{pi}, β_{eff} point on the same chart as the family of demand spectra and the capacity spectrum. Figure 8-32 shows five of these points.
7. As illustrated in Figure 8-33, connect the points created in step 6, to form a line. The intersection of this line with the capacity spectrum defines the performance point. This procedure provides the same results as the other procedures if the performance point is at point a*, d*. The results will differ slightly from the other procedures if the performance point is not at point a*, d*. If the performance point is found to be distant from point a*, d*, then the engineer may want to verify the results using procedure A or C.

Commentary: *Although procedure B plots multiple d_{pi}, β_{eff} points, the only d_{pi}, β_{eff} point has any real significance is the one that lies on the capacity spectrum curve. This point defines the intersection point of the capacity spectrum with the appropriately damped demand spectrum, and thus defines the demand displacement. The other d_{pi}, β_{eff} points plotted are merely a means of zeroing in on the demand displacement.*

The steps listed above for procedure B could all be automated in a spreadsheet, or some other type of computer program, except for steps 6 and 7 where the d_{pi}, β_{eff} points are plotted. Those steps need to be done by hand. The procedure could be extended to automate these steps as well, although the procedure becomes more complex. With this extension, it would not be necessary to plot the multiple demand curves. The following steps would apply:

1. *Plot the 5% damped (elastic) spectrum and the capacity spectrum on the same chart.*
2. *Develop a bilinear representation of the capacity spectrum as illustrated in Figure 8-12.*
3. *Choose values of d_{pi} and solve for the corresponding a_{pi}' and β_{eff} values using equations 8-14 and 8-15 respectively.*
4. *Solve for T_s, the period where the 5% damped spectrum changes from the constant acceleration range to the constant velocity range as $T_s = C_V/2.5C_A.5$.*
5. *For each value of d_{pi}, solve for the corresponding period as $T = 2\pi(d_{pi}/a_{pi}')^{1/2}$.*
6. *For each period, T, (or displacement d_{pi}) solve for the corresponding spectral acceleration on the 5% damped spectrum, $S_{a5\%}$, as $S_{a5\%} = 2.5C_A$ if $T \leq T_s$ or $S_{a5\%} = CV/T$ if $T > T_s$.*
7. *For each spectral acceleration on the 5% damped spectrum, $S_{a5\%}$, calculate the corresponding spectral displacement, $S_{d5\%}$, as $S_{d5\%} = S_{a5\%}(T/2\pi)^2$.*
8. *For each T (or displacement d_{pi}), if $T \leq T_s$, then solve for the spectral reduction value, SR_A, using equation 8-9 with a_{pi}' substituted for a_{pi}. If $T > T_s$ then solve for the spectral reduction*

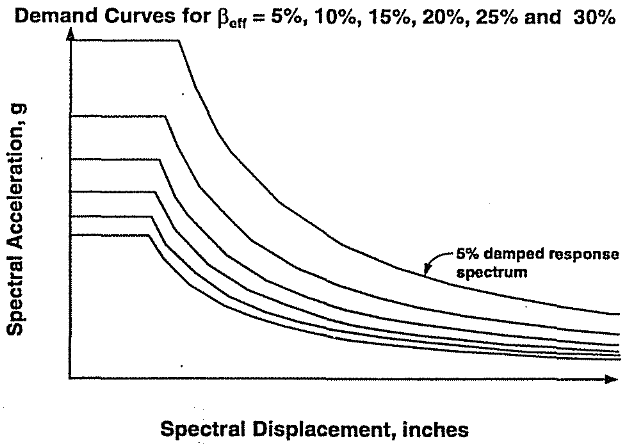


Figure 8-34. Capacity Spectra Procedure "C" After Step 2

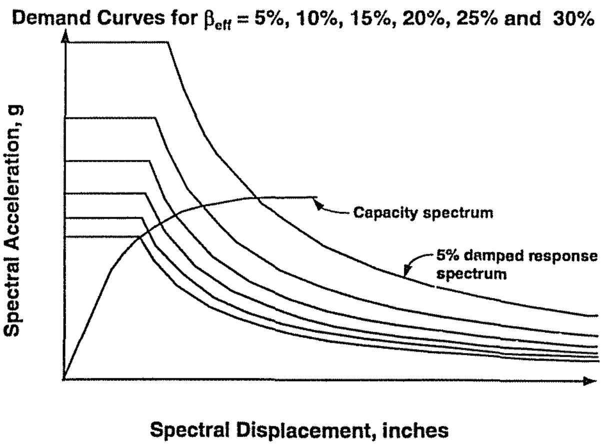


Figure 8-35. Capacity Spectra Procedure "C" After Step 3

value, SR_V, using equation 8-10 with a_{pi}' substituted for a_{pi}.

9. *For each T (or displacement d_{pi}), plot the point where $S_a = SR_X S_{a5\%}$ and $Sd = SR_X S_{d5\%}$ where $SR_X = SR_A$ if $T \leq T_s$, and $SR_X = SR_V$ if $T > T_s$.*
10. *Draw a line connecting the Sa, Sd points plotted in Step 9. The intersection of this line with the capacity spectrum is the demand displacement.*

8.2.2.1.4 Calculating Performance Point Using Procedure C. This procedure has been developed to provide a graphical solution using hand methods. It has been found to often be reasonably close to the performance point on the first try. The following steps are involved:

1. Develop the 5 percent damped response spectrum appropriate for the site using the procedures provided in Chapter 4.
2. Draw the 5 percent damped response spectrum and draw a family of reduced spectra on the same chart, as illustrated in Figure 8-34. It is convenient if the spectra plotted correspond to effective damping values (β_{eff}) ranging from 5 percent to the maximum value allowed for the building's structural behavior type. The maximum β_{eff} for Type A construction is 40 percent, Type B construction is 29 percent and Type C construction is 20 percent.
3. Transform the capacity curve into a capacity spectrum as described in Section 8.2.2.1.1, using equations 8-1, 8-2, 8-3 and 8-4, and plot it on the same chart as the family of demand spectra, as illustrated in Figure 8-35.
4. Develop a bilinear representation of the capacity spectrum as described in Section 8.2.2.1.1 and illustrated in Figure 8-9. Select the initial point a_{pi}, d_{pi} at the furthest point out on the capacity spectrum or at the intersection with the 5 percent damped spectrum, whichever is less. A displacement slightly larger than that calculated using the equal displacement approximation (say 1.5 times larger) may also be a reasonable estimate for the initial d_{pi}. See Figure 8-36 for an illustration of this step.
5. Determine the ratios d_{pi}/d_y and $[(a_{pi}/a_y) - 1]/[(d_{pi}/d_y) - 1]$. Note that the second term is the ratio of the post yield stiffness to the initial stiffness.

Commentary: Figure 8-37 provides some examples of the physical significance of the ratios d_{pi}/d_y and $[(a_{pi}/a_y) - 1]/[(d_{pi}/d_y) - 1]$. The figure shows example bilinear representations of capacity spectra along with the corresponding ratios.

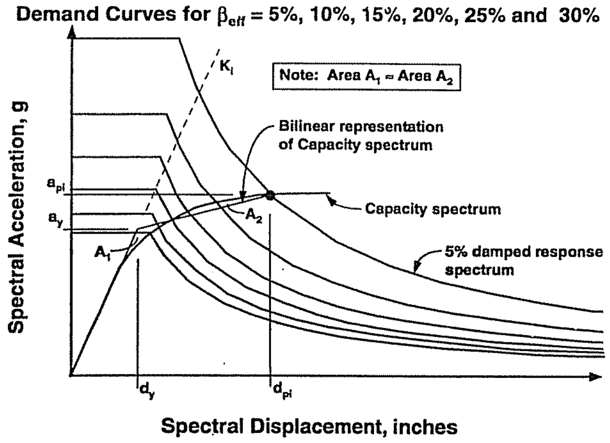


Figure 8-36. Capacity Spectra Procedure "C" After Step 4

6. Based on the ratios obtained in step 5, enter either Table 8-5, 8-6, or 8-7, depending on the building's structural behavior type, (see Table 8-4 for definition of structural behavior types) and find the effective damping value, β_{eff}.

   Commentary: The β_{eff} term can also be calculated using the formula given in equation 8-8. However, for the purposes of this graphical procedure, it may be easier to calculate β_{eff} using the above tables, which are based on the slopes of the two legs of the bilinear representation of the capacity spectrum. The values given in Tables 8-5, 8-6 and 8-7 are derived from equation 8-8. The equation and the tables give the same results.
7. Refer to Figure 8-38. Extend the initial stiffness line, labeled Line 1 in the figure, up to intersect the 5 percent damped curve. Also, draw a line, labeled Line 2 in the figure, from the origin to point a_{pi}, d_{pi}).
8. Refer to Figure 8-39. Draw a line, labeled Line 3 in the figure, from the intersection point of Line 1 and the 5 percent damped response spectrum to the intersection point of Line 2 and the reduced spectrum which corresponds to the β_{eff} determined in step 6. Note that the figure is drawn for a β_{eff} of approximately 24 percent.

Table 8-5. Effective Damping, β_{eff}, in percent—Structural Behavior Type A

	Slope Ratio: $[(a_{pi}/a_y) - 1]/[(d_{pi}/d_y) - 1]$						
d_{pi}/d_y	0.5	0.4	0.3	0.2	0.1	0.05	0
10	10	12	16	21	30	37	40
8	11	14	18	23	31	37	40
6	13	16	20	25	33	37	40
4	16	19	23	28	34	37	40
3	16	19	23	27	33	36	39
2	16	19	22	25	29	31	33
1.5	13	16	18	20	23	24	24
1.25	11	12	13	15	16	17	18

Table 8-6. Effective Damping, β_{eff}, in percent—Structural Behavior Type B

	Slope Ratio: $[(api/ay) - 1]/[(dpi/dy) - 1]$						
d_{pi}/d_y	0.5	0.4	0.3	0.2	0.1	0.05	0
10	9	10	12	16	23	27	29
8	9	11	13	17	24	27	29
6	10	12	15	19	25	27	29
4	11	14	17	21	25	27	29
3	12	14	17	21	25	27	29
2	12	14	16	19	22	24	25
1.5	11	12	14	15	17	18	18
1.25	9	10	10	11	12	13	13

Table 8-7. Effective Damping, β_{eff}, in percent—Structural Behavior Type C

	Slope Ratio: $[(a_{pi}/a_y) - 1]/[(d_{pi}/d_y) - 1]$						
d_{pi}/d_y	0.5	0.4	0.3	0.2	0.1	0.05	0
10	7	7	9	10	14	17	20
8	7	8	9	11	15	18	20
6	7	9	10	12	16	18	20
4	8	9	11	13	16	18	20
3	9	10	11	13	16	17	19
2	9	10	11	12	14	15	16
1.5	8	9	9	10	11	11	11
1.25	7	7	8	8	9	9	9

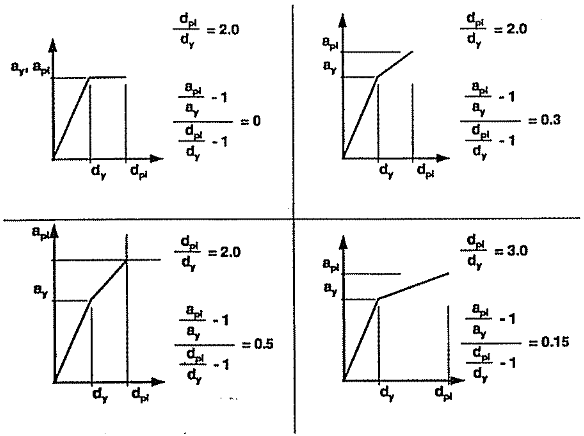


Figure 8-37. Example Slope Ratios

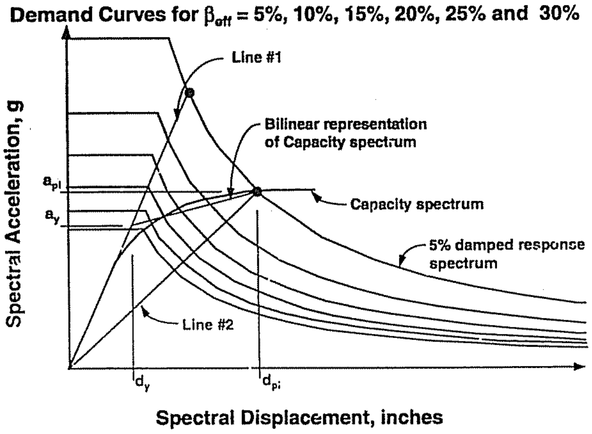


Figure 8-38. Capacity Spectrum Procedure C After Step 7

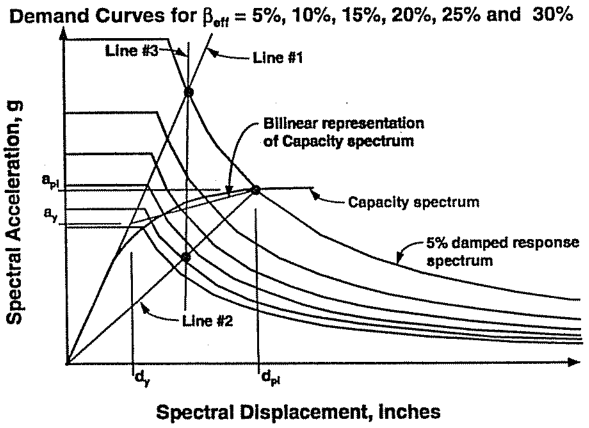


Figure 8-39. Capacity Spectrum Procedure C After Step 8

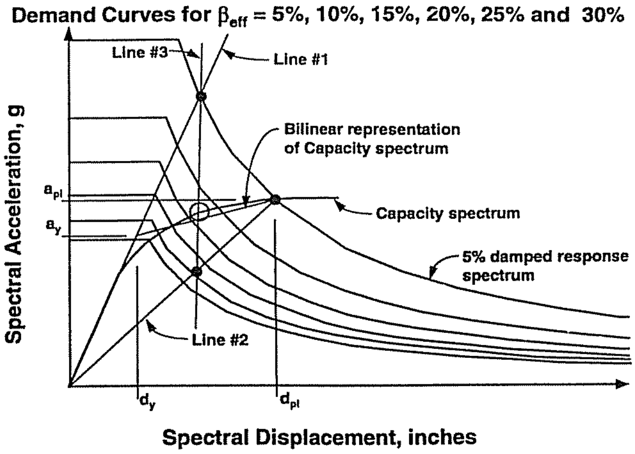


Figure 8-40. Capacity Spectrum Procedure C After Step 9

9. Refer to Figure 8-40. The point where Line 3 intersects the capacity spectrum is taken as the estimated performance point a_{p2}, d_{p2} point.

   Commentary: *In the case of a composite "sawtooth" capacity spectrum, constructed from a family of capacity spectrum curves, the bilinear representation should be based on the individual capacity spectrum curve that makes up the portion of the composite capacity spectrum where Line 3 intersects with the composite capacity spectrum (i.e., the point a_{pi}, d_{pi}).*

10. If displacement d_{p2} is within ±5 percent of displacement d_{p1}, then the point a_{p2}, d_{p2} is the performance point (or in more general terms, if displacement $d_{p(i+1)}$ is within ±5 percent of displacement d_{pi}, then the point $a_{p(i+1)}$, $d_{p(i+1)}$ is the performance point. If the displacements are not within the specified tolerance, then proceed to step 11.

    Commentary: *When the exact performance point is critical for the acceptability, then the ±5 percent tolerance should be adhered to, and additional iterations should be performed. If the exact performance point is not critical for the acceptability, then additional iterations are probably*

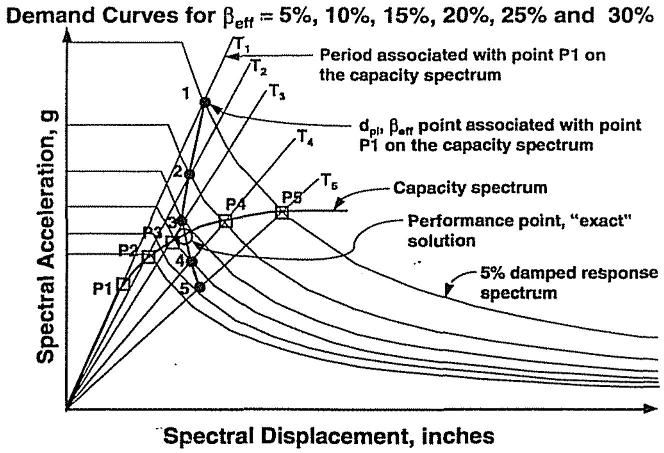


Figure 8-41. "Exact" Capacity Spectrum Solution

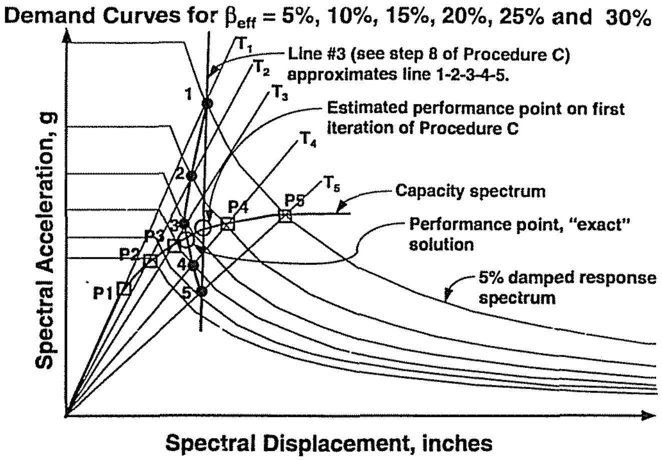


Figure 8-42. Approximation Used in Capacity Spectrum Procedure C

unnecessary. An example of when the exact performance point is not critical for the acceptability is if the current estimate of the performance point is well below the assumed acceptable performance for the structure.

11. Repeat the process starting at step 4, incrementing i by 1. Thus in the second iteration, Line 2 is drawn from the origin to point a_{p2}, d_{p2}.

Commentary: Procedure C is probably at first glance the least transparent application of the three capacity spectrum procedures presented. It can best be understood as follows. Given a capacity spectrum and a earthquake ground motion represented by a 5% damped spectrum, for any point on the capacity spectrum, a_{pi}, d_{pi}, *the corresponding* β_{eff} *can be calculated using equation 8-8, and the associated point* d_{pi}, β_{eff} *can be plotted. If the points* a_{pi}, d_{pi} *and* d_{pi}, β_{eff} *are the same, that is, if the point* d_{pi}, β_{eff} *happens to fall on the capacity spectrum, then the solution has been found. Note that this is essentially Procedure B without the simplifying assumption that point* a_y, d_y *and the post-yield slope must remain constant.*

Suppose that unlike Procedure B, no simplifying assumptions are made and the bilinear representation of the capacity spectrum changes for each point chosen on the post yield portion of the capacity spectrum. Then imagine plotting d_{pi}, β_{eff} *points associated with each point on the post-yield portion of the capacity spectrum. Each of these points would have a different bilinear representation of the capacity spectrum. The result would be a "banana" shaped curve as shown in Figure 8-41.*

The intersection of the "banana" shaped curve with the capacity spectrum is the "exact" performance point (no approximations have been made) and the displacement at this point is the demand displacement. Procedure C simplifies the above described method by replacing the "banana" shaped curve with a straight line drawn between its two end points, as illustrated in Figure 8-42, thus approximating the "exact" solution.

Note that after a few iterations, the approximate Procedure C solution should converge to the "exact solution," and that in some cases, the approximate solution may well be close enough to the "exact" solution in one or at most two iterations.

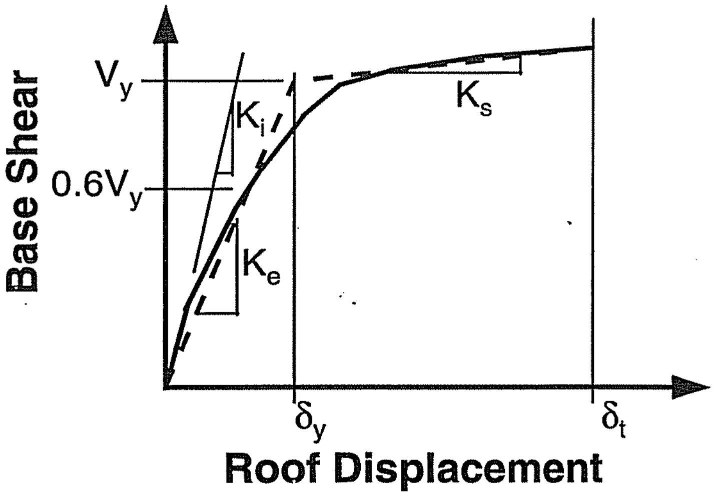


Figure 8-43. Bilinear Representation of Capacity Curve for Displacement Coefficient Method

8.2.2.2 Calculating Demand Displacement Using the Displacement Coefficient Method

The displacement coefficient method provides a direct numerical process for calculating the displacement demand. It does not require converting the capacity curve to spectral coordinates. The following step-by-step process is excerpted from the FEMA 273 Guidelines (ATC 1996a). There are some minor differences in terminology between this excerpt and the remainder of this document. For example, this excerpt refers to the target displacement, which is the same as the performance point in the rest of this document. It also refers to a performance objective of collapse prevention, which is the same as structural stability in the rest of this document.

The provisions included in this excerpt are limited in application to buildings that are regular and do not have adverse torsional or multimode effects. If the engineer uses this method for any structure that does not meet the limitations described above, then the full source document should be used.

Before applying this method, the user is encouraged to review the current version of FEMA 273 to determine if any of the criteria described below have been updated.

1. Construct a bilinear representation of the capacity curve as follows (refer to Figure 8-43):
    - Draw the post-elastic stiffness, K_s, by judgment to represent an average stiffness in the range in which the structure strength has leveled off.
    - Draw the effective elastic stiffness, K_e, by constructing a secant line passing through the point on the capacity curve corresponding to a base shear of $0.6V_y$, where V_y is defined by the intersection of the K_e and K_s lines.

    Commentary: *The above process requires some trial and error effort because the value for V_y is not known until after the K_e line is drawn. Thus a trial K_e line should be drawn, a*

Table 8-8. Values for Modification Factor C_0

Number of Stories	Modification Factor [1]
1	1.0
2	1.2
3	1.3
5	1.4
10+	1.5

1. Linear interpolation should be used to calculate intermediate values.

Table 8-9. Values for Modification Factor C_2

	$T = 0.1$ second		$T \geq T_0$ second	
Structural Performance Level	Framing Type 1[1]	Framing Type 2[2]	Framing Type 1[1]	Framing Type 2[2]
Immediate Occupancy	1.0	1.0	1.0	1.0
Life Safety	1.3	1.0	1.1	1.0
Collapse Prevention	1.5	1.0	1.2	1.0

1. Structures in which more than 30 percent of the shear at any level is resisted by components or elements whose strength and stiffness may deteriorate during the design earthquake. Such elements 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.
2. All frames not assigned to Framing Type 1.

V_y value defined, and then the point where the K_e line crosses the capacity curve should be checked to see if it is equal to $0.6V_y$. If the crossing point is not equal to $0.6V_y$, then a new K_e should be drawn and the process should be repeated.

Note that the bilinear curve constructed for the displacement coefficient method will generally be different from one constructed for the capacity spectrum method.

2. Calculate the effective fundamental period (T_e) as:

$$T_e = T_i\sqrt{\frac{K_i}{K_e}} \qquad (8\text{-}16)$$

where:

T_i = elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis.

K_i = elastic lateral stiffness of the building in the direction under consideration (refer to Figure 8-43).

K_e = effective lateral stiffness of the building in the direction under consideration (refer to Figure 8-43).

3. Calculate the target displacement, (δ_t) as:

$$\delta_t = C_0C_1C_2C_3S_a\frac{T_e^{\,2}}{4\pi^2} \qquad (8\text{-}17)$$

where:

T_e = effective fundamental period as calculated in step 2 above.

C_0 = modification factor to relate spectral displacement and likely building roof displacement; estimates for C_0 can be calculated using either:

- The first modal participation factor at the roof level.
- The modal participation factor at the roof level calculated using a shape vector corresponding to the deflected shape of the building at the target displacement.
- The appropriate value from Table 8-8.

C_1 = modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

= 1.0 for $T_e \geq T_0$

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

C_1 need not exceed 2.0 for $T_e < 0.1$ second

T_0 = a 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. See the FEMA Guidelines (ATC 1996a) Commentary for information on calculating T_0 for a site specific spectrum.

R = ratio of inelastic strength demand to calculated yield strength coefficient calculated as follows:

$$R = \frac{S_a/g}{V_y/W} \cdot \frac{1}{C_0} \qquad (8\text{-}18)$$

C_2 = modification factor to represent the effect of hysteresis shape on the maximum displacement response. Values of C_2 for different framing systems and performance levels are listed in Table 8-9. Linear interpolation shall be used to estimate values of C_2 for intermediate values of T_e.

C_3 = modification factor to represent increased displacements due to second-order effects. For buildings with positive post-yield stiffness, C_3 shall be set equal to 1.0. For buildings with negative post-yield stiffness, C_3 shall be calculated as

$$C_3 = 1 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \qquad (8\text{-}19)$$

Where R and T_e are defined above and α is the ratio of post-yield stiffness to elastic stiffness when the nonlinear force-displacement relation is characterized by a bilinear relation.

S_a = response spectrum acceleration as determined from Section 4.4.3.3, at the effective fundamental period of the building.

V_y = Yield strength calculated using the capacity curve, where the capacity curve is characterized by a bilinear relation. See Figure 8-43.

W = Total dead load and anticipated live load as indicated below.

- In storage and warehouse occupancies, a minimum of 25 percent 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 — see the *NEHRP Recommended Provisions* (BSSC 1995).
- The total weight of permanent equipment and furnishings.

8.2.3 Step By Step Procedures Checking Performance at the Expected Maximum Displacement

The following steps should be followed in the performance check:

1. For global building response verify the following:
    - The lateral force resistance has not degraded by more than 20 percent of the peak resistance
    - The lateral drifts satisfy the limits given in Table 11-2
2. Identify and classify the different elements in the building. Any of the following element types may be present: beam-column frames, slab-column frames, solid walls, coupled walls, perforated walls, punched walls, floor diaphragms and foundations.
3. Identify all primary and secondary components, as defined in Chapter 9. This classification is needed for the deformation check in step 5.
4. For each element, use the guidelines in the appropriate subsection of Section 11.4 to identify the critical components and actions to be checked. Note that there is a separate subsection in Section 11.4 for each of the element types listed in step 2 above.

5. The strength and deformation demands at the structure's performance point shall be equal to or less than the capacities given in Chapter 11 considering all co-existing forces acting with the demand spectrum.

   Commentary: As indicated in Chapter 9, no load factors are applied to gravity loads.
6. The performance of structural elements not carrying vertical load shall be reviewed for acceptability for the specified performance level.
7. Nonstructural elements shall be checked for acceptability for the specified performance level.

8.2.4 Other Considerations

Primary and Secondary Elements: To apply the nonlinear static procedures presented, it is important to understand the distinction between primary and secondary members. The engineer should carefully read and understand Sections 9.3.1 and 11.4.2.1 prior to developing the capacity curve.

Torsional Considerations: For buildings that are non-symmetric about a vertical plane parallel to the design lateral forces, the effects of torsion should be included in the development of the pushover curve. If a three dimensional model is used to capture the torsional effects, then the static lateral forces should be applied at the center of mass of each floor, and the displacements plotted on the capacity curve should be at the center of mass of the roof.

Two dimensional modeling and analysis may be used if the torsional effects are sufficiently small such that the maximum displacement at any point on the floor is less than 120 percent of the displacement at the corresponding center of mass. If the maximum displacement exceeds 120 percent of the displacement at the center of mass, then three dimensional analysis is required. For two dimensional analysis, an acceptable approach to considering the effects of torsion when developing the capacity curve is to identify the ratio of maximum displacements to the center of mass displacement at each floor level using linear static analysis or linear dynamic analysis of a three dimensional model, and to then increase the displacement at the center of mass of the roof, at each point on the capacity curve, by the maximum of these ratios.

Multimode Considerations: For structures with long fundamental periods , higher mode effects may be more critical on some components of the structure than the effects of the fundamental mode. Pushover analyses may be done for additional mode shapes. For example, force distributions are applied to deform the building into the second and the third translational mode shapes. Yield patterns will be substantially different than those obtained for the first mode shape. The V versus Δ_{roof} values for the higher modes are converted to S_a versus S_d curves using the higher mode participation factors and effective modal weights. These curves are plotted on the ADRS format and the demands on each of the modes can be determined. Each component of the structure is then evaluated for the different modes.

Commentary: Commentary in Section 8.2.1 indicates that pushover analyses using the fundamental mode shape are generally valid for fundamental periods of vibration up to about one second. Thus the engineer may want to consider using the above described process for structures with fundamental modes exceeding one second.

8.3 Illustrative Example

The example building is a seven-story 66-foot tall reinforced concrete frame structure in seismic zone 4. The weight, W, is 10,540 kips. Only one direction of loading is considered.

The example includes a brief discussion of nonlinear static (pushover) analysis results, followed by demand checks using the various procedures described in Section 8.2.2. The demand checks use the Design Earthquake described in Chapter 4.

Table 8-10. Modal Properties for 7-Story Building

Mode		1	2	3	4	5
Period, T (seconds)		0.880	0.288	0.164	0.106	0.073
Period ratio, T/T_m		1.00	3.05	5.37	8.30	12.05
Participation Factor, PF_{Rm}, at Roof		1.31	-0.47	0.24	-0.11	0.05
Effective mass coefficient, α_m		0.828	0.120	0.038	0.010	0.000
Mode Shape	Roof	1.000	1.000	1.000	1.000	1.000
at	7	0.938	0.550	-0.059	-0.852	-1.749
Story Levels	6	0.839	-0.056	-0.942	-1.080	0.194
(normalized)	5	0.703	-0.631	-0.921	0.526	1.674
ϕ_{lm}	4	0.535	-0.961	-0.034	1.259	-1.068
	3	0.351	-0.933	0.883	-0.088	-1.139
	2	0.188	-0.625	0.990	-1.150	1.310
	1	0	0	0	0	0

8.3.1 Building Structural Dynamic Characteristics

This section provides the modal properties for the example building and demonstrates a modal analysis of the building. The modal analysis provides more information than is actually required to perform the pushover analysis, obtain the capacity curve, and convert it to the capacity spectrum. The complete modal analysis is included here to provide background and to provide a clearer picture of the relationship between modal analysis and the pushover analysis. The equations used in the modal analysis are given in Section 8.5.2. The modal properties actually needed to create the capacity curve (with a pushover analysis) are the masses at each level and the first mode shape. The information needed to convert the capacity curve to the capacity spectrum is the modal participation factor and the modal mass coefficient for the first natural mode.

Structural dynamic properties are given in Table 8-10. These values were obtained from a computer aided analysis of the example structure modeled in accordance with Chapter 9.

Table 8-10 shows the periods, participation factors, effective mass coefficients and mode shapes for the first five modes of vibration parallel to the transverse axis of the building. The mode shapes have been normalized so that roof values equal 1.0. The period ratios (T/T_m), the first mode period divided by the higher mode period, indicate common mode shape characteristics. Ratios of 1,3,5 to 1,4,6 for the first three modes are typical for regular buildings (i.e., no significant vertical irregularities).

Table 8-11 is an extension of Table 8-10, showing the results of a modal analysis for a response spectrum with S_a=0.276g for the first mode period of 0.88 sec and S_a=0.500g at the plateau of the response spectrum (i.e., constant acceleration region) for the second and third mode periods (i.e., C_V=0.24 and C_A=0.20). The mode shapes (ϕ) are obtained directly from the computer printout. The values of ϕ have been normalized such that the sum of story mass (w/g) times ϕ^2 is equal to 1.0 (i.e., $\Sigma m\phi^2=1.0$). In the computer printout, the participation factor of the first mode (PF_1) is equal to 16.46. When this value is multiplied by ϕ_{roof} (i.e., 0.0794), the roof first mode participation factor is 1.31. The table shows the story accelerations, a, for each mode by use of Equation 8-22. Note that the roof acceleration multiplied by the ϕ-factors in Table 8-10 ($\phi=1.0$ at roof) gives the same values for story accelerations given in Table 8-11. The modal story accelerations combined by the square root of the sum of the squares (SRSS) are shown on the last column of Table 8-11.

Table 8-11. Modal analysis of 7-story building for story accelerations.

		Mode 1				Mode 2				Mode 3				
		T_1 = 0.880 sec				T_2 = 0.288 sec				T_3 = 0.164 sec				
		S_a = 0.276 g				S_a = 0.500 g				S_a = 0.500 g				
	W_x/g	ϕ_1	(W/g) X ϕ_1	(W/g) X ϕ_1^2	a_1 (g)	ϕ_2	(W/g) X ϕ_2	(W/g) X ϕ_2^2	a_2 (g)	ϕ_3	(W/g) X ϕ_3	(W/g) X ϕ_3^2	a_3 (g)	a SRSS
Roof	43.78	.0794	3.48	.276	.362	0.0747	3.27	0.244	-0.235	0.0684	2.99	0.205	0.120	.448
7	45.34	.0745	3.38	.252	.340	0.0411	1.86	0.076	-0.129	-0.0040	-0.18	0.001	-0.007	.364
6	45.34	.0666	3.02	.201	.304	-0.0042	-0.19	0.001	0.013	-0.0644	-2.92	0.188	-0.113	.325
5	45.34	.0558	2.53	.141	.254	-0.0471	-2.14	0.101	0.148	-0.0630	-2.86	0.180	-0.111	.314
4	45.34	.0425	1.93	.082	.194	-0.0718	-3.26	0.234	0.226	-0.0023	-0.10	0.000	-0.004	.298
3	45.34	.0279	1.27	.035	.127	-0.0697	-3.16	0.220	0.219	0.0604	2.74	0.166	0.106	.275
2	56.83	.0149	0.85	.013	.068	-0.0467	-2.65	0.124	0.147	0.0677	3.85	0.261	0.119	.201
1	--	0	0	0	0	0	0	0	0	0	0	0	0	0
Σ	327.31		16.46	1.000			-6.27	1.000			3.52	1.001		
PF_{RF}	eq 8-20a	16.46 x 0.0794/1.000 = 1.31				-6.27 x 0.0747/1.000 = -0.47				3.52 x 0.0684/1.000 = 0.24				
α_m	eq 8-21	16.46^2/(327.31 x 1.000) = 0.828				6.27^2/(327.31 x 1.000) = 0.120				3.52^2/(327.31 x 1.000) = 0.038				
a_{RF}	eq 8-22	1.31 x 0.276 = 0.362g				-0.47 x 0.500 = 0.235g				0.24 x 0.500 = 0.120g				.448g
V_m	eq 8-24	0.828 x 0.276 x 10.539 = 2408 kips				0.12 x 0.500 x 10.539 =632 kips				0.038 x 0.500 x 10.539 =200 kips				2498 kips
V_m/W		0.229				0.060				0.019				0.237

W = 10,540 kips

Table 8-12 shows the process for calculating story forces, shears, overturning moments, and displacements. Interstory displacements for the first mode are obtained by taking differences between story displacements. The process for determining story forces is similar to that used in the building code procedure when distributing the base shear as story forces, except that wϕ/Σwϕ is used instead of wh/Σwh. The story displacements can be calculated directly in the table (using equation 8-26, in Section 8.5.2) because the stiffness characteristics had been incorporated into the computer analysis and is represented by the period T.

Table 8-13 summarizes the results for the first three modes of vibration. The first mode values are the same as in Table 8-12. The last column shows the interstory drift ratios (i.e., interstory displacement divided by height of the story). Graphical representations of Table 8-13 are in Figure 8-44.

8.3.2 Capacity Curve

Figure 8-45 shows the capacity curve resulting from a pushover analysis of the example building. Forces were applied in proportion to the first mode shape. The initial set of forces are shown in column 7 of Table 8-12 for a base shear of 2408 kips. The analysis showed that some beams may require significant post-yield deformation capacity, because elastic moments exceeded beam strengths.

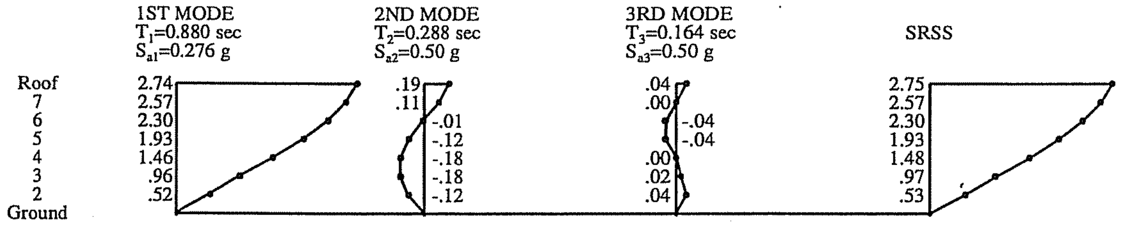
Table 8-12. First Mode Forces for 7-Story Building

(1) Story	(2) ϕ	(3) h ft	(4)) Δh ft	(5) w kips	(6) $w\phi/\Sigma w\phi$	(7) F kips V_1x(6)	(8) V kips Σ (7)	(9) ΔOTM k-ft (4)x(8)	(10) OTM k-ft Σ (9)	(11) Accel g (7) ÷ (5)	(12) δ ft	(13) $\Delta\delta$ ft
Roof	.0794	65.7		1410	.211	508			0	.360	.228	
			8.7				508	4,420				.014
7	.0745	57.0		1460	.205	495			4,420	.338	.214	
			8.7				1,002	8,717				.022
6	.0666	48.3		1460	.184	443			13,137	.303	.192	
			8.7				1,445	12,572				.031
5	.0558	39.6		1460	.154	371			25,709	.254	.161	
			8.7				1,816	15,799				.039
4	.0425	30.9		1460	.117	282			41,508	.193	.122	
			8.7				2,098	18,253				.042
3	.0279	22.2		1460	.077	185			59,761	.127	.080	
			8.7				2,283	19,862				.037
2	.0149	13.5		1830	.052	125			79,623	.068	.043	
			13.5				2,408	32,508				.043
Ground	0	0		0	0	0			112,131	0	0	
Σ				10,540	1.000	2408^k		112,131				

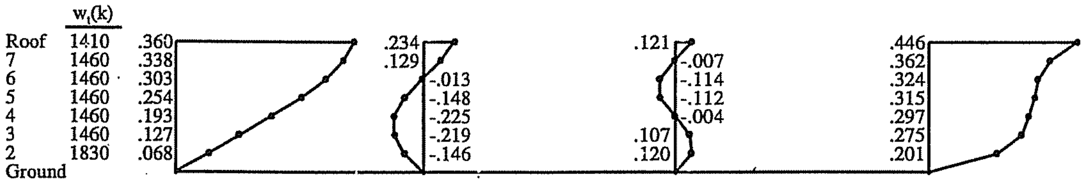
Table 8-13. Summary of Forces for 7-Story Building

Story		Forces				Shears (kips)				OTM (k-ft)			
Level	Wt (kips)	1	2	3	SRSS	1	2	3	SRSS	1	2	3	SRSS
Roof	1410	508	-330	170	629	508	-330	170	629	0	0	0	0
7	1460	494	-188	-10	529	1,002	-518	160	1,139	4,420	-2,871	1,479	5,474
6	1460	443	19	-166	473	1,445	-499	-6	1,529	13,137	-7,378	2,871	15,338
5	1460	371	216	-163	459	1,816	-283	-169	1,846	25,709	-11,719	2,819	28,394
4	1460	282	329	-6	433	2,098	46	-175	2,106	41,508	-14,181	1,349	43,884
3	1460	185	319	156	400	2,283	365	-19	2,312	59,761	-13,781	-174	61,330
2	1830	125	267	219	367	2,408	632	200	2,498	79,623	-10,605	-339	80,327
Ground		0	0	0	0					112,131	-2,073	2,361	112,175

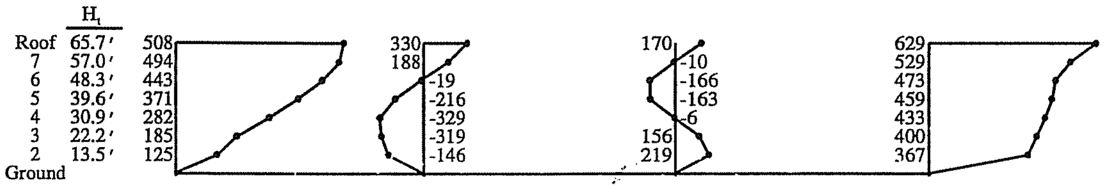
	Acceleration (g)				Displacement (ft)				Interstory Drift(ft)				
Story	1	2	3	SRSS	1	2	3	SRSS	1	2	3	SRSS	$\Delta\delta/\Delta h$
Roof	0.360	-0.234	0.121	0.446	0.228	-0.016	0.003	0.229	0.014	0.007	0.003	0.016	0.0018
7	0.338	-0.129	-0.007	0.362	0.214	-0.009	0.000	0.214	0.022	0.010	0.003	0.024	0.0028
6	0.303	0.013	-0.114	0.324	0.192	0.001	-0.003	0.192	0.031	0.009	0.000	0.032	0.0037
5	0.254	0.148	-0.112	0.315	0.161	0.010	-0.003	0.161	0.039	0.005	0.003	0.039	0.0045
4	0.193	0.225	-0.004	0.297	0.122	0.015	0.000	0.123	0.042	0.000	0.002	0.042	0.0048
3	0.127	0.219	0.107	0.275	0.080	0.015	0.002	0.081	0.037	0.005	0.001	0.037	0.0043
2	0.068	0.146	0.120	0.201	0.043	0.010	0.003	0.044	0.043	0.010	0.003	0.044	0.0033
Ground	0	0	0	0	0	0	0	0					



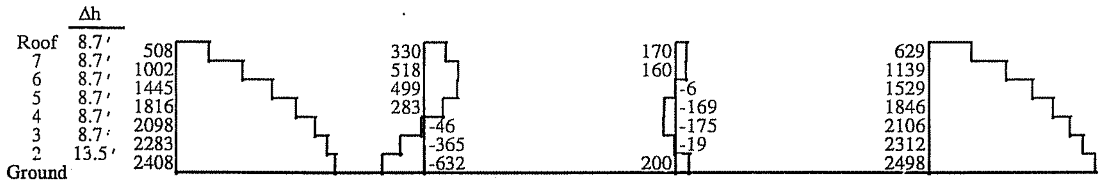
(a) MODAL LATERAL DISPLACEMENTS (inches)



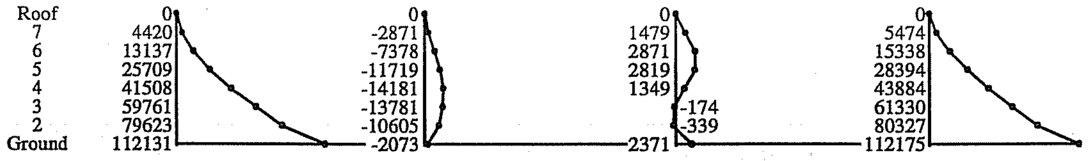
(b) MODAL STORY ACCELERATIONS (g's)



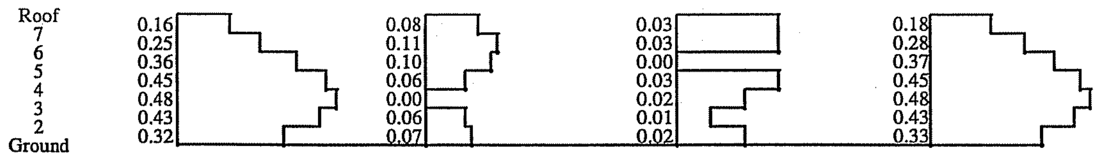
(c) MODAL STORY FORCES (kips)



(d) MODAL STORY SHEARS (kips)



(e) MODAL STORY OVERTURNING MOMENTS(kip-ft)



(f) INTER STORY DRIFT RATIOS (in %)

Figure 8-44. Story Forces and Displacements for Seven-Story Example

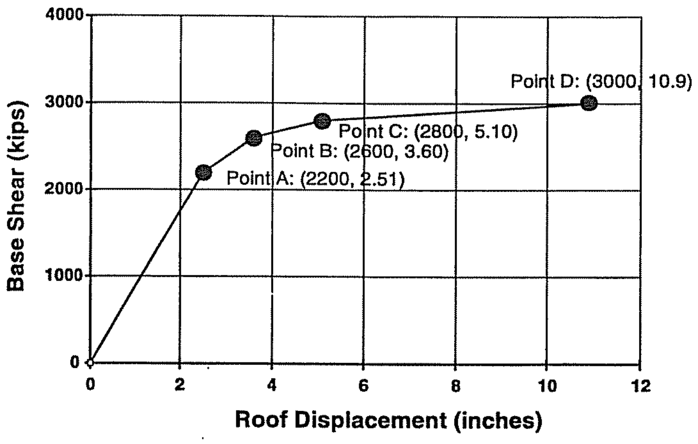


Figure 8-45. Capacity Curve

The forces were scaled to a base shear of 2200 kips to establish the first point of yielding (point A). The roof displacement of 2.74 inches (0.228 ft x 12, Table 8-12) at 2408 kips was scaled to 2.51 inches at 2200 kips.

The mathematical model was modified to account for plastic hinging at a number of beams. The new model, represented by segment AB on Figure 8-20, took an additional 400 kips with an incremental displacement of 1.09 inches. Thus, point B was established at 2600 kips (2200 + 400) and 3.60 inches (2.51 + 1.09). The model was again modified and point C was established by an incremental force of 200 kips and 1.50 inches. A third modification to the model was made to push the building from point C to point D. An increment of 200 kips produced an increment of 5.7 inches. Interstory displacements are determined by superposition of the lateral story displacements of the sequential models.

At this point some interstory displacements were exceeding 2 inches in the 8'-8" story heights, giving interstory drift ratios exceeding 0.02. From Table 8-12 it can be seen that maximum interstory drifts are at 0.5 inches (i.e., 12 x .04 ft) for a roof displacement of 2.74 inches (0.228 ft). At 10.9 inches (point D) these interstory drifts would increase to 2 inches (i.e., 0.5 times 10.9/2.74 = 2.0). Because some stories have softened relative to others, the interstory displacements will exceed 2 inches at some stories. Point D, which corresponds to 2% drift, the deformation limit given in Table 11-2 for the Life Safety Performance Level, appears to be a good stopping point for the pushover analysis and the capacity curve.

The capacity curve shown in Figure 8-45 can now be converted to a capacity spectrum curve for use in the ADRS format. The procedure is summarized in Table 8-14. Note that this conversion is only required when demand is determined by the Capacity Spectrum Method (see Sections 8.2.2 and 8.3.3).

The coordinates of points A, B, C, and D of Figure 8-45 are shown in Table 8-14 under columns V and δ_R. The weight W of the building is 10,540 kips (refer to Table 8-11). The base shear coefficients V/W are calculated by dividing V by 10,540. For the elastic mathematical model of the building (Point A), the roof participation factor, PF_{R1}, is 1.31 ($PF_{R1} = PF_1\phi_{roof,1}$) and the effective mass coefficient, α_1, is 0.828 (Table 8-10, mode 1).

Thus, S_a at Point A is 0.254g ($S_a = (V/W)/\alpha_1 = 0.209/0.828$ per equation 8-3) and S_d is 1.92 inches ($S_d = \delta_R/PF_{R1} = 2.51/1.31$ per equation 8-4). Note that the values for PF_{R1} and α_1 vary for points B, C, and D. These variations are due to changes in mode shapes caused by inelastic deformation of the structural system. These variations are less than 10 percent in this example and could be ignored (i.e., use the point A values for points B, C, and D). However, in the case of a borderline validation of performance, this variation could be a deciding factor.

Continuing with the procedure, the values of S_a and S_d are calculated for each point on the curve. The period T for point A is given in Table 8-10. The periods for points B, C, and D are calculated from S_d and S_a using Equation 8-29 in Section 8.5.2. The capacity spectrum curve is plotted in ADRS format in Figure 8-46. Periods

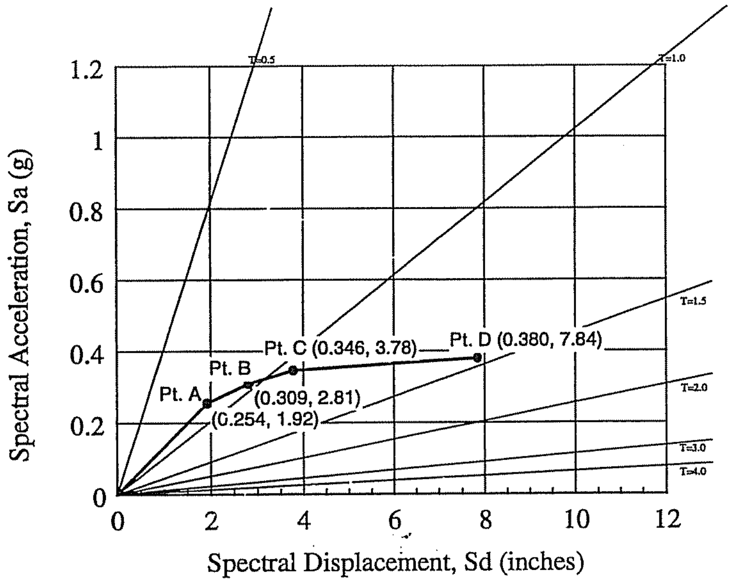


Figure 8-46. Capacity Spectrum Curve

Table 8-14. Conversion of V and δ_R to S_a and S_d

Point	V (kips)	δ_R (in)	V/W	PF_{R1}	α_1	S_a (g)	S_d (in)	T (sec)
A	2200	2.51	0.209	1.31	0.828	0.254	1.92	0.88
B	2600	3.60	0.247	1.28	0.800	0.309	2.81	0.96
C	2800	5.10	0.266	1.35	0.770	0.346	3.78	1.06
D	3000	10.90	0.285	1.39	0.750	0.380	7.84	1.45

*Note: PF's and α's change because the mode shape is changing as yielding occurs.

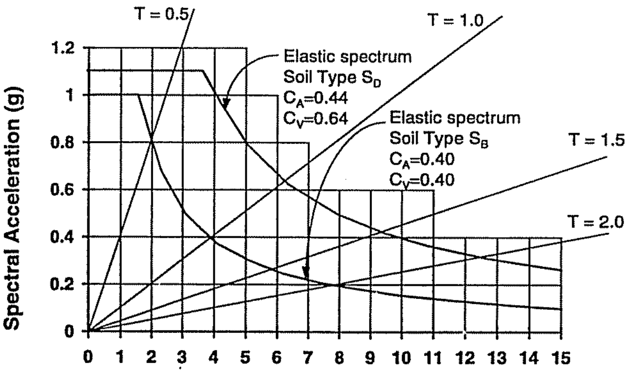


Figure 8-47. Example Building Elastic Response Spectra

can be estimated by interpolation between the radial period lines. This is a useful verification of the values of T in Table 8-14 (i.e., point C is between lines 1.0 and 1.5. Table 8-14 gives T=1.06 at point C).

8.3.3 Demand Check

8.3.3.1 Elastic (5 percent Damped) Response Spectra

This example assumes a demand represented by the Design Earthquake described in Chapter 4. In seismic zone 4, with no near-fault effects, ZEN is equal to 0.40. Two sites are considered to illustrate effects of different soil profile types on expected performance:

Type S_B: $C_A=0.40$, $C_V=0.40$
Type S_D: $C_A=0.44$, $C_V=0.64$

Figure 8-47 shows the 5 percent damped elastic response spectra for both soil types. Figure 8-48 shows the elastic spectra together with the capacity spectrum from Figure 8-46.

8.3.3.2 Preliminary Estimate of Demand Displacement

Figure 8-48 allows a preliminary demand check. The elastic spectrum for soil type S_B crosses the capacity spectrum near point C, indicating that some inelasticity will be demanded. (If the intersection had been to the left of point A, the elastic limit, it would have indicated elastic behavior without the need for inelastic reduction factors SR_A or SR_V.) The performance point(determined below) will be between point A and the intersection shown, perhaps around S_d = 3 to 4 inches. For soil type S_D, inelastic reduction will be needed to achieve a performance point to the left of point D, the limiting roof displacement discussed above. It may turn out that the required amount of inelasticity and effective damping will exceed limits discussed in Section 8.2.2.1.1. If so, reliable performance in the Design Earthquake can not be expected.

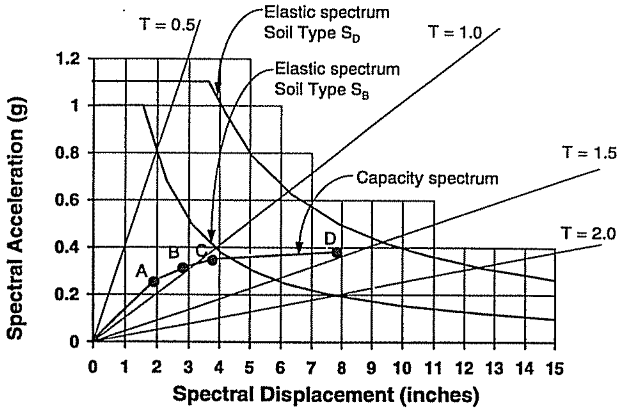


Figure 8-48. Example Building Capacity Spectrum Overlayed on Elastic Response Spectra

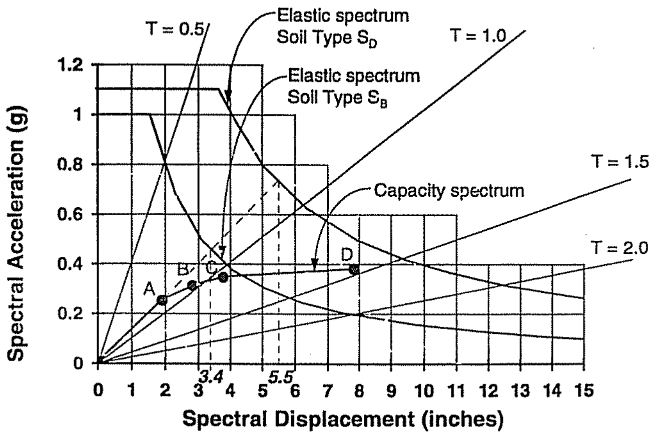


Figure 8-49. Equal Displacement Approximation for Example Building

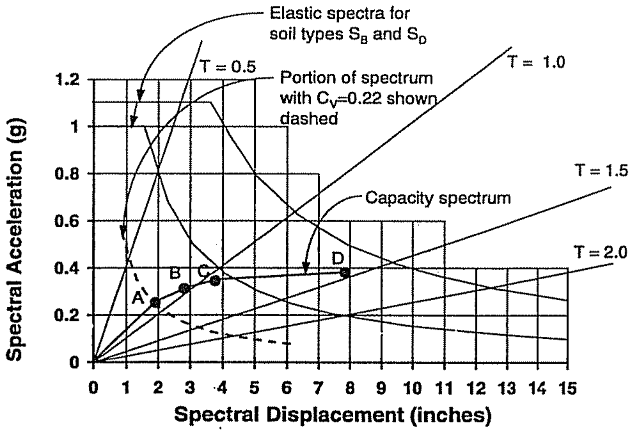


Figure 8-50. Spectrum With $C_V = 0.22$

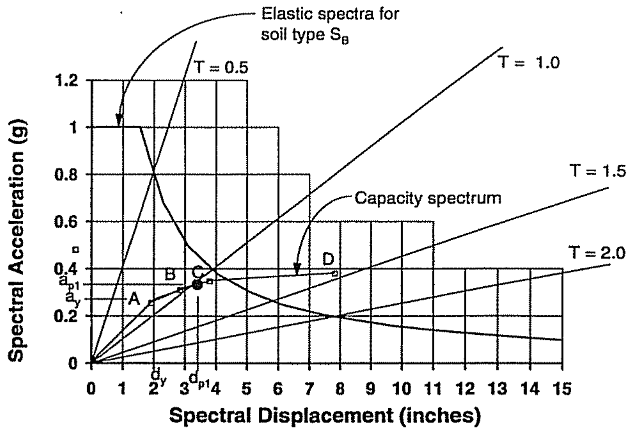


Figure 8-51. Initial a_{p1}, d_{p1} Point (Soil Type B)

Simple quantitative estimates of performance can be derived graphically from Figure 8-48 based on the equal displacement approximation. As shown in Figure 8-49, project the elastic portion of the capacity spectrum beyond point A until it intersects the elastic spectra. If the intersection is in the constant velocity region (i.e. not in the plateau), the displacement at the intersection can be taken as an estimate of the performance point displacement. As shown in the figure, an initial estimate of the spectral displacement is 3.4 and 5.5 inches for soil types S_B and S_D, respectively.

Figure 8-50 also shows part of an elastic response spectrum that intersects the capacity spectrum just at its elastic limit (point A). This hypothetical spectrum can be derived by trial and error; the one in Figure 8-50 happens to have a Cv equal to 0.22. This value can be used to estimate the maximum earthquake in which the structure is expected to remain elastic. With reference to Table 4.8, Cv = 0.22 corresponds to a ZEN value of 0.22 on soil type S_B and a ZEN value of about 0.1 on type S_D.

8.3.3.3 Performance Point Calculation by Capacity Spectrum Method—Procedure A

Procedure A is described in Section 8.2.2.1.2. This section demonstrates the procedure for soil type S_B first, and then repeats the demonstration for soil type S_D.

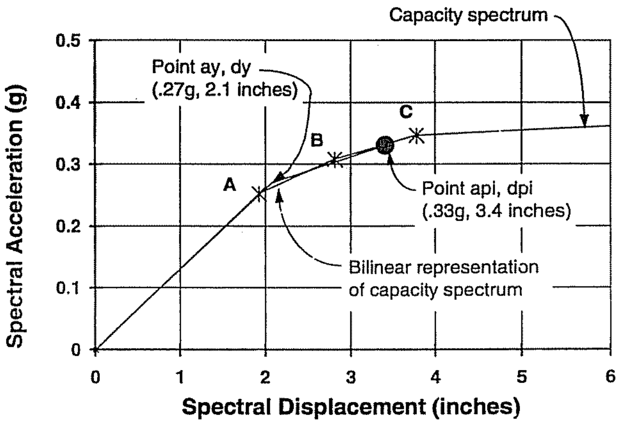


Figure 8-52. Bilinear Representation of Capacity Spectrum (Soil Type B)

With reference to the step-by-step procedure given there, steps 1 and 2 are already complete per sections 8.3.3.1 and 8.3.2 respectively. The next steps are to pick an initial a_{pi}, d_{pi} point, designated a_{p1}, d_{p1}, and to construct a bilinear representation of the capacity spectrum using that point, as illustrated in Figure 8-51 and blown up in Figure 8-52. The initial a_{p1}, d_{p1} point (0.33g, 3.4 inches) is chosen with d_{p1} equal to the displacement obtained using the equal displacement approximation, 3.4 inches for soil type S_B. The areas A_1 and A_2 (defined in Figure 8-9) are balanced by eye. The point a_y, d_y is read as (0.27g, 2.1 inches).

Thus with:

$a_{p1} = 0.33g$

$d_{p1} = 3.4$ inches

$a_y = 0.27g$

$d_y = 2.1$ inches

calculate

$$\beta_0 = \frac{63.7(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} =$$

$$\frac{63.7(.27 * 3.4 - 2.1 * .33)}{.33 * 3.4} = 12.8$$

and use this value to enter Table 8-1. This example frame building is assumed to be an Average Existing Building, as described in Table 8-4. It is assumed to be in seismic zone 4 with the seismic source that governs ground shaking at the site having a moment magnitude, M, of 7. As discussed in Section 4.5.2, such a building should be checked assuming long duration shaking. Thus, by Table 8-4, the example structural behavior is type C, and by Table 8-1, the κ factor is taken as 0.33 independent of effective damping.

Commentary: *Determination of behavior types and κ factors should be based on the expected behavior of primary elements; potential degrading of elements that can be categorized as secondary need not be considered.*

Calculate the effective damping, β_{eff}, using equation 8-8.

$$\beta_{eff} = \frac{63.7 * .33(.27 * 3.4 - 2.1 * .33)}{.33 * 3.4} + 5 = 9.2\%$$

Calculate the spectral reduction factor in the constant acceleration range, SR_A, using equation 8-9.

$$SR_A = \frac{3.21 - 0.68 \ln(9.2)}{2.12} = 0.80$$

Calculate the spectral reduction factor in the constant velocity range, SR_V, using equation 8-10.

$$SR_V = \frac{2.31 - 0.41 \ln(9.2)}{1.65} = 0.85$$

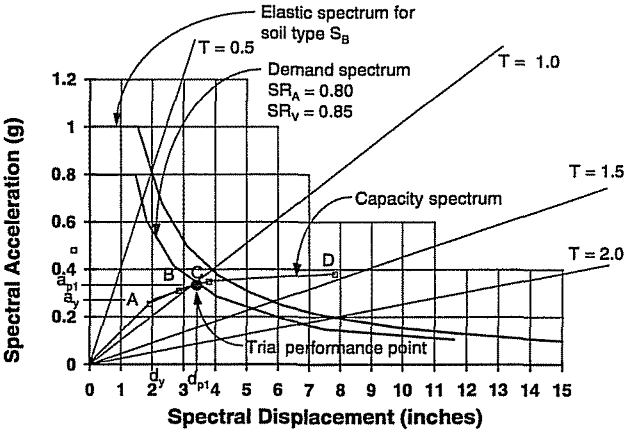


Figure 8-53. Reduced Spectrum Associated With Point a_{p1}, d_{p1} (Soil Type B)

Note that values of SR_A and SR_V similar to these could be interpolated from Table 8-3.

Calculate values needed to plot the demand spectrum associated with point a_{p1}, d_{p1}:

$S_a = 2.5 SR_A C_A = 2.5 * 0.80 * 0.40 = 0.80g$

$T_S = SR_V C_V / (2.5 SR_A C_A)$
$= 0.85 * 0.40 / (2.5 * 0.80 * 0.40)$
$= 0.425$ seconds

S_d at $T_S = S_a (T/2\pi)^2$
$= 0.80 * 386.4 * (0.425/2\pi)^2$
$= 1.41$ inches

Plot the demand spectrum associated with point a_{p1}, d_{p1} as shown in Figure 8-53. Referring to Figure 8-53, the demand spectrum intersects the capacity spectrum within ±5% of the trial point, a_{p1}, d_{p1}, and thus that point is the performance point. No iteration is required in this case.

Therefore, the demand displacement calculated for soil type B using Procedure A is 3.4 inches.

As shown in Figure 8-54, for soil type D, the initial a_{p1}, d_{p1} point (0.37g, 5.5 inches) is chosen with d_{p1} equal to the displacement obtained using the equal displacement approximation, 5.5 inches for soil type S_D. The areas A_1 and A_2 (defined in Figure 8-9) are again balanced by eye. The point a_y, d_y is read as (0.31g, 2.3 inches).

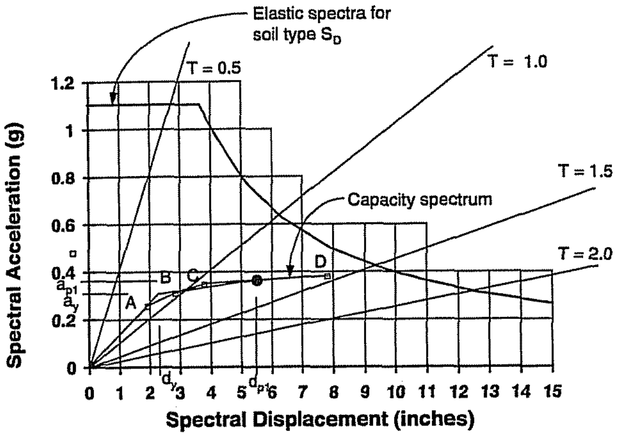


Figure 8-54. Initial a_{p1}, d_{p1} Point (Soil Type D)

The effective damping, β_{eff}, and the spectral reduction factors, SR_A and SR_V, are calculated as:

a_{p1} = 0.37g
d_{p1} = 5.5 inches
a_y = 0.31g
d_y = 2.3 inches

$$\beta_{eff} = \frac{63.7*.33(.31*5.5-2.3*.37)}{.37*5.5} + 5 = 13.8\%$$

$$SR_A = \frac{3.21 - 0.68\ln(13.8)}{2.12} = 0.67$$

$$SR_V = \frac{2.31 - 0.41\ln(13.8)}{1.65} = 0.75$$

$S_a = 2.5SR_AC_A = 2.5*0.67*0.44 = 0.74g$

$T_S = SR_VC_V/(2.5SR_AC_A)$
$= 0.75*0.64/(2.5*0.67*0.44)$
$= 0.65$ seconds

S_d at $T_S = S_a(T/2\pi)^2$
$= 0.74*386.4*(0.65/2\pi)^2$
$= 3.07$ inches

Plot the demand spectrum associated with point a_{p1}, d_{p1} for soil type D as shown in Figure 8-55.

As Figure 8-55 shows, the demand spectrum intersects the capacity spectrum at a displacement, d_i, of approximately 6.25 inches. As noted in Figure 8-22, for this displacement to be acceptable, $0.95d_{p1} \le d_i \le 1.05\,d_{p1}$, and since d_{p1} = 5.5 inches, $5.23 \le 6.25 \le 5.78$. Obviously, the 6.25 inch displacement is not acceptable because it is greater than 5.78 inches. Thus a second iteration is required.

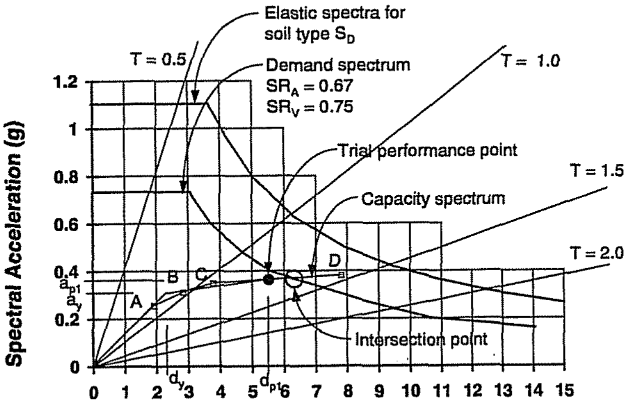


Figure 8-55. Reduced Spectrum Associated With Point a_{p1}, d_{p1} (Soil Type D)

A point a_{p2}, d_{p2} must be selected for the second iteration. One choice for that point could be the intersection point for the previous iteration, the point on the capacity spectrum with a spectral displacement of 6.25 inches. Choosing the intersection point from the previous iteration as the new a_{pi}, d_{pi} point would eventually lead to an acceptable solution. However, for this example, instead of choosing that point, some engineering judgment is applied, and a point halfway between point a_{p1}, d_{p1}, and the intersection point obtained in the first iteration is chosen. Thus, as shown in Figure 8-56, point a_{p2}, d_{p2} is chosen as (0.37g, 5.9 inches). The areas A_1 and A_2 (defined in Figure 8-9) are again balanced by eye. The point a_y, d_y is read as (0.31g, 2.3 inches).

The effective damping, β_{eff}, and the spectral reduction factors, SR_A and SR_V, are calculated as:

a_{p1} = 0.37g
d_{p1} = 5.9 inches
a_y = 0.31g
d_y = 2.3 inches

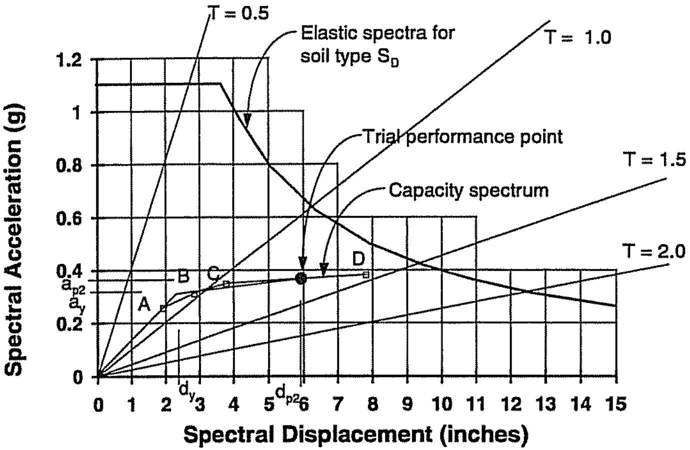


Figure 8-56. a_{p2}, d_{p2} Point (Soil Type D)

$$\beta_{eff} = \frac{63.7*.33(.31*5.9 - 2.3*.37)}{.37*5.9} + 5 = 14.4\%$$

$$SR_A = \frac{3.21 - 0.68\ln(14.4)}{2.12} = 0.66$$

$$SR_V = \frac{2.31 - 0.41\ln(14.4)}{1.65} = 0.74$$

$S_a = 2.5SR_AC_A = 2.5*0.66*0.44 = 0.73g$

$$\begin{aligned} T_S &= SR_VC_V/(2.5SR_AC_A) \\ &= 0.74*0.64/(2.5*0.66*0.44) \\ &= 0.65 \text{ seconds} \end{aligned}$$

$$\begin{aligned} S_d \text{ at } T_S &= S_a(T/2\pi)^2 \\ &= 0.73*386.4*(0.65/2\pi)^2 \\ &= 3.02 \text{ inches} \end{aligned}$$

Plot the demand spectrum associated with point a_{p2}, d_{p2} for soil type D as shown in Figure 8-57.

As the figure shows, the demand spectrum intersects the capacity spectrum essentially at the trial performance point a_{p2}, d_{p2} and thus the solution has been found. Therefore, the demand displacement calculated for soil type D using Procedure A is 5.9 inches.

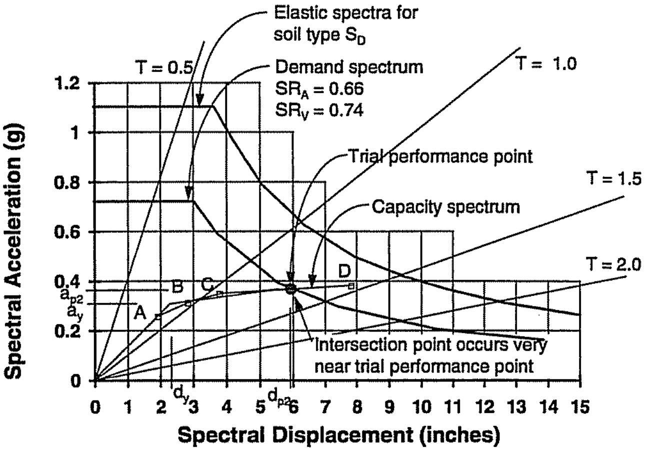


Figure 8-57. Reduced Spectrum Associated With Point a_{p2}, d_{p2} (Soil Type D)

Comparing the displacements obtained for the two different soil types, 3.4 inches for soil type S_B, and 5.9 inches for soil type S_D, it is clear that soil type can have a significant affect on building displacement, and consequently on structural performance.

8.3.3.4 Performance Point Calculation by Capacity Spectrum Method—Procedure B

The first steps in Procedure B are to develop and plot the elastic spectrum and a family of reduced spectra. Since this example building is assumed to have structural behavior type C, the effective damping β_{eff}, must be less than or equal to 20%. Thus, in addition to the elastic (5% damped) spectrum, spectra for 10%, 15% and 20% damping are developed.

The spectral reduction factors associated with those levels of damping are tabulated below. These factors can be obtained from equations 8-9 and 8-10 for any values of β_{eff}. The C_A and C_V factors for this example are:

Soil Type S_B: $C_A=0.40$, $C_V=0.40$
Soil Type S_D: $C_A=0.44$, $C_V=0.64$

The period where the spectral curve switches from constant acceleration to constant velocity, T_S, can be determined as:

$T_S = SR_V C_V / 2.5 SR_A C_A$

Note that this formula comes from equating the spectral acceleration in the constant acceleration range, $2.5SR_AC_A$, with the spectral acceleration in the constant velocity range, C_V/T, and solving for T. Values of T_S for both soil types are also tabulated for the various effective damping values.

β_{eff}	SR_A	SR_V	T_S for Soil S_B	T_S for Soil S_D
5%	1.00	1.00	0.40 sec	0.58 sec
10%	0.78	0.83	0.43 sec	0.62 sec
15%	0.64	0.73	0.45 sec	0.66 sec
20%	0.55	0.66	0.47 sec	0.69 sec

Next the spectral acceleration in the constant acceleration range, S_{amax}, can be calculated for each damping level as $S_{amax} = 2.5SR_AC_A$. In addition, the spectral displacement at the location where the spectral curve switches from constant acceleration to constant velocity, S_{ds}, can be calculated as $S_{ds} = S_{amax}(T_S/2\pi)^2$. These values are tabulated below.

β_{eff}	S_{amax} for Soil S_B	S_{amax} for Soil S_D	S_{ds} for Soil S_B	S_{ds} for Soil S_D
5%	1.00 g	1.10 g	1.57 in	3.65 in
10%	0.78 g	0.85 g	1.38 in	3.22 in
15%	0.65 g	0.71 g	1.28 in	2.98 in
20%	0.55 g	0.61 g	1.22 in	2.83 in

With the above information, any spectral curve can be defined as follows

T	S_a	S_d
0	S_{amax}	0
T_S	S_{amax}	S_{ds}
$T>T_S$	SR_VC_V/T	$S_a(T/2\pi)^2$

Thus, values of S_a and S_d used to plot the demand spectra can be tabulated for soil types S_B and S_D at β_{eff} values of 5%, 10%, 15% and 20%, as follows.

β_{eff} = 5%, Soil Type S_B			β_{eff} = 10%, Soil Type S_B		
T (sec)	S_a (g)	S_d (in)	T (sec)	S_a (g)	S_d (in)
0	1.00	0	0	0.78	0
T_S=0.40	1.00	1.57	T_S=0.43	0.78	1.38
0.50	0.80	1.95	0.50	0.66	1.62
0.60	0.67	2.35	0.60	0.55	1.94
0.80	0.50	3.13	0.80	0.41	2.59
1.00	0.40	3.92	1.00	0.33	3.24
1.20	0.33	4.70	1.20	0.28	3.89
1.40	0.29	5.48	1.40	0.24	4.54
1.60	0.25	6.26	1.60	0.21	5.19
1.80	0.22	7.05	1.80	0.18	5.83
2.00	0.20	7.83	2.00	0.17	6.48
2.25	0.18	8.81	2.25	0.15	7.29
2.50	0.16	9.79	2.50	0.13	8.10
2.75	0.15	10.77	2.75	0.12	8.91
3.00	0.13	11.75	3.00	0.11	9.72

β_{eff} = 15%, Soil Type S_B			β_{eff} = 20%, Soil Type S_B		
T (sec)	S_a (g)	S_d (in)	T (sec)	S_a (g)	S_d (in)
0	0.65	0	0	0.55	0
T_s=0.45	0.65	1.28	T_s=0.47	0.55	1.22
0.50	0.58	1.42	0.50	0.53	1.28
0.60	0.48	1.71	0.60	0.44	1.54
0.80	0.36	2.28	0.80	0.33	2.05
1.00	0.29	2.85	1.00	0.26	2.57
1.20	0.24	3.42	1.20	0.22	3.08
1.40	0.21	3.99	1.40	0.19	3.59
1.60	0.18	4.55	1.60	0.16	4.11
1.80	0.16	5.12	1.80	0.15	4.62
2.00	0.15	5.69	2.00	0.13	5.13
2.25	0.13	6.40	2.25	0.12	5.78
2.50	0.12	7.12	2.50	0.10	6.42
2.75	0.11	7.83	2.75	0.10	7.06
3.00	0.10	8.54	3.00	0.09	7.70

β_{eff} = 5%, Soil Type S_D			β_{eff} = 10%, Soil Type S_D		
T (sec)	S_a (g)	S_d (in)	T (sec)	S_a (g)	S_d (in)
0	1.10	0	0	0.85	0
T_s=0.58	1.10	3.65	T_s=0.62	0.85	3.22
0.80	0.80	5.01	0.80	0.66	4.15
1.00	0.64	6.26	1.00	0.53	5.19
1.20	0.53	7.52	1.20	0.44	6.22
1.40	0.46	8.77	1.40	0.38	7.26
1.60	0.40	10.02	1.60	0.33	8.30
1.80	0.36	11.28	1.80	0.29	9.33
2.00	0.32	12.53	2.00	0.26	10.37
2.25	0.28	14.10	2.25	0.24	11.67
2.50	0.26	15.66	2.50	0.21	12.96
2.75	0.23	17.23	2.75	0.19	14.26
3.00	0.21	18.79	3.00	0.18	15.56

β_{eff} = 15%, Soil Type S_D			β_{eff} = 20%, Soil Type S_D		
T (sec)	S_a (g)	S_d (in)	T (sec)	S_a (g)	S_d (in)
0	0.71	0	0	0.61	0
T_s=0.66	0.71	2.98	T_s=0.69	0.61	2.83
0.80	0.58	3.64	0.80	0.52	3.29
1.00	0.47	4.55	1.00	0.42	4.11
1.20	0.39	5.47	1.20	0.35	4.93
1.40	0.33	6.38	1.40	0.30	5.75
1.60	0.29	7.29	1.60	0.26	6.57
1.80	0.26	8.20	1.80	0.23	7.39
2.00	0.23	9.11	2.00	0.21	8.21
2.25	0.21	10.25	2.25	0.19	9.24
2.50	0.19	11.39	2.50	0.17	10.27
2.75	0.17	12.53	2.75	0.15	11.29
3.00	0.16	13.66	3:00	0.14	12.32

The families of response spectra, along with the capacity spectrum (whose coordinates are derived in Table 8-14) can now be plotted as shown in Figures 8-58 and 8-59 for soil types S_B and S_D respectively. Note that the calculation of the points on the spectra and the actual plotting of the spectra can be easily accomplished using a spreadsheet.

Figures 8-60 and 8-61 illustrate the bilinear representation of the capacity curve for soil types S_B and S_D respectively. The bilinear representation is constructed such that it passes through the point on the capacity spectrum which has a displacement equal to the displacement obtained using the equal displacement approximation. In procedure B, unlike procedures A and C, once the bilinear representation of the capacity spectrum is obtained, it is not changed.

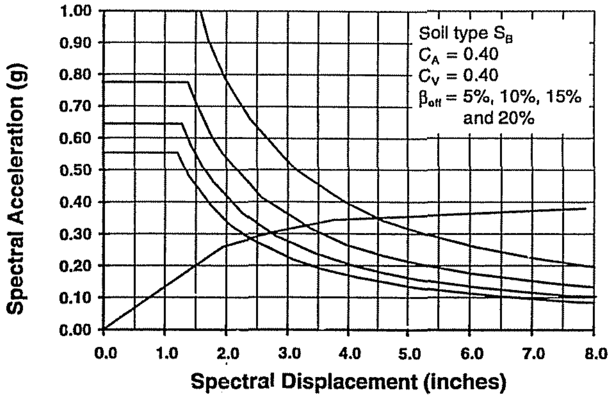


Figure 8-58. Family of Response Spectra Soil Type S_B

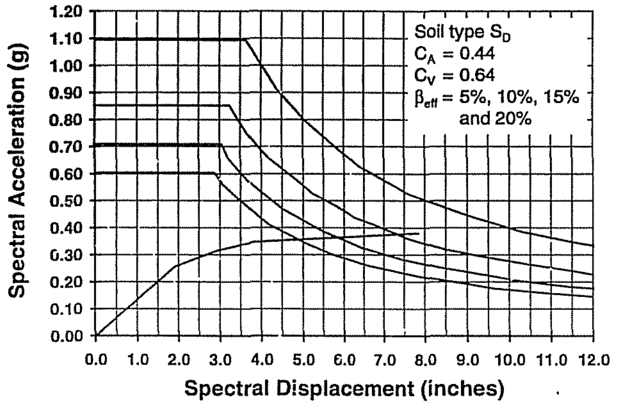


Figure 8-59. Family of Response Spectra Soil Type S_D

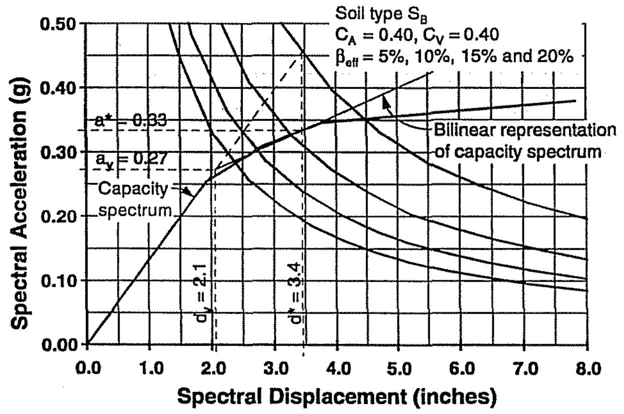


Figure 8-60. Bilinear Representation of Capacity Spectrum - Soil Type S_B

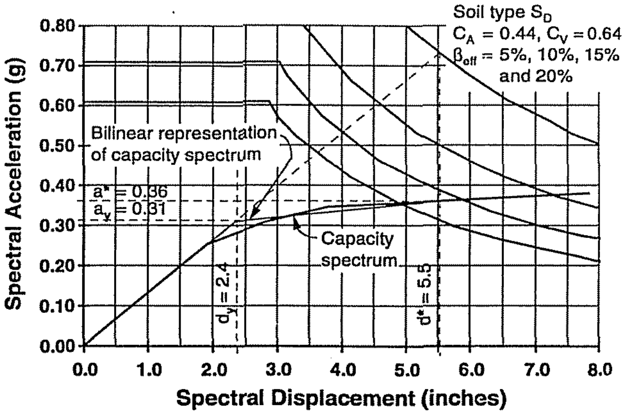


Figure 8-61. Bilinear Representation of Capacity Spectrum - Soil Type S_D

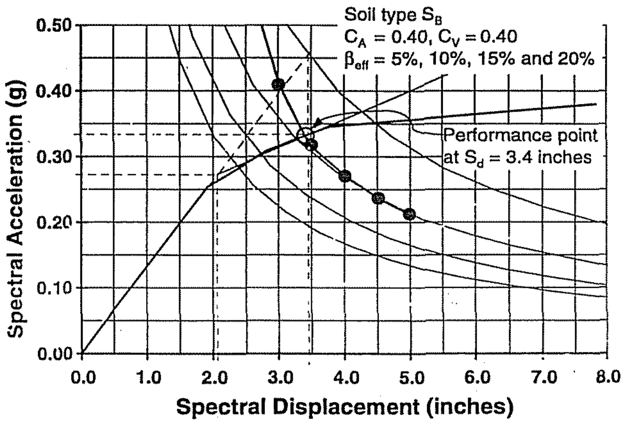


Figure 8-62. Performance Point Using Procedure B Soil Type S_B

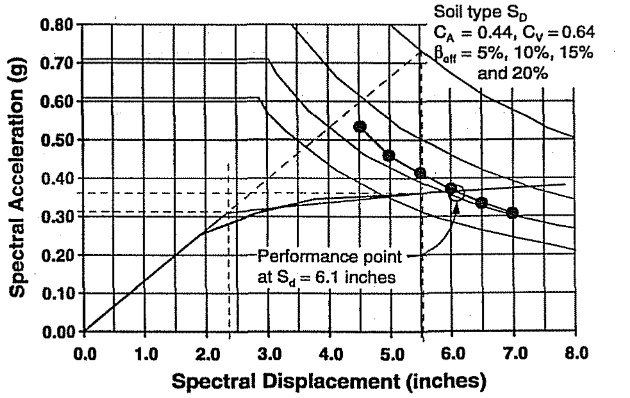


Figure 8-63. Performance Point Using Procedure B Soil Type S_D

Now, for each soil type, select displacement values for various points along the post yield portion of the bilinear representation of the capacity spectrum and solve for a_{pi}' and β_{eff} using equations 8-14 and 8-15, respectively. The selected displacements and the calculated values are tabulated below for each soil type. Note that since this example building is assumed to have structural behavior type C (see discussion in example for Procedure A), the factor κ, used in calculating the effective damping, β_{eff}, is a constant 0.33.

Values of a_{pi}' and β_{eff}
Soil Type S_B

S_d (in)	a_{pi}' (g)	β_{eff}
2.5	0.29	7.0
3.0	0.31	8.5
3.5	0.33	9.3
4.0	0.36	9.8
4.5	0.38	10.1
5.0	0.40	10.2

Values of a_{pi}' and β_{eff}
Soil Type S_D

S_d (in)	a_{pi}' (g)	β_{eff}
4.5	0.34	12.7
5.0	0.35	13.4
5.5	0.36	13.9
6.0	0.37	14.3
6.5	0.38	14.6
7.0	0.38	14.8

Figures 8-62 and 8-63 show the tabulated S_d, β_{eff} points plotted for each soil type, and indicate the intersection of the line connecting those points with the capacity spectrum. That intersection point, which is the performance point, occurs at a spectral displacement of 3.4 inches for soil type S_B, and at 6.1 inches for soil type S_D.

The solution obtained for soil type S_B is identical for both the equal displacement approximation and the "exact" solution obtained using procedure A. The same answer, 3.4 inches, is obtained using procedure B. It is expected that procedure B would give the same answer because the bilinear approximation of the capacity spectrum is based on the displacement obtained using the equal displacement approximation.

The solution obtained for soil type D is somewhat different for the equal displacement approximation (5.5 inches) and the "exact" solution of procedure A (5.9 inches). Thus it is expected that in this case, the solution obtained using procedure B will be slightly different from the "exact" solution of procedure A, and it is (6.1 inches). Thus this example illustrates the point made in Section 8.2.2.1.3, that when using procedure B, if the performance point does not fall at a displacement in the area of the displacement obtained using the equal displacement approximation, then the engineer may want to verify the results using either procedure A or C.

In this case, the performance point is reasonably close to the displacement obtained using the equal displacement approximation, and consequently, the solution obtained (6.1 inches) is reasonably close, within approximately 2% of the "exact" solution (5.9 inches).

8.3.3.5 Performance Point Calculation by Capacity Spectrum Method—Procedure C

Procedure C is described in Section 8.2.2.1.4. Development of the family of response spectra for this example is illustrated in Section 8.3.3.4 (example of Procedure B). Figures 8-58 and 8-59 show the capacity spectrum (whose coordinates are derived in Table 8-14) superimposed over the family of response spectra for soil types S_B and S_D respectively.

From this point procedure C will be demonstrated for soil type S_B first and then for soil type S_D. For soil type S_B, the initial trial performance point, a_{p1}, d_{p1} is chosen at the intersection of the capacity spectrum with the elastic (5% damped) response spectrum. Note that for procedure C to work correctly, the initial trial performance point should have a larger displacement than the final performance point. Figure 8-64 shows the point a_{p1}, d_{p1} and its associated bilinear approximation of the capacity curve.

The effective damping, β_{eff}, is calculated using the points defined in Figure 8-64 and using Table 8-7, since the building is assumed to have

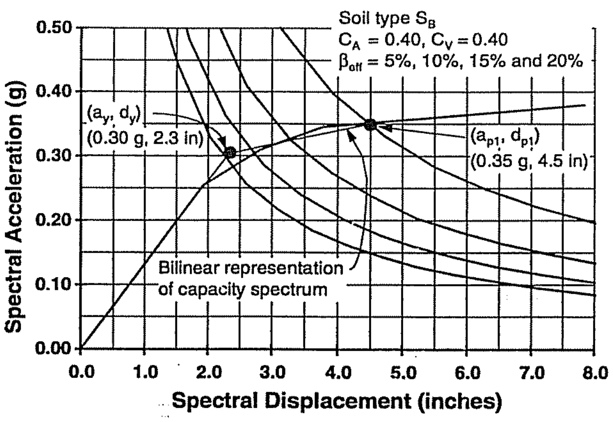


Figure 8-64. Initial Bilinear Approximation for Procedure C, Soil Type S_B

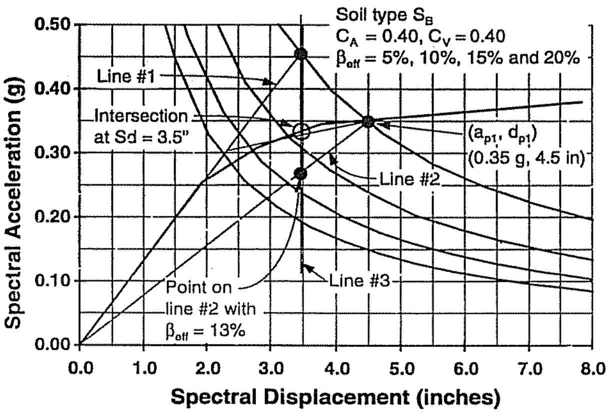


Figure 8-65. Initial Iteration for Procedure C Soil Type S_B

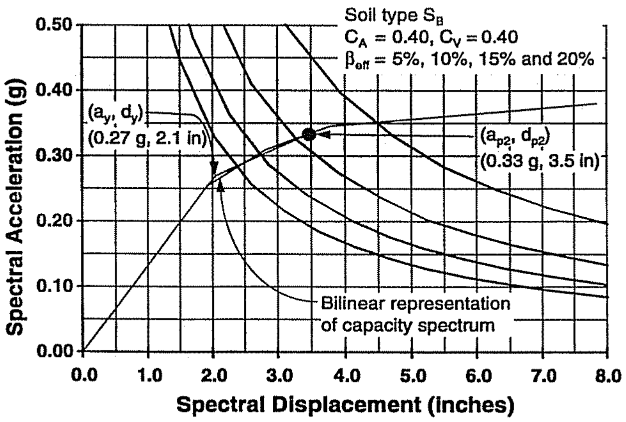


Figure 8-66. Second Bilinear Approximation for Procedure C, Soil Type S_B

structural behavior type C, as discussed in Section 8.3.3.3 (example of Procedure A).

$a_{pl} = 0.35g$
$d_{pl} = 4.5$ inches
$a_y = 0.30g$
$d_y = 2.3$ inches

$$\frac{d_{p1}}{d_y} = \frac{4.5}{2.3} = 1.96$$

$$\text{slope ratio} = \frac{\frac{a_{p1}}{a_y} - 1}{\frac{d_{p1}}{d_y} - 1} = \frac{\frac{0.35}{0.30} - 1}{\frac{4.5}{2.3} - 1} = 0.17$$

Entering Table 8-7 with the above values for d_{pl}/d_y and the slope ratio yields, upon interpolation, a β_{eff} value of approximately 13%. Now the first iteration of the graphical procedure can progress. Referring to Figure 8-65, line #1 is drawn from the origin to the elastic (5% damped) spectrum at a slope matching the initial stiffness. Line #2 is drawn from the origin to point a_{pl}, d_{pl}, which in this case is (0.35g, 4.5 inches). The point on line #2 corresponding to β_{eff} = 13% is plotted. Line #3 is drawn from the intersection of line #1 with the elastic spectrum to the point on line #2 corresponding to β_{eff} = 13%. Finally, the displacement at the intersection point of line #3 with the capacity spectrum is read as S_d = 3.5 inches.

Since the spectral displacement at the intersection point, 3.5", is not within ± 5% of the displacement at point d_{pl}, a second iteration is required. For the second iteration, the displacement, d_{p2}, is chosen equal to the displacement at the intersection point in the first iteration, 3.5 inches. Figure 8-66 shows the point a_{p2}, d_{p2} and its associated bilinear approximation of the capacity curve.

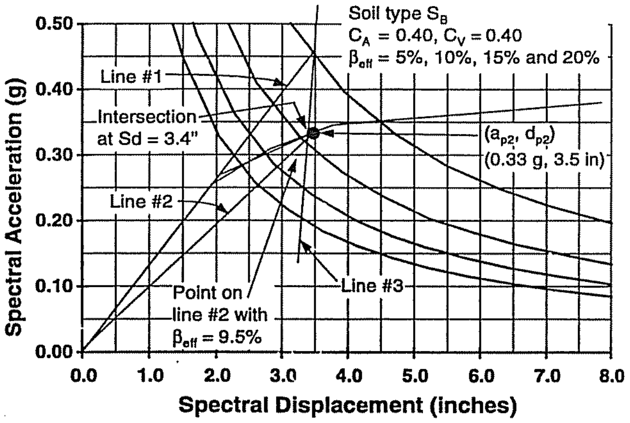


Figure 8-67. Second Iteration for Procedure C Soil Type S_B

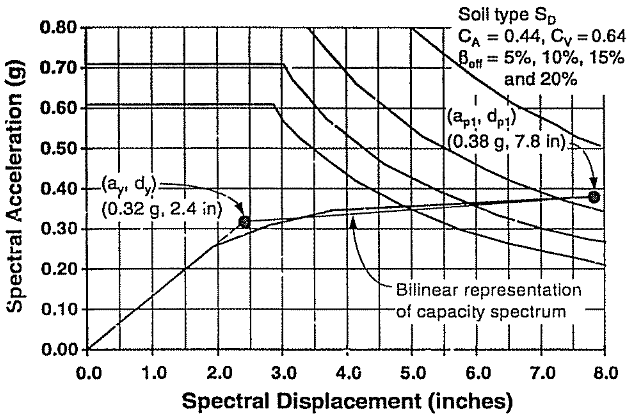


Figure 8-68. Initial Bilinear Approximation for Procedure C, Soil Type S_D

The effective damping, β_{eff}, is calculated using the points defined in Figure 8-66 and Table 8-7.

a_{p2} = 0.33g
d_{p2} = 3.5 inches
a_y = 0.27g
d_y = 2.1 inches

$$\frac{d_{p2}}{d_y} = \frac{3.5}{2.1} = 1.67$$

$$\text{slope ratio} = \frac{\frac{a_{p2}}{a_y} - 1}{\frac{d_{p2}}{d_y} - 1} = \frac{\frac{0.33}{0.27} - 1}{\frac{3.5}{2.1} - 1} = 0.33$$

Entering Table 8-7 with the above values for d_{p2}/d_y and the slope ratio yields, upon interpolation, a β_{eff} value of approximately 9.5%. Now the second iteration of the graphical procedure can be performed as illustrated in Figure 8-67.

Since the spectral displacement at the intersection point, 3.4", is within ± 5% of the displacement at point d_{p2}, 3.5", then the displacement at the intersection, 3.4", is taken as the demand displacement. Note that for soil type S_B, the first iteration yielded results quite close (within approximately 3%) to the final answer.

A similar procedure is followed for soil type S_D. For soil type S_D, the initial trial performance point, a_{p1}, d_{p1} is chosen at the end of the capacity spectrum because the capacity spectrum does not intersect the elastic (5% damped) response spectrum. Figure 8-68 shows the point a_{p1}, d_{p1} and its associated bilinear approximation of the capacity curve.

The effective damping, β_{eff}, is calculated using the points defined in Figure 8-68 and using Table 8-7.

a_{p1} = 0.38g
d_{p1} = 7.8 inches
a_y = 0.32g
d_y = 2.4 inches

$$\frac{d_{p1}}{d_y} = \frac{7.8}{2.4} = 3.25$$

$$\text{slope ratio} = \frac{\frac{a_{p1}}{a_y} - 1}{\frac{d_{p1}}{d_y} - 1} = \frac{\frac{0.38}{0.32} - 1}{\frac{7.8}{2.4} - 1} = 0.08$$

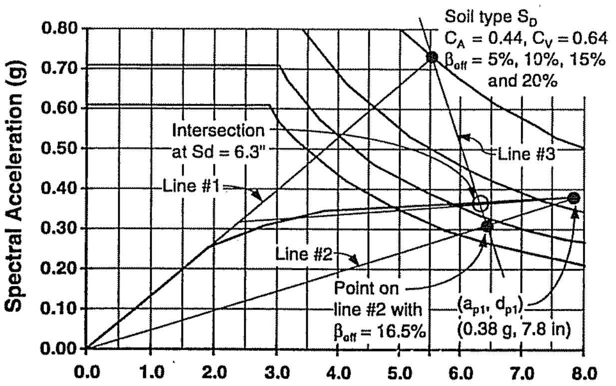


Figure 8-69. Initial Iteration for Procedure C Soil Type S_D

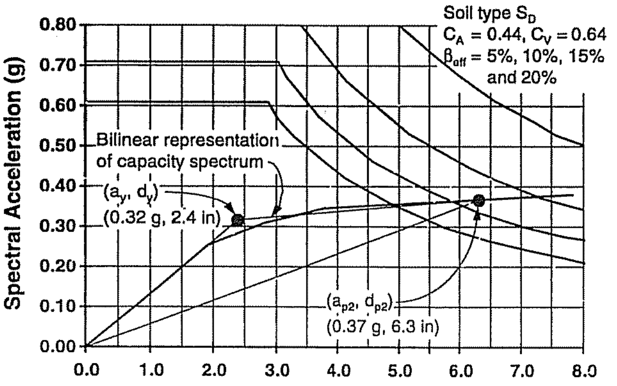


Figure 8-70. Second Bilinear Approximation for Procedure C, Soil Type S_D

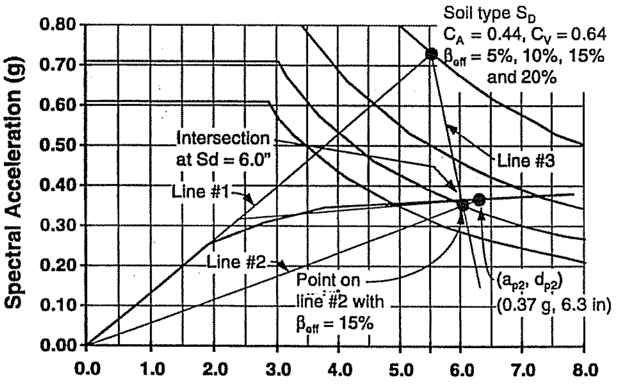


Figure 8-71. Second Iteration for Procedure C Soil Type S_D

Entering Table 8-7 with the above values for d_{p1}/d_y and the slope ratio yields, upon interpolation, a β_{eff} value of approximately 16.5%. Now the first iteration of the graphical procedure can be performed as illustrated in Figure 8-69.

Since the spectral displacement at the intersection point, 6.3", is not within ± 5% of the displacement at point d_{p1}, a second iteration is required. For the second iteration, the displacement, d_{p2}, is chosen equal to the displacement at the intersection point in the first iteration, 6.3 inches. Figure 8-70 shows the point a_{p2}, d_{p2} and its associated bilinear approximation of the capacity curve.

The effective damping, β_{eff}, is calculated using the points defined in Figure 8-70 and Table 8-7.

$a_{p2} = 0.37g$
$d_{p2} = 6.3$ inches
$a_y = 0.32g$
$d_y = 2.4$ inches

$$\frac{d_{p2}}{d_y} = \frac{6.3}{2.4} = 2.62$$

$$\text{slope ratio} = \frac{\dfrac{a_{p2}}{a_y} - 1}{\dfrac{d_{p2}}{d_y} - 1} = \frac{\dfrac{0.37}{0.32} - 1}{\dfrac{6.3}{2.4} - 1} = 0.10$$

Entering Table 8-7 with the above values for d_{p2}/d_y and the slope ratio yields, upon interpolation, a β_{eff} value of approximately 15%. Now the second iteration of the graphical procedure can be performed as illustrated in Figure 8-71.

Since the spectral displacement at the intersection point, 6.0", is within ± 5% of the displacement at point d_{p2}, 6.3", then the displacement at the intersection, 6.0", is taken as the demand displacement. Note that for soil type S_D, the first iteration yielded results within approximately 5% of the final answer, not quite as close as for soil type S_B, but still not a bad estimate for a single iteration.

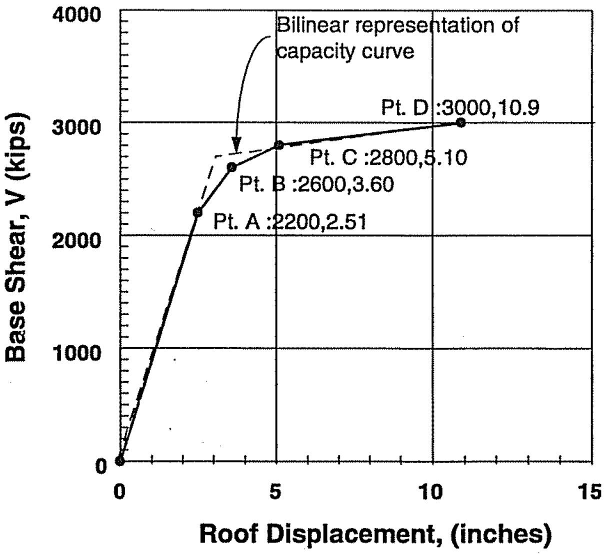


Figure 8-72. Bilinear Representation of Capacity Curve for Displacement Coefficient Method

8.3.3.6 Demand Displacement Using the Displacement Coefficient Method

The first step in determining demand using the displacement coefficient method (described in Section 8.2.2.2) is to construct a bilinear representation of the capacity curve. Note that the displacement coefficient method uses the capacity curve (V-Δ_{roof}), not the capacity spectrum (S_a-S_d).

The bilinear representation is shown in Figure 8-72. Note that in this example, the effective stiffness, K_e, is equal to the initial stiffness, K_i (see Figure 8-43), which is 2200/2.51 = 876 k/in.

From Table 8-10, the elastic fundamental period, T_i, is 0.88 seconds. The effective fundamental period, T_e, is calculated using equation 8-16 as:

$$T_e = T_i\sqrt{\frac{K_i}{K_e}} = 0.88\sqrt{\frac{876}{876}} = 0.88 \text{ seconds.}$$

The target displacement, δ_t, is calculated using equation 8-17.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2}$$

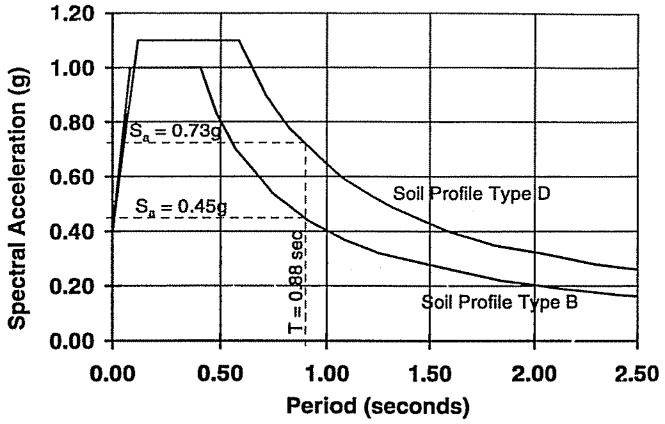


Figure 8-73. Spectral Accelerations for Displacement Coefficient Method

C_0 = Referring to column labeled $PF_{R,1}$ in Table 8-14, and interpolating, the first mode participation factor (roof) for soil type S_B, at a displacement of approximately 5.1 inches is 1.35

Commentary: *The 5.1 inch displacement is estimated by calculating δ_t using an assumed value of 1.37 for C_0, which is based on the approximate values in Table 8-17.*

C_0 = Referring to column labeled $PF_{R,1}$ in Table 8-14, and interpolating, the first mode participation factor (roof) for soil type S_D, at a displacement of approximately 8.3 inches is 1.37

Commentary: *The 8.3 inch displacement is estimated by calculating δ_t using an assumed value of 1.37 for C_0, which is based on the approximate values in Table 8-17.*

C_1 = 1.0, since $T_e > T_0$

C_2 = 1.1, assuming framing type 1 and life safety structural performance level

C_3 = 1.0, positive post-yield stiffness

S_a = 0.45g = 174 in/sec^2 at T=0.88 seconds for soil profile type S_B, see Figure 8-73.

S_a = 0.73g = 282 in/sec^2 at T=0.88 seconds for soil profile type S_D, see Figure 8-73

T_e = 0.88 seconds

Thus for soil profile type S_B,

$$\delta_t = 1.35 * 1.0*1.1*1.0 * 174 * \frac{0.88^2}{4\pi^2} = 5.1 \text{ in.}$$

For soil profile type S_D,

$$\delta_t = 1.37 * 1.0*1.1*1.0 * 282 * \frac{0.88^2}{4\pi^2} = 8.3 \text{ in.}$$

Note that the above target displacements are actual roof displacements; they are not spectral displacements as would be obtained using the capacity spectrum method.

8.3.3.7 *Comparison of Performance Point / Demand Displacement Procedures*

Table 8-15 compares the performance point displacements resulting from the equal displacement approximation, the capacity spectrum method procedures A, B, and C and from the displacement coefficient method. Note that the equal displacement approximation and the capacity spectrum method results must be converted back from spectral displacements to absolute (actual) displacements. This is done by multiplying the spectral values by appropriate modal participation factors, PF. Referring to column labeled PF_{R1} in Table 8-14, and interpolating, a PF value of about 1.32 is appropriate for the soil type S_B performance point at a spectral displacement of about 3.4 inches, and a PF value of about 1.37 is appropriate for type S_D, where the performance point spectral displacement is about 5.9 inches.

The results of the capacity spectrum methods A, B and C are similar. The displacement coefficient method gives results approximately 10% different for soil type S_B and similar for soil type S_D. Of course, had a different structural behavior type been assumed for the capacity spectrum method (structural behavior type C was assumed), then the difference between the capacity spectrum methods and the displacement coefficient method would have been greater. In this comparison, the results of the displacement coefficient method depend on the coefficient C_2, which depends on, among other things, the structural performance level. If a structural performance level other than life safety had been assumed, then the results for the displacement coefficient method would be different.

Table 8-15. Comparison of Performance Point Displacements from Various Methods

			CSM Procedures			
Soil Type	**Disp. Type**	**Equal Disp. Approx.**	**A**	**B**	**C**	**Disp. Coef. Method**
S_B	Spectral Actual	3.4" 4.5"	3.4" 4.5"	3.4" 4.5"	3.4" 4.5"	N.A. 5.1"
S_D	Spectral Actual	5.5" 7.5"	5.9" 8.1"	6.1" 8.4"	6.0" 8.2"	N.A. 8.3"

For this example, the equal displacement approximation gave good initial estimates of the demand displacements. As stated above for the displacement coefficient method, had a different structural behavior type been assumed for the capacity spectrum method, then the comparison would be different.

8.4 Other Analysis Methods

As previously mentioned, both elastic (linear) and inelastic (nonlinear) methods are available for the analysis of existing concrete buildings. Section 8.4.1 describes available elastic methods. Section 8.4.2 describes inelastic methods other than the capacity spectrum method and the displacement coefficient method described above.

8.4.1 Other Elastic Methods of Analysis

The other elastic methods of analysis include standard code analysis procedures, both static and dynamic, and procedures using demand capacity ratios.

8.4.1.1 Standard Code Procedures

Standard code procedures include both static and dynamic analysis methods. The code static lateral force procedure is commonly used by the engineering profession to design buildings. In this methodology, the building code prescribes a formula that determines lateral forces. These forces are applied in a prescribed manner to determine the adequacy of the structural system. If some of the components of the designed structural system are not adequate, the design is revised and the modified design is reanalyzed. This process is repeated until all the provisions of the building code are satisfied.

The procedure relies on principles of statistics and the structural components are evaluated for serviceability in the elastic range of strength and deformation. Additional requirements are prescribed to supply ductile and energy dissipating characteristics to the structural system to enable it to survive excursions into the inelastic range of lateral displacements during major earthquakes.

Although this procedure is commonly called a static lateral force procedure, it does include some implicit elements of dynamics. These include the use of the fundamental period of vibration (T) to determine the amplification (C-factor) of ground motion acceleration (Z-factor) and the use of vertical distribution of force equations to approximate modal response. Because of these features, the methodology is sometimes referred to as the *equivalent* lateral force procedure.

In some cases, a building requires an explicit dynamic lateral force procedure, which may be either a response spectrum analysis or an elastic time history analysis. While these procedures add aspects of dynamics to the design procedure, resulting forces are generally scaled to match the lateral force used in the static procedure. Also, components are still evaluated for serviceability in the elastic range of strength and deformation.

Standard code procedures include all those used by model building codes (e.g., UBC, BOCA, SBCC) and those recommended by code development bodies (e.g., NEHRP, SEAOC, and BSSC). The Division of State Architect has developed an elastic analysis methodology for evaluation and retrofit of existing State buildings (CBSC 1996) which will be adopted into the *California Building Code*. This category also includes procedures from source documents, such as ATC-14 (ATC 1987), FEMA-273 (ATC 1996a), and FEMA 178 (BSSC 1992).

Code procedures may be considered as alternative methodologies for evaluation and retrofit of existing concrete buildings. The advantages of code procedures are that design professionals and building officials are familiar and comfortable with these procedures, and the simplified analysis methods often allow design costs to be minimized. The disadvantages are that there are difficulties in applying current code provisions for new construction to the complexities of retrofitting existing concrete buildings. Also, there is a greater uncertainty of satisfying performance goals than there would be by using procedures with a more rational approach.

8.4.1.2 Elastic Procedures Using Demand Capacity Ratios

Demand/capacity ratio (DCR) procedures are slightly different from standard code procedures. While the code approach reduces the full earthquake demand by an R-factor and adds the resulting seismic forces to gravity forces, the DCR approach takes the full earthquake force, without reduction, and adds it to the gravity demands. In both cases, the sum of seismic and gravity demands is compared with available capacity.

In equation form, this can be stated as:

Code procedure: $D+L+\frac{E}{R} \le C0$

DCR procedure: $0\frac{D+L+E}{C} = DCR \le m$

The code procedure checks element strengths for capacity to withstand a fraction of the earthquake demand; the DCR procedure checks the overstress ratio considering the full earthquake. There are several variations of DCR procedures.

The U.S. Army, Navy, and Air Force (Army 1986, 1996) have the inelastic demand ratio (IDR) method that can be used for essential as well as other buildings as a dynamic analysis procedure. Prescribed values of IDR (i.e., "m" in the equation above) are given as limits to DCR values. FEMA 273 (ATC 1996a) provides a similar methodology for a linear static procedure that establishes acceptable values of "m". ATC-14 (ATC 1987) and FEMA 178 (BSSC 1992) also use elements of a DCR procedure, as does the simplified preliminary analysis technique discussed in Section 5.6 of this document.

8.4.2 Other Nonlinear Methods of Analysis

The other inelastic methods of analysis include the secant method and nonlinear time history analysis.

8.4.2.1 Secant Method

The secant method is an analysis method presented in the City of Los Angeles Proposed Division 95 document titled *Earthquake Hazard Reduction in Existing Reinforced Concrete Buildings and Concrete Frame Buildings with Masonry Infill* (COLA 1995). This section summarizes the methodology.

Commentary: *The secant method of design can be derived from a "substitute structure" procedure similar to a methodology developed by Sozen and others (Shibata and Sozen 1976).*

When analyzing a building with the secant method, a global elastic model of the structure is constructed. The model may represent individual structural components such as beams, columns, or wall piers, or they may represent elements such as frames, stories, or walls as assemblages of individual components.

Special stiffness values are calculated for the modeled elements and components as follows:

- Force-deflection (pushover) curves are developed from analytical approaches or test data for each element or component
- A displacement pattern for the structure is assumed
- Based on the assumed displacement pattern, the element displacements are determined
- The element pushover curves are used to determine the associated element secant stiffness that would represent the force-displacement behavior of the element at the assumed displacement level

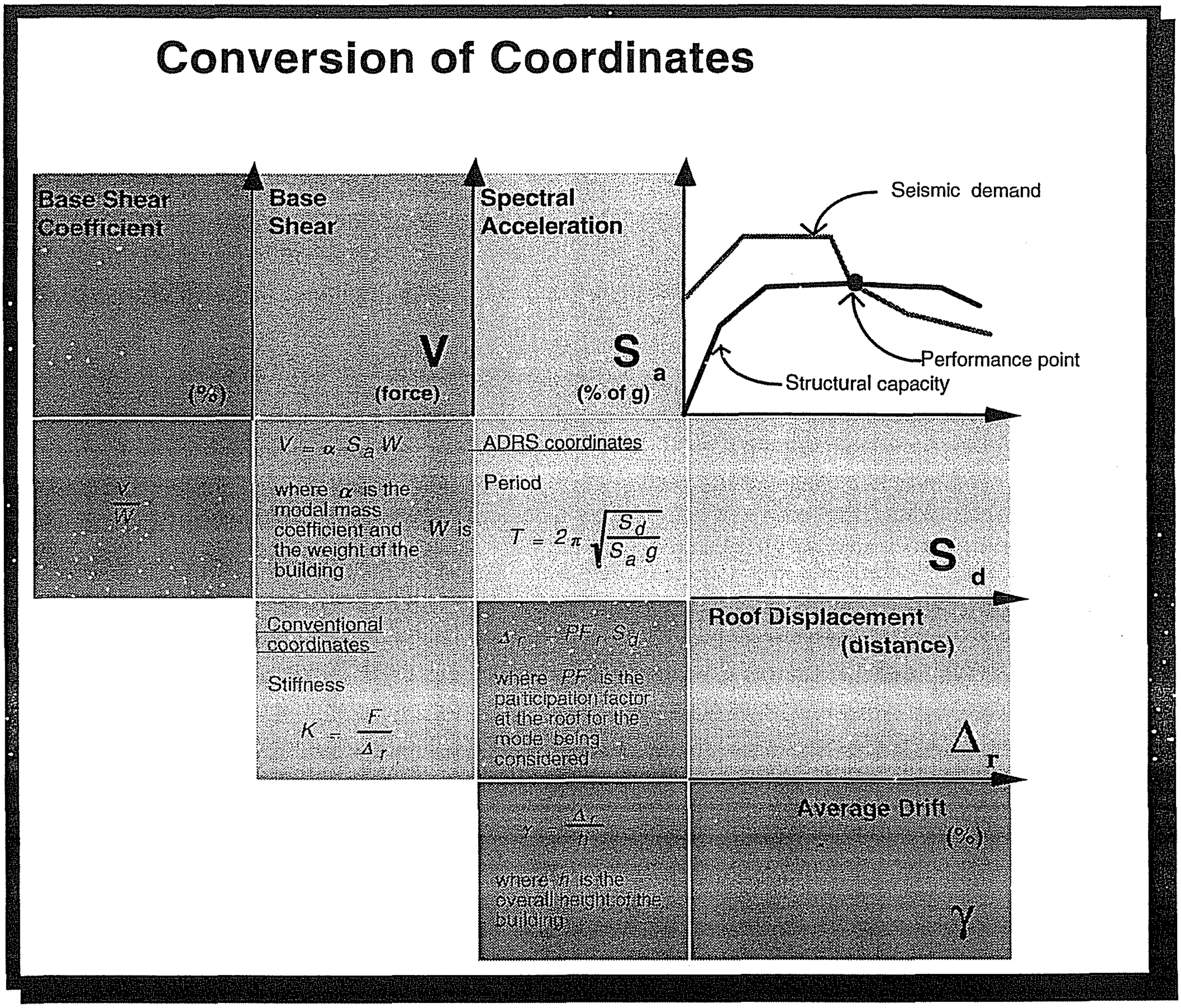
The element secant stiffnesses are applied to each of the elements in the global elastic model. The global elastic model is analyzed using elastic response spectrum analysis. The ground motion used in the analysis is either a code provided 5 percent damped response spectrum or a site specific 5 percent damped response spectrum.

In general, the response spectrum analysis will predict a different displacement pattern than was originally assumed. At this point, iteration begins. The pushover curves are used to select a new set of element secant stiffnesses based on the displacements predicted by the global analysis. The global elastic model is modified with the new secant stiffnesses, and the response spectrum analysis is repeated. This process continues until the displacements predicted by the computer model reasonably match the displacements used to calculate the secant stiffnesses, at which point the analysis has predicted the earthquake demand.

The principal advantages of the secant method are that it accounts for three dimensional effects including torsion and multi-direction loading and that it accounts for higher mode effects. The main disadvantage of the approach is that it can be somewhat more time consuming than other static nonlinear procedures.

8.4.2.2 Nonlinear Time History Analysis

Although nonlinear time history analysis is becoming more feasible, it is currently complex and time consuming, and requires considerable judgment. This procedure is listed here for completeness, but guidelines for application are beyond the scope of this document.



8.5 Basics of Structural Dynamics

8.5.1 General

This section presents background information on basic principles of structural dynamics. Equation formats and symbols may be different from those used elsewhere.

8.5.2 Modal Analysis Equations

The quantities used in modal analysis models are described below. Note that the modal quantities are usually calculated by computer analysis; they are presented here as an aid to understanding modal analysis and as a tool for back-checking computer results.

1. **Modal Participation Factor.** The modal participation factor will be calculated for each mode using equation 8-20:

$$PF_m = \left[\frac{\sum_{i=1}^{N}(w_i\phi_{im})/g}{\sum_{i=1}^{N}(w_i\phi_{im}^2)/g}\right] \qquad (8\text{-}20)$$

where:

PF_m = Modal participation factor for mode m

w_i/g = Mass assigned to level i

ϕ_{im} = Amplitude of mode m at level i

N = Level N, the level which is uppermost in the main portion of the structure

The units of the participation factor PF_m are dependent on the normalization procedure, in some references, ϕ is normalized to 1.0 at the uppermost mass level, other references will normalize the value of $\Sigma(w/g)\phi^2$ to 1.0. It should be noted that some references define a "modal story participation factor", PF_{im} as the quantity within the brackets in equation 8-20, multiplied by the quantity ϕ_{im}, the amplitude of mode m at level i. The modal story participation factor, PF_{im}, is unitless.

$$PF_{im} = PF_m\ \phi_{im} \qquad (8\text{-}20a)$$

2. **Effective Mass Coefficient α_m.** The effective mass coefficient will be calculated for each mode using equation 8-21:

$$\alpha_m = \frac{\left[\sum_{i=1}^{N}(w_i\phi_{im})/g\right]^2}{\left[\sum_{i=1}^{N}w_i/g\right]\sum_{i=1}^{N}(w_i\phi_{im}^2)/g} \text{(unit-less)} \qquad (8\text{-}21)$$

3. **Modal Story Accelerations.** The story accelerations for mode m are calculated using equation 8-22:

$$a_{im} = PF_m\phi_{im}S_{am} \qquad (8\text{-}22)$$

where:

a_{im} = Story acceleration at level i for mode m (as a ratio of the acceleration of gravity, g).

ϕ_{im} = Amplitude at level i for mode m.

S_{am} = Spectral acceleration for mode m from the response spectrum (as a ratio of the acceleration of gravity, g).

4. **Modal Story Lateral Forces.** The lateral forces (mass x acceleration) for mode m are calculated using equation 8-23:

$$F_{im} = PF_m\phi_{im}S_{am}w_i \qquad (8\text{-}23)$$

where:

F_{im} = Story lateral force at level i for mode m.

w_i = Weight at or assigned to level i.

S_{am} = Spectral acceleration for mode m from the response spectrum (as a ratio of the acceleration of gravity, g).

5. **Modal Shears and Moments.** Story shears and overturning moments for the building and shears and flexural moments for the structural elements will be computed for each mode separately, by linear analysis, in conformance with the story forces determined in equation 8-23.

6. **Modal Base Shear.** The total lateral force corresponding to mode m is calculated using the equation 8-24. Note that the sum of F_{im} from roof to base will equal V_m:

$$V_m = \alpha_m S_{am} W \qquad (8\text{-}24)$$

where:

V_m = Total lateral force for mode m.

α_m = Effective mass coefficient for mode m.

W = Total dead load of the building and applicable portions of other loads (Ref. UBC, NEHRP, et al.).

7. **Modal Displacements and Drifts.** Modal lateral story displacements are related to modal spectral displacements by equation 8-25:

$$\delta_{im} = PF_m\phi_{im}S_{dm} = PF_{im}S_{dm} \qquad (8\text{-}25)$$

where:

δ_{im} = Lateral displacement at level i for mode m.

S_{dm} = Spectral displacement for mode m calculated from the acceleration response spectrum (i.e., $S_{dm} = S_{am}\,(T/2\pi)^2 g$).

Using equation 8-25 and the relationship $S_{dm} = S_{am}(T_m/2\pi)^2 g$, displacements can also be calculated by equation 8-26:

$$\delta_{im} = PF_m\phi_{im}S_{am}(T_m/2\pi)^2 g \qquad (8\text{-}26)$$

where:

T_m = Modal period of vibration

The modal drift in a story, $\Delta\delta_{im}$, will be computed as the interstory displacement which is the difference between the displacements (δ_{im}) at the top and bottom of the story under consideration, i.e., $\Delta\delta_{im} = \delta_{(i+1)m} - \delta_{im}$.

8. **Modal Periods of Vibration.** Estimated building period when loading *approximates* mode shape:

$$T_m = 2\pi\sqrt{(\Sigma w_i\ \delta_{im}^2) \div (g\ \Sigma\ F_{im}\ \delta_{im})} \qquad (8\text{-}27)$$

Modal periods: when loading is consistent with mode shape (period will be the same at any floor i).

$$T_m = 2\pi\sqrt{\delta_{im}\ w_i / (F_{im}\ g)} \qquad (8\text{-}28)$$

Modal period:

$$T_m = 2\pi\sqrt{S_{dm} / (S_{am}\ g)} \qquad (8\text{-}29)$$

8.5.3 Formats of Response Spectra

The traditional graphical format for showing response spectra has been to use a linear coordinate system of spectra acceleration (S_a) and period of vibration (T). A log tripartite coordinate system has also been used that includes S_a, S_v, and S_d with T. Linear coordinates can also be used for S_v versus T and S_d versus T. A more convenient form is called the ADRS format (Mahaney, et al. 1993). The ADRS format has linear coordinates of S_a versus S_d. T is shown as a straight line radiating out from the origin (0,0) and S_v can be shown by a curved line. Figure 8-74 illustrates all the formats for response spectra and their relationships to each other. Families of response spectra are shown for a Zone 4 (EPA=0.4g) for soil-sites A, B, C, D, and E (refer to Chapter 4).

1. For soil-site E (i.e., SE), C_A=0.4 and C_V=0.96:

   S_a at T=0.3 sec is 2.5 x C_A = 1.0 g.

   S_a at T=1.0 sec = 0.96 g.

   S_v = $(T/2\pi)$ S_a x 386 in/sec^2 = 59 ips (inches per second)

   S_d (assumed to be constant at T=4.2 sec) is equal to $(T/2\pi)$ S_v = 39.5 in.

2. If a point is located on the SE curve with a period T = 3.19 sec, S_v = 59 ips,

   S_a = $(2\pi/T)$ S_v/386 = 0.30g, and

   S_d = $(T/2\pi)$ S_v = 30.0 in.

This point is shown on each of the formats to illustrate the relationship between the different curves.

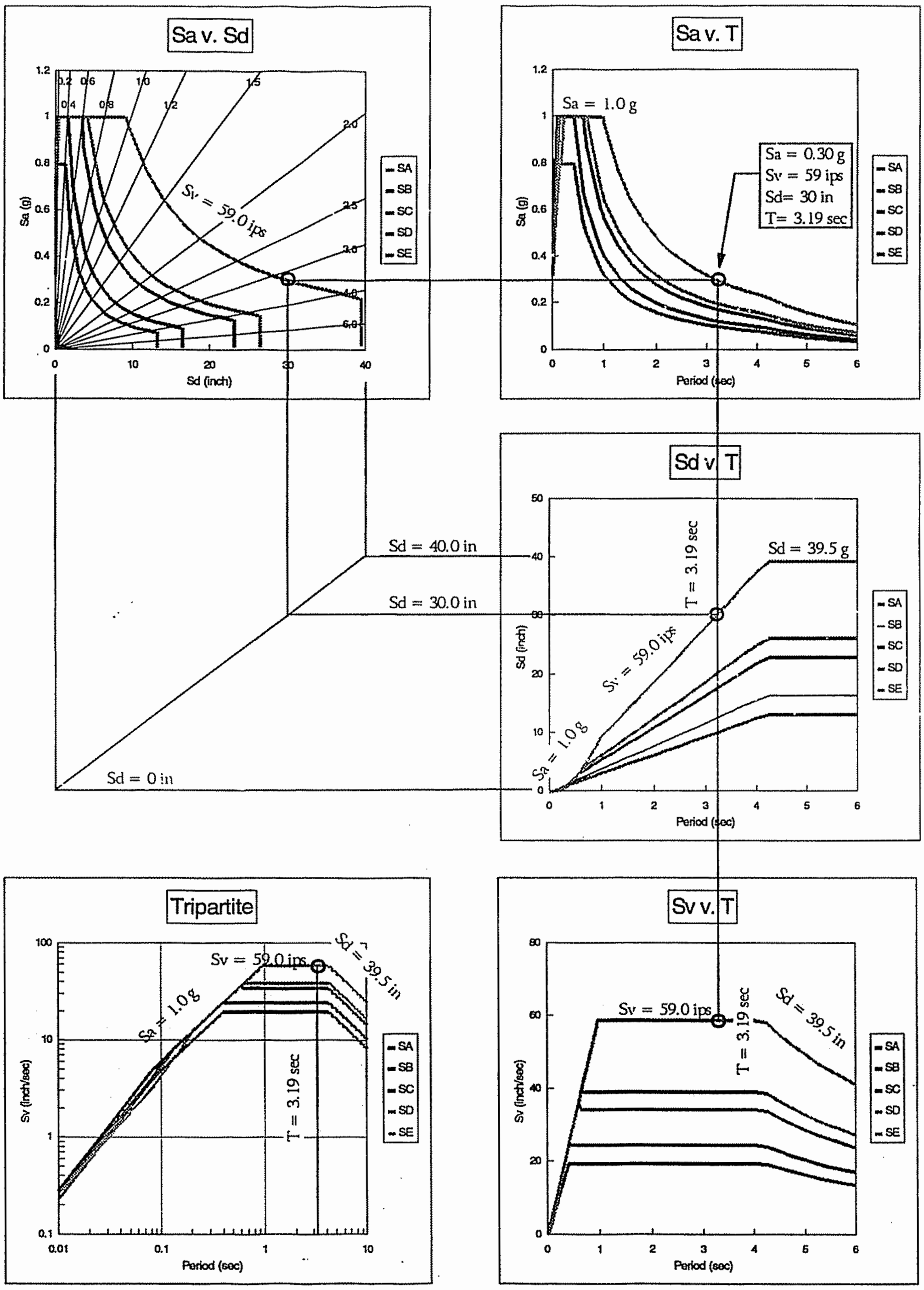
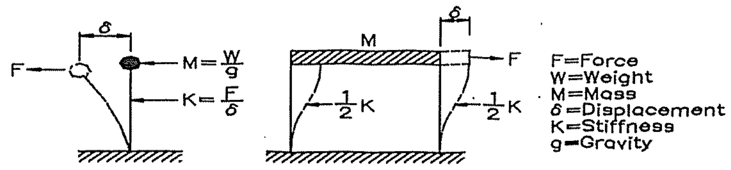
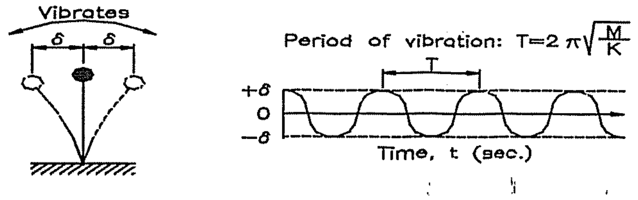


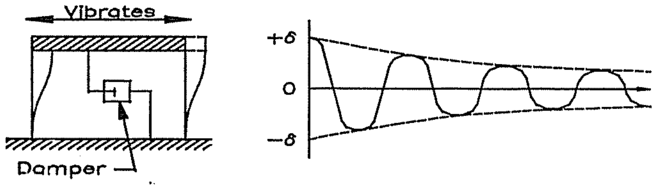
Figure 8-74. Formats of Response Spectra



a. Idealized single lumped mass system



b. Free vibration (no damping)



c. Damped free vibration

Figure 8-75. Single Degree of Freedom Systems

8.5.4 Explanation and Use of Modal Participation Factors and Effective Mass Coefficients.

1. **Single-Degree-of-Freedom (SDOF) System.** The fundamental structural system is the simple oscillator or SDOF system shown in Figure 8-75a. This system is represented by a single lump of mass on the upper end of a vertically cantilevered pole or by a mass supported by two columns. This system is used in textbooks to illustrate fundamental principles of dynamics. It represents two kinds of real structures: a single-column structure with a relatively large mass at its top; and a single-story frame structure with flexible columns and a rigid roof system. In the idealized system, the mass (M) represents the weight (W) of the system divided by the acceleration of gravity (g). These quantities are related by the formula $M = W/g$. The pole or columns represent the stiffness (K) of the system, which is equal to a horizontal force (F) applied to the mass divided by the displacement (δ) resulting from that force. These quantities are related by the formula $K = F/\delta$. If the mass is deflected and then quickly released, it will freely vibrate at a certain frequency which is called its natural frequency of vibration. The period of vibration (T), which is the inverse of the frequency of vibration, is the time taken for the mass to move through one complete cycle (i.e. from one side to the other and back again (Figure 8-75b). The period is equal to $2\pi(M/K)^{1/2}$.

   The internal energy dissipation or friction within a structure causes the vibrational motion

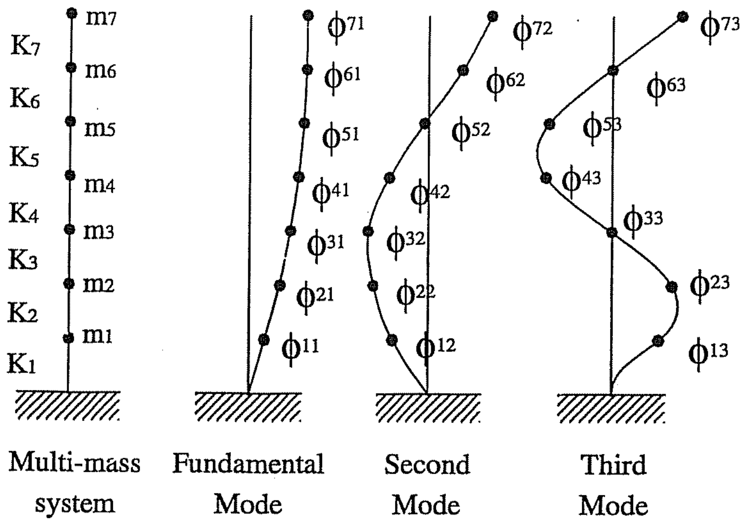
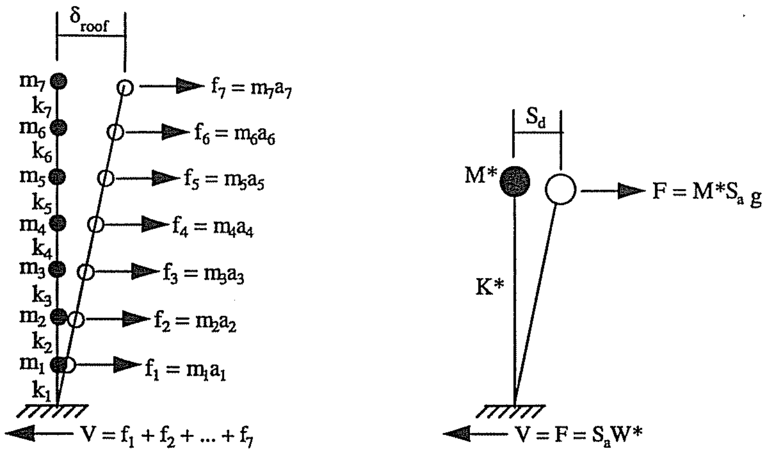


Figure 8-76. Multi-Degree of Freedom System

and to damp out as shown in Figure 8-75c. The amount of damping is defined in terms of a ratio (β), or percentage, of critical damping. In an ideal system having no damping (β = 0), a system, once displaced, would vibrate forever, i.e., as in Figure 8-75b. In a real system where there is some damping, the amplitude of motion will decrease for each cycle until the structure stops oscillating and comes to rest (Figure 8-75c). The greater the damping, the sooner the structure comes to rest. If the structure has damping equal to 100 percent of critical damping (β = 1.0), the displaced structure will come to rest without crossing the initial point of zero displacement.

2. **Multi-degree-of-freedom (MDOF) systems.** Multistory buildings are analyzed as MDOF systems. They can be represented by lumped masses attached at intervals along the length of a vertically cantilevered pole (Figure 8-76). Each mass can be deflected in one direction or another; for example, all masses may simultaneously deflect in the same direction (the fundamental mode of vibration), or some masses may go to the left while others are going to the right (higher modes of vibration) An idealized system, such as the one shown in Figure 8-76, has a number of modes equal to the number of masses. Each mode has its own natural modal period of vibration with a unique mode shape being formed by a line connecting the deflected masses (the first three mode shapes are shown in Figure 8-76). When oscillating motion is applied to the base of the multi-mass system, these masses move. The deflected shape is a combination of all the mode shapes; but modes having periods that are near, or equal to, predominant periods of the base motion will be amplified more than the other modes.

   Each mode of an MDOF system can be represented by an equivalent SDOF system having a normalized (M*) and stiffness (K*) where the period equals $2\pi(M^*/K^*)^{1/2}$. M* K* are functions of mode shapes, mass, and stiffness. This concept, as shown in Figure 8-77, provides the computational basis for using site specific earthquake response spectra based on SDOF systems for analyzing multi-storied buildings. With the period, mode



$$\delta_{roof} = PF_{R1}S_d$$
$$V = S_a\alpha_1W$$
$$W = (m_1 + m_2 + ... + m_7)g$$
$$W^* = \alpha_1W$$

a. Fundamental mode of a multi-mass system

b. Equivalent single mass system

M* and K* are effective values of mass and stiffness that represent the equivalent combined effects of the story masses (m) and the siffnesses (k). W* is the effective weight (=M*g).

Figure 8-77. MDOF System Represented By a Single Mass System

shape, mass distribution, and response spectrum, one can compute the deflected shape, story accelerations, forces, and overturning moments.

3. **Modal Participation Factors.** In Figure 8-77 diagram b is equivalent to diagram a. In other words, if during an earthquake the mass M* moves distance S_d, the roof of the building will move distance δ_{ROOF}. The ratio of δ_{ROOF} to S_d is, by definition, the modal participation for the fundamental mode at the roof level. This is PF_{im} in Equation 8-20a, where i is the roof and m is mode 1. PF_m is calculated from Equation 8-20 using the m-values and the fundamental ϕ-values in Figure 8-76 (note mass m equals weight w divided by gravity), where m7 is the mass at the roof and ϕ_{71} is the mode shape at the roof (level 7) for mode 1.

   Therefore, PF_{R1} =

$$\left(\frac{m_7\ \varphi_{71} + m_6\ \varphi_{61} + \text{-----} + m_1\ \varphi_{11}}{m_7\ \varphi_{71}^2 + m_6\ \varphi_{61}^2 + \text{----} + m_1\ \varphi_{11}^2}\right)\varphi_{71}\ \text{and}$$

$\delta_{ROOF} = PF_{R1}S_d$ (Figure 8-77a).

4. **Effective Mass Coefficient.** In Figure 8-77a the sum of f1 through f7 is the shear, V, at the base of the structure for the fundamental mode. The f-values are the same as the F_{im} values in Equation 8-23 (e.g., $f_7 = F_{71}$, $m_7 = w_7/g$, and $a_7 = PF_1\phi_{71}S_{a1}g$ per Equation 8-22). The sum of story forces, F_{im}, for mode 1 is equal to the base shear V_m for m=1. $V_m = \alpha_mS_{am}W$ (Equation 8-24) is the base shear in diagram a in Figure 8-77 for mode 1. $V = S_aW^*$ is the base shear in diagram b. W is the total weight (or mass x g) and W* is the effective weight. $W^* = \alpha_mW$ where α_m is the effective mass coefficient for mode m. The formula for calculating α_m is given in Equation 8-21. Thus for mode 1,

$$\alpha_1 = \frac{(m_7\ \varphi_{71} + m_6\ \varphi_{61} + \text{----} + m_1\ \varphi_{11})^2}{(m_7 + m_6 + \text{----} + m_1)(m_7\ \varphi_{71}^2 + m_6\ \varphi_{61}^2 + \text{----} + m_1\ \varphi_{11}^2)}$$

and $V_1 = \alpha_1\ W\ S_{a1}$ (Figure 8-77a).

5. **Sample Values of α_1 and PF_{R1}.** Table 8-16 presents standard values of α and PF that are consistent with regular buildings. These values may be used as approximations in lieu of calculated values.

   Commentary: The values for $PF_{Roof,1}$ in Table 8-16 are similar to the modal participation factors given in Table 8-8, with some differences at taller buildings. The values in Table 8-8 are slightly conservative one decimal place approximations.

8.5.5 Estimating the Fundamental Period of Vibration and the Mode Shape

General use structural analysis computer programs will produce the dynamic characteristics of a structure. For the case where this information is not available, the period and mode shape of the fundamental mode may be approximated by use of statistics. However, to complete this procedure force-displacement calculations are required. In other words, when the lateral forces are applied to the structure, calculations must be made to determine the lateral displacements of each story. The procedure is illustrated in Figure 8-78. The numbers are consistent with the example of Section 8.3.

8.5.5.1 Estimating the Mode Shape

Step 1: Static forces are applied to the building, using any reasonable distribution, in this case V of Table 8-12, but the distribution is in accordance with the code (i.e., wh/Σ wh with the F_t force at the roof).

Step 2: Deflections corresponding to the applied forces are obtained using a computer model of the building.

Step 3: The deflections are normalized by dividing by D_{roof}. This is assumed to be the mode shape, φ.

Table 8-16. α_1 and PF_{ROOF} Coefficients for Regular Buildings **(with uniform mass and straight line mode shape)**

Number of Stories	α_1	$PF_{ROOF,1}$
1	1	1
2	0.9	1.2
3	0.86	1.3
5	0.82	1.35
10 and greater	0.78	1.4

$$S_a = \left(\frac{1}{\alpha}\right)\left(\frac{V}{W}\right)$$

$$S_d = \delta_{ROOF} / PF_{ROOF,1}$$

Step 4: A new set of forces, proportional to $w\phi/\Sigma w\phi$ are applied to the building. Either masses m) or weights (w) may be used. The gravity, g, cancels out.

Step 5: Deflections corresponding to second iteration forces are obtained using a computer model. (Similar to step 2.)

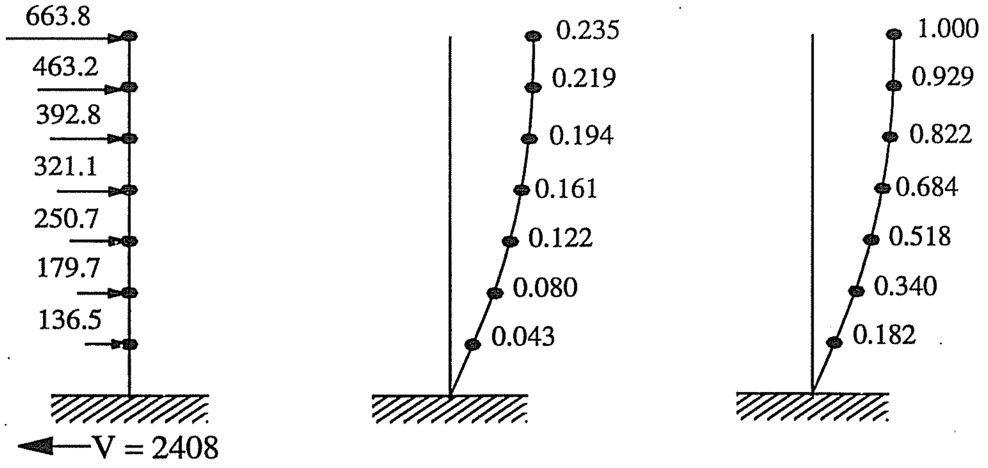
Step 6: The second iteration deflections are normalized by dividing by D_{roof} to get a revised mode shape. The process is repeated until the mode shapes converge (i.e., compare step 3 and step 6; all values are within 5 percent difference). For comparison, see Table 8-10 where step 6 is nearly identical to mode shape 1.

8.5.5.2 Estimating the Period of Vibration, T

The period can be calculated by use of equation 8-27, where:

$$T = 2\pi\sqrt{(\Sigma w_i \delta_i^2) \div (g\,\Sigma F_i\ \delta_i)}$$

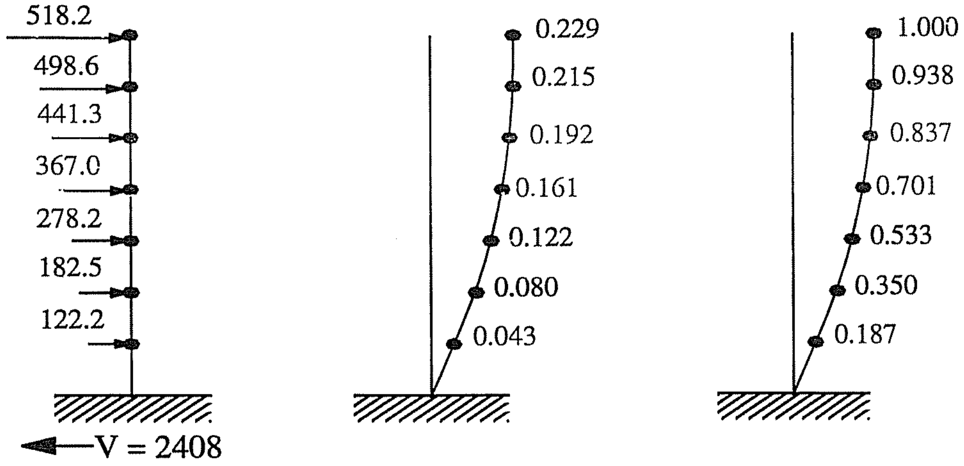
From Figure 8-78 use the forces in Step 4 for F_i and use the deflections in Step 5 for δ_i. The story weights, w_i, are found in Table 8-12.



Step 1: Apply static forces to the building.

Step 2: Calculate the deflections corresponding to the applied forces.

Step 3: Normalized the deflections by dividing by Δ_{roof}. This is assumed to be the mode shape, ϕ.



Step 4: Apply a new set of forces,proportional to $w\phi$ / $\Sigma w\phi$.

Step 5: Calculate the deflections corresponding to second iteration forces are obtained using a computer model.

Step 6: Normalize the second iteration deflections, and repeat the process until the mode shapes converge.

Figure 8-78. Estimating the Fundamental Mode Shape

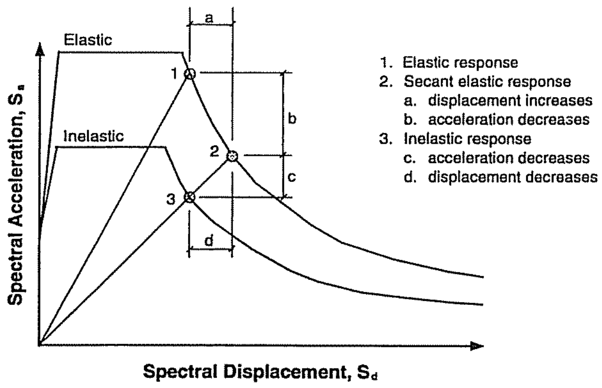


Figure 8-79. Response Spectrum Reduction: Inelastic vs Elastic Response (Long Period, Constant Velocity)

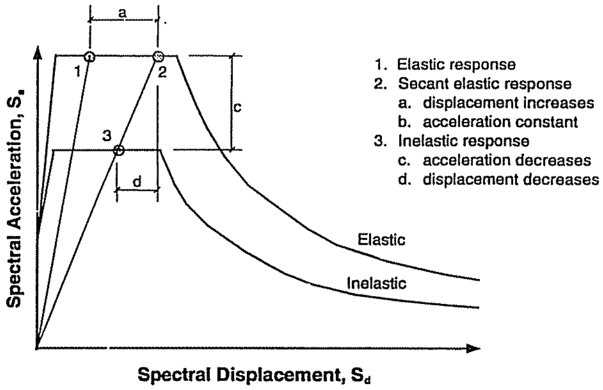


Figure 8-80. Response Spectrum Reduction: Inelastic vs Elastic Response (Short Period, Constant Acceleration)

Thus,

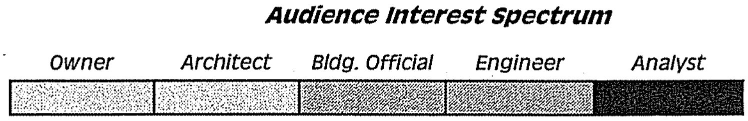
$$T = 2\pi\left[\frac{1410 * 0.229^2 + 1460 * 0.215^2 + \ldots + 1830 * 0.043^2}{g(518.2 * 0.229 + 498.6 * 0.215 + \ldots + 122.2 * 0.043)}\right]^{1/2} = 0.88 \text{ sec}$$

8.5.6 Explanation of Inelastic Response Reduction (SR_A, SR_V)

An elastic response spectrum can be reduced to an equivalent inelastic response spectrum. This topic has been the subject of much research. Figures 8-79 and 8-80 illustrate a simplified explanation of why there is an inelastic reduction, in other words, why inelastic response is generally less than elastic response.

On each figure, point 1 represents the elastic demand. If there is reduction in stiffness of the structure due to inelastic behavior, the effective period lengthens and the demand of the elastic spectrum is at point 2 (i.e., displacement increases by a value "a" and acceleration decreases by a value "b"). However, if the structure does behave inelastically (i.e., nonlinearly), the demand will be reduced to the inelastic response spectrum demand at point 3. Thus, there is a further reduction in acceleration by a value "c" and the displacement is reduced by a value "d". The net effect is that acceleration is reduced by "b" plus "c" and displacement is modified by "a" minus "d". If "a" is approximately equal to "d", inelastic displacement is equal to elastic displacement (Figure 8-79). If "a" is greater than "d" there is a net increase in displacement (Figure 8-80).

Chapter 9
Modeling Rules



9.1 General

This chapter presents rules for developing analytical models of existing concrete buildings. The rules are intended for use with a nonlinear static procedure of the type described in Chapter 8. As such, they address the full range of concrete element and component behavior, considering cracking, hinging, potential degradation, and loss of gravity resistance. The rules are based on principles of mechanics, observed earthquake performance, a broad range of experimental results, and engineering judgment.

Create a model with Chapters 9 and 10.
Analyze it with Chapter 8.
Compare results with limits in Chapter 11.

The following sections address loads; global building modeling; material models; element models, including frames, walls, diaphragms, and foundations; and component models, considering stiffness, strength, and deformability. Notation specific to this chapter is given in Section 9.6.

***Commentary:** Modeling rules presented in this chapter are intended to guide development of the analytical model used to evaluate an existing building or to design its retrofit. They are both qualitative and quantitative. Analytical building models based on these rules will be complete and accurate enough to support nonlinear static pushover analyses, described in Chapter 8, and acceptability limits, given in Chapter 11. The modeling rules will also support linear elastic analyses, described in Chapter 5. Additional considerations (regarding mass, damping, reversibility, etc.) may be required for dynamic time history analyses.*

Except for very simple buildings, analysis will usually rely on one or more specialized computer programs. Some available programs can directly represent the nonlinear load-deformation behavior of individual components, whereas others represent only linear response. In the latter case, a series of linear analyses must be carried out with component properties modified in each analysis to represent nonlinear response, the results being superimposed to obtain the nonlinear capacity curve. Some available computer packages will not directly model the degrading strength of individual components, in which case approximate approaches must be used.

9.2 Loads

9.2.1 Gravity Loads

The nonlinear analysis of a structure should include the simultaneous effects of gravity and lateral loads. Gravity loads should include dead loads and likely live loads.

Commentary: *The nonlinear response of a structure to lateral loads depends (in a nonlinear way) on the gravity loads present at the time of lateral loading. This dependence is illustrated in Figure 9-1, as follows:*

- *Considering the example beam (Figure 9-1a), the effect of light gravity load is to reduce the reserve moment and shear strengths at the right end and increase the reserve strengths at the left end (reserve strength is defined as the difference between the total strength and the resistance used up by gravity load). Therefore, for a given lateral drift, the gravity load will increase the inelastic rotation demands at the right end of the beam and decrease them at the left end. For larger gravity loads, the effects are increased, and the inelastic mechanism may shift from beam hinging at the ends to hinging along the beam span.*
- *For the example column (Figure 9-1b), variations in gravity load produce variations in column axial force, with consequent changes in both column strength and deformability. Increases in axial load invariably decrease flexural deformability. Increases in moment strength result in increased shear demands and may result in shear failure that would not be expected at lower axial loads.*

In general, because of the nonlinear nature of the interactions, it is not appropriate to carry out the gravity load analysis and lateral load analysis separately and then superimpose their results. Instead, the gravity loads should be applied to the numerical model and should be maintained as the lateral deformations are imposed.

Analysis for gravity load effects is complicated by the fact that live loads (and less frequently, dead loads) vary during the service life, and the magnitude at the time of the earthquake is generally unknown. Two approaches are commonly applied in nonlinear analyses. The first approach is to assume a range of gravity loads that bound the likely values, to carry out a nonlinear analysis for the bounding cases, and to use the most critical value from all the analyses. The second approach is to carry out one nonlinear analysis with gravity load set equal to the most likely value. The second approach is considered adequate in most cases and is recommended, except that the first case should be considered where live load is a significant proportion of the total load and where variations in live load are suspected to have a significant impact on the final assessment.

Dead load can be taken as the calculated structure self-weight without load factors, plus realistic estimates of flooring, ceiling, HVAC, partition, and other nonstructural weights.

Likely live loads should be evaluated for each structure; consideration should be given to current and expected future occupancies. Default values of typical live loading are provided in Table 9-1.

Table 9-1. Typical Service Live Loads for Various Occupancies

Occupancy	Live Load, psf[1]
General office area	13.6
Clerical area	16.9
Lobby	9.4
Conference room	11.1
File area	43.7
Storage area	28.9
Library	34.6
All rooms	17.8

1. Tabulated loads represent mean load plus one standard deviation (source: Culver 1976)

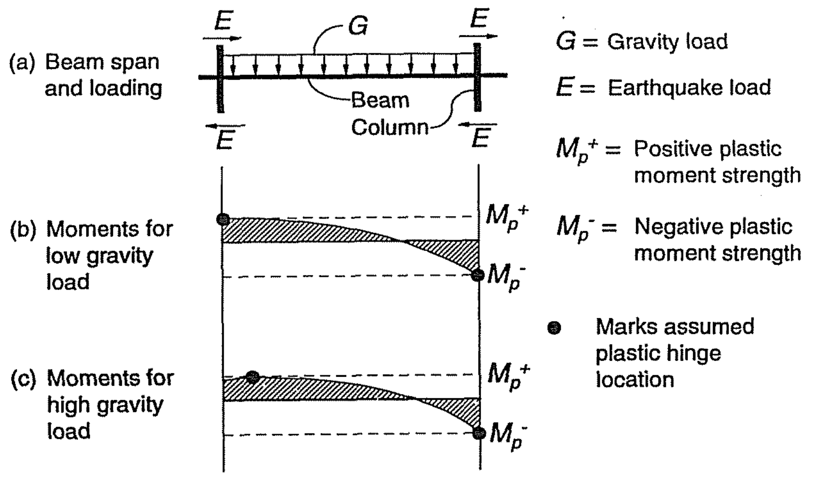


Figure 9-1a. Gravity Load Effect on Seismic Behavior of Components

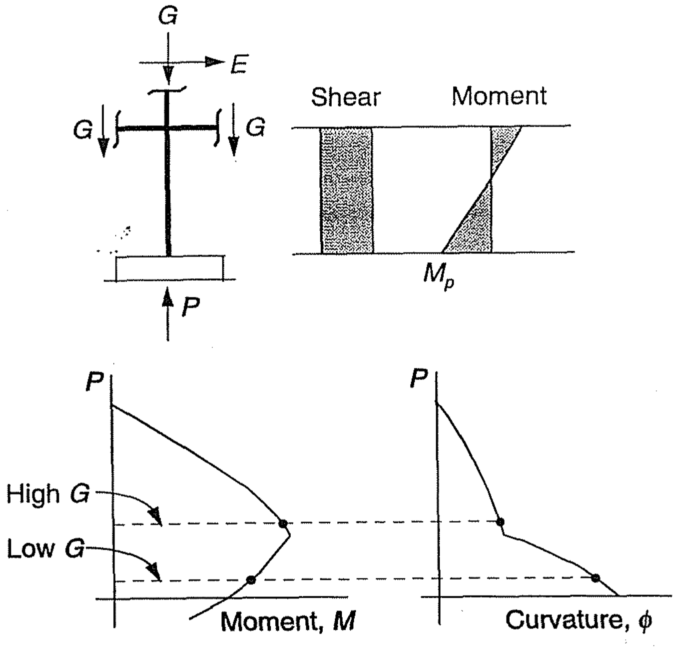


Figure 9-1b. Gravity Load Effect on Seismic Behavior of Components

The ability of a computer package to adequately represent gravity load effects should be determined as part of the analysis. Many nonlinear analysis programs will directly model the effects of gravity loads on stiffnesses and strengths. Others will not. In the latter case, it is usually possible to represent the effects by setting component strengths equal to reserve strengths, where reserve strength is equal to the calculated strength plus or minus the action induced by the gravity load. The plus or minus sign depends on whether the gravity load acts in the opposite or the same sense, respectively, as the lateral load.

9.2.2 Lateral Loads

Lateral loads should be applied in predetermined patterns that represent predominant distributions of lateral inertial loads during critical earthquake response. Chapter 8 defines relevant lateral load patterns. Lateral loads commonly may be lumped at floor levels. Lateral loads should be applied in increments that allow the engineer to track the development of the inelastic mechanism. Gravity loads should be in place during lateral loading. The effect of gravity loads acting through lateral displacements, the so-called P-Δ effect, should be modeled.

Example building studies in Volume 2 suggest ways to deal with software limitations.

Commentary: *As a structure is displaced laterally, its lateral load stiffness usually decreases with increasing lateral displacement. At large lateral displacements, the lateral load resistance may decrease with increasing displacement. Some computer programs for static inelastic lateral load analysis require that the lateral forces increase with each loading increment, a condition that cannot be met for a structure whose true strength is degrading. Therefore, the program might stop at the displacement corresponding to peak lateral load, even though the structure may be capable of larger displacements without collapse. In this case, it may be necessary to use special techniques to continue loading to larger displacements. See Section 9.5.1.*

9.3 Global Building Considerations

Analytical models for evaluation or retrofit must represent complete three-dimensional characteristics of building behavior, including mass distribution, strength, stiffness, and deformability, through a full range of global and local displacements. Two-dimensional models may be used if they adequately represent overall lateral response. Building models may be composed of simplified substructures derived from individual component properties as long as substructure forces and deformations are used to check local effects.

Commentary: Full three-dimensional static inelastic analysis often requires significant effort. Few available computer programs are able to directly model three-dimensional inelastic response of a structure. Furthermore, at the time of this writing, complete numerical models of component three-dimensional response are not well developed. Therefore, it is seldom justified to conduct a three-dimensional inelastic static analysis as part of the proposed methodology. Two-dimensional models are usually satisfactory.

Although the analysis may use two-dimensional models, certain aspects of the building's three-dimensional behavior should be considered. For example, the axial load in a corner column can be affected strongly by three-dimensional response because axial loads accumulate from the framing action of intersecting frame elements (Figure 9-2). Assuming that the building is displaced roughly

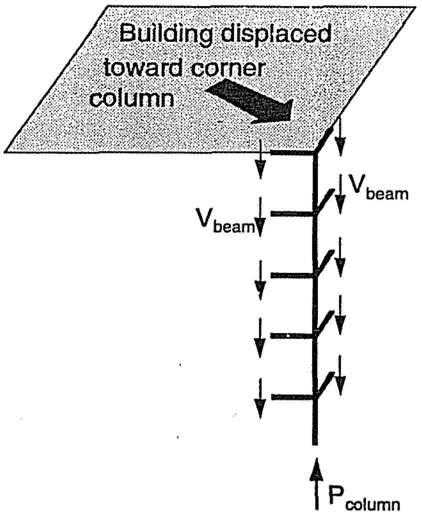


Figure 9-2. Axial Load In Corner Column Due to Load Transfer from Intersecting Frames

along a diagonal axis at some point during an earthquake, beams from frames in both directions may develop flexural plastic hinging, in which case the column axial load is equal to the sum over the building height of the beam plastic shears from both directions. The engineer may attempt to account for these effects directly by using a three-dimensional analysis model. Alternatively, it may be suitable to carry out a two-dimensional lateral load analysis and to modify the results to reflect aspects of expected three-dimensional response. For structures with stiffness or strength plan asymmetry, either a three-dimensional model or a two-dimensional model may be used. In either case, it is necessary to establish the demands considering torsional effects. Studies (Goel and Chopra 1991; Sedarat and Bertero 1990) show that actual inelastic torsional response tends to exceed results calculated using linearly-elastic dynamic analysis. Static inelastic methods and dynamic elastic methods are not able to adequately represent the full effect of torsional response. Response amplitudes associated with inelastic torsion may be much larger than those indicated by these approaches. For structures influenced by inelastic torsion, it often is more appropriate to use simple models or procedures to identify approximately the effect of the irregularity on torsional response, and to apply this effect independently to either a two- or three-dimensional static inelastic analysis of the building. Available research may provide insight into the required analysis process (Goel and Chopra 1991; Sedarat and Bertero 1990; Otani and Li 1984). Where inelastic torsional response is expected to be a dominant feature of the overall response, it usually is preferable to engineer a retrofit strategy that reduces the torsional response, rather than try to engineer an analysis procedure to represent inelastic torsion.

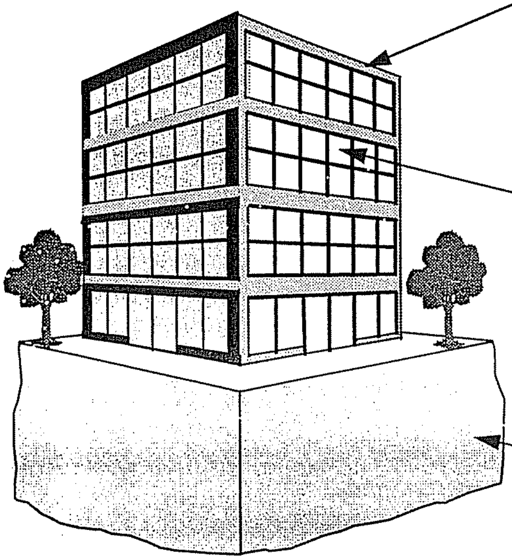
Substructuring involves the replacement of many components by single model elements or constraints. Examples include the modeling of complex coupled or perforated wall groups with an equivalent column and the representation of rigid floor diaphragms with slaved degrees of freedom. See Section 9.4.5 regarding diaphragm modeling.

Any structural, nonstructural, and soil elements that can affect the building assessment must be modeled. In addition, every component carrying gravity loads must be checked.

9.3.1 Building Model

The analytical model of the building should represent all new and existing components that influence the mass, strength, stiffness, and deformability of the structure at or near the expected performance point. Elements and components shown not to significantly influence the building assessment need not be modeled.

Building Elements



Structural Elements
- frames
- walls
- diaphragms

Non-structural Elements
- elements that influence structural behavior
- elements whose damage affects performance

Foundation Elements
- soil components
- structural components

Commentary: Elements and components expected still to provide lateral strength and/or stiffness after several cycles of earthquake loading must be modeled. The requirement to model the structure "at or near the expected performance point" is intended to facilitate the analysis by allowing exclusion of certain elements. For example, stiff, weak components expected to yield or degrade long before the overall lateral system reaches its limits and at a point on the capacity curve well in advance of the expected performance point, such as coupling beams or some wall segments, need not be modeled.

However, all components carrying gravity loads, even flexible framing not modeled, must be checked against deformation limits in Chapter 11, as either primary or secondary components. In Chapter 11, primary and secondary components are defined in terms of their significance to building performance levels, not in terms of relative stiffness or strength and not in terms of the need to model them explicitly.

Still, it is likely that most primary and some secondary elements will need to be modeled, at least initially. As analysis proceeds, it may become clear that some initially primary elements may be treated as secondary or, per the requirements of this section, treated as non gravity load carrying members with no specific deformation limits.

Some nonstructural elements (e.g., infills, stairs) can significantly modify the stiffness and strength of a reinforced concrete frame, and these elements should not be overlooked. Furthermore, damage or failure of these elements can affect structural assessment. As noted elsewhere, infilled frames are not considered in this document.

9.3.2 Soil-Structure Interaction

Behavior of foundation components and effects of soil-structure interaction should be modeled or shown to be insignificant to building assessment. Chapter 10 gives recommendations for modeling soil-structure interaction.

***Commentary:** Soil-structure interaction refers to response modification because of interaction effects, which could include reduction or increase in the target displacement, and modeling of the foundation-soil-superstructure system.*

Soil flexibility results in period elongation and damping increase. In the context of inelastic static analysis as described in this methodology, the main relevant impacts of soil-structure interaction are to modify the target lateral displacement and to provide additional flexibility at the base level that may relieve inelastic deformation demands in the superstructure. Because the net effect is not readily assessed before carrying out the detailed analysis, it is recommended that foundation flexibility be included routinely in the analysis model.

9.4 Element Models

9.4.1 General

An element is defined as either a vertical or a horizontal portion of a building that acts to resist lateral and/or vertical load. Common vertical elements in reinforced concrete construction include frames, shear walls, and combined frame-wall elements. Horizontal elements commonly are reinforced concrete diaphragms. Reinforced concrete foundations are elements with both vertical and horizontal aspects. Elements comprise components such as beams, slabs, columns, joints, wall segments, and others. Section 9.5 describes component modeling.

9.4.2 Concrete Frames

Concrete frame elements should be classified as either beam-column frames or slab-column frames. Slab-column frames may include capitals, drop panels, and drop caps. In the following discussion, frames are considered planar elements, although it should be recognized that intersecting frames interact with one another. In a typical planar model of a building, interaction effects from intersecting frames should be taken into account indirectly.

***Commentary:** Waffle slabs, shallow pan-joist systems, and slabs with "embedded beam"* *reinforcing within the slab depth may qualify as slab-column frames; some judgment is required.Some buildings have both slab-column and beam-column frames. For example, buildings with flat slabs and perimeter spandrel beams are likely to have slab-column frames along interior column lines and beam-column frames along perimeter column lines where the spandrels are.*

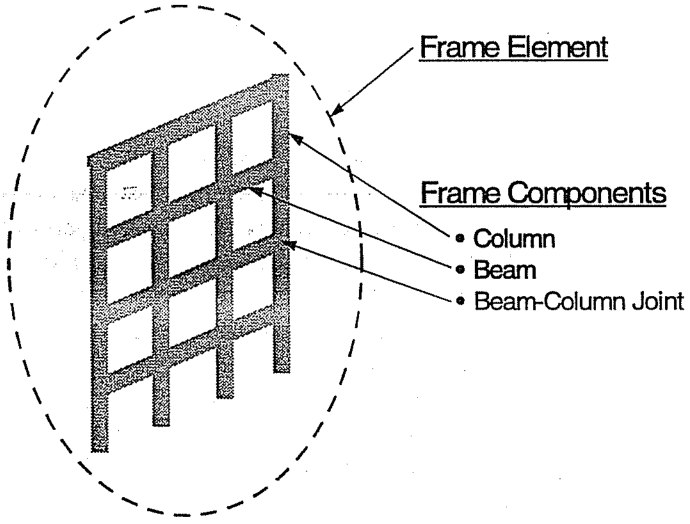
9.4.2.1 Beam-Column Frames

The analysis model for a beam-column frame element should represent the strength, stiffness, and deformation capacity of beams, columns, beam-column joints, and other components that may be part of the frame. Beam and column components should be modeled considering flexural and shear rigidities, although the latter may be neglected in many cases. Potential failure of anchorages and splices may require modeling of these aspects as well. Rigid beam-column joints may be assumed, except where joint strength may limit capacity development in adjacent components. Interaction with other elements including nonstructural elements should be represented.

9.4.2.1.1 Overall Geometry. The analytical model generally can represent a beam-column frame by using line elements with properties concentrated at component centerlines. In some cases the beam and column centerlines will not coincide, in which case a portion of the framing components may not be fully effective to resist lateral loads, and component torsion may result. Where minor eccentricities occur (the centerline of the narrower component falls within the middle third of the adjacent framing component measured transverse to the framing direction), the effect of the eccentricity can be ignored. Where larger eccentricities occur, the effect should be represented either by a concentric frame model with reduced effective stiffnesses, strengths, and deformation capacities or by direct modeling of the eccentricity. Where beam and column component cross sections do not intersect, but instead beams and columns are connected by transverse slabs or

Elements and Components

Elements are major vertical or horizontal parts of the building that act to resist lateral and vertical loads. Frames, diaphragms, walls, and foundations are examples of elements in a building. Elements are composed of **components.**



beams, the transverse slabs or beams should be modeled directly.

The beam-column joint in monolithic construction generally should be represented as a rigid zone having horizontal dimensions equal to the column cross-sectional dimensions and a vertical dimension equal to the beam depth. Where joint force levels reach nominal failure limits, the joint should be modeled by using a nonlinear spring element.

The model of the connection between the columns and foundation will depend on details of the column-foundation connection and the rigidity of the soil-foundation system.

The slab will act as a diaphragm that determines interactions among different frames. The slab will also act compositely as a beam flange in tension and compression; this action is to be represented in the beam component model. Section 9.5.4.2 recommends an effective width to be used in determining the stiffness and strength of a beam with a flange.

Nonstructural components that interact importantly with the frame should be modeled. Important nonstructural components that should be modeled include partial infills (which may restrict the framing action of the columns) and full-height solid or perforated infills and curtain walls (which may completely interrupt the flexural framing action of a beam-column frame). In general, stairs (which may act as diagonal braces) need not be modeled, but engineering judgment should be applied to unique cases; ramps in parking garages can add significant stiffness.

Commentary: *Conventional modeling assumptions are recommended. Beams and columns in older existing construction may frame eccentric to one another. The eccentricity may lead to torsional distress in the frame. Test data on eccentric connections and wide-beam connections (Joh et al. 1991; Raffaelle and Wight 1995; Gentry and Wight 1994) may guide definition of the analysis model. A slab connected monolithically with a beam will significantly influence the strength and stiffness of the beam, especially when the beam is flexed so that the slab is in tension; this effect needs to be taken into account (French and Moehle 1991).*

As described previously, infilled reinforced concrete frames are outside the scope of this document.

9.4.2.1.2 Modeling Local Response. The analytical models for beams, columns, and joints should be capable of representing the controlling deformation and failure modes. The requirements for stiffness, strength, and deformability limits are in Section 9.5.

Beams may develop inelastic response associated with flexure, shear, development, splices, and slip of bars embedded in joints. Torsion may be a consideration in a link beam that connects eccentric beams and columns. The analytical model should be developed to represent the likely modes of inelastic response.

Columns may develop inelastic response associated with flexure, axial load, shear, and development and splice failure. The analytical model should represent these potential modes where they may occur.

Beam-column joint strength may limit the forces that can be developed in the adjacent framing members. The primary failure mode of concern is joint shear failure. The analytical model should represent these potential modes where they may occur.

Commentary: *Likely modes of inelastic response may be identified by examining a simple free-body diagram of the isolated beam subjected to gravity loads and beam end rotations due to lateral loading. The process is illustrated in Figure 9-1a. In many cases, beam flexural strength will vary along the span, and this may influence how inelastic response develops. This plastic hinging pattern can be identified for individual members as illustrated in Figure 9-3. Flexural strength (broken line in Figure 9-3b) is calculated by the usual procedures, with rebar stress capacities limited if necessary on the basis of available development length. As a starting point, it is assumed that plastic hinging is at member ends, so the plastic*

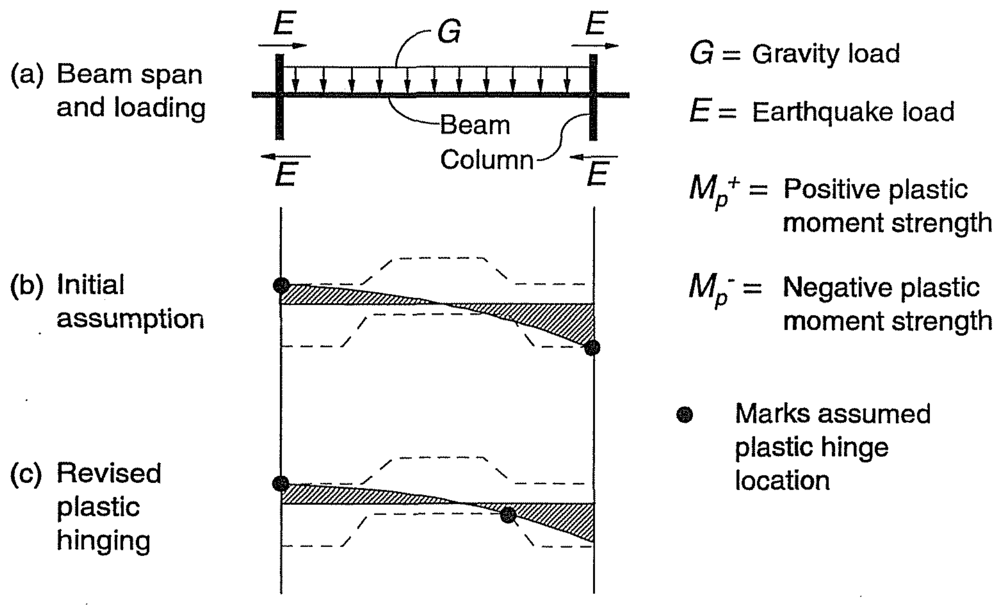
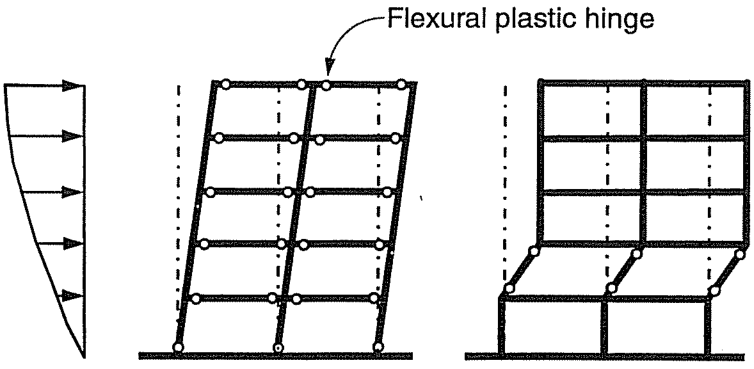


Figure 9-3. Procedure to Identify Plastic Hinge Location in Horizontal Spanning Components

moment capacities are assigned at those locations. The static moment diagram is then constructed considering gravity load. If the static moment diagram exceeds strength along the span, as in Figure 9-3b, then it is likely that plastic hinging occurs along the span, not at the ends. The moment diagram and plastic hinging locations are revised as shown in Figure 9-3c. To model this behavior it may be necessary to assign nodal degrees of freedom along the span so that plastic hinging can occur and be monitored at the interior nodes.

Where inelastic flexure is the controlling mode, this response may be represented directly by using concentrated or distributed hinge models (Spacone et al. 1992). Most computer codes do not provide a ready and direct means of representing shear and bond failures. These may be represented by modifying the flexural resistance to correspond to the value at which the shear or bond failure is likely to occur. For example, in many older frames the beam bottom reinforcement will be embedded only a short distance into the joint. Although slip of this reinforcement is strictly a bond failure, its effects can be represented in the analysis model by calculating the stress capacity of the embedded bars as described in Section 9.5.4.5 and setting the moment strength equal to the moment resistance corresponding to that stress capacity.

Beam plastic hinging may be represented directly in computer programs that model inelastic response. Alternatively, the same effect may be achieved in computer programs that model only linearly elastic response. In the latter case, the analysis is run until yield is reached at one or more locations. To model post yield response, a hinge or very flexible spring is inserted at the yielded location and analysis is continued until subsequent yielding occurs. The process is repeated, and the results are superimposed to obtain the complete solution. Where linear models



(a) Loading (b) Beam sway mechanism (c) Column sway mechanism

Figure 9-4. Idealized Flexural Mechanisms in Multi-Story Frames

are used, the use of very flexible springs (rather than hinges) has the advantage that hinge rotation, θ, can be monitored as $\theta = \frac{M}{K}$*, where M is the moment and K is the rotational stiffness of the spring.*

Considerations in modeling the response of a column are similar to those for beams, described above. A widely held misconception is that column flexural yielding is not possible if the sum of the column strengths exceeds the sum of the beam strengths at all connections. While it is true that strong columns promote formation of beam-sway types of mechanisms (Figure 9-4), column flexural yielding at the foundation and at intermediate levels is still possible. Therefore, the analytical model should allow for column hinging at all levels of the building. Inelastic flexure along the unsupported length of the column is not usually a consideration because there is no significant lateral load applied along the unsupported length of the column. The possibility of shear, splice, or development failure along the column length should not be overlooked, however.

As a building is loaded laterally, column axial loads will change, especially for perimeter columns. Preferably, the column analytical model will directly incorporate interaction between axial load and flexural strength. Where the model does not account for this action directly, the analyst should manually modify flexural strengths to represent values corresponding to expected axial loads, and results should be verified.

The potential for joint failure can be investigated on a free-body diagram of the joint and adjacent framing components subjected to representative gravity and lateral load effects. Where adjacent component strengths are sufficient to induce shear failure in the joint, joint hinging should be modeled. Most inelastic analysis computer programs have nonlinear rotational spring elements that can be inserted between columns and beams to model the joint. Alternatively, one may limit the strengths of adjacent framing components to values corresponding to the development of joint shear failure. Note that when a joint fails, the connection to all adjacent beams and columns is lost. Representing this aspect may be important to determining performance and may require additional modeling efforts.

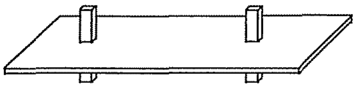
9.4.2.2 Slab-Column Frames

The analysis model for a slab-column frame element should be sufficiently detailed to represent the strength, stiffness, and deformation capacity of slabs, columns, slab-column joints, and other components that may be part of the frame. Slab and column components should be modeled by considering flexural and shear rigidities, although the latter may be neglected in certain cases. The potential failure of anchorages and splices may require the modeling of these aspects as well. Slab-column joints (that is, the volume of concrete common to the slab and column, including the capital) may be assumed to be rigid. Interaction with other elements, including nonstructural elements, should be represented.

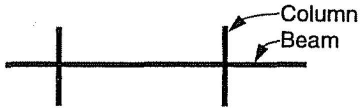
Commentary: Conventional practice in regions of high seismicity is to ignore the contributions of the slab-column frame to lateral load resistance. This approach is inappropriate for the methodology proposed in this document because the slab-column frame may provide appreciable lateral stiffness and strength that may reduce retrofit requirements, and because slab-column frame damage or collapse must be recognized in the overall performance evaluation. Therefore, the slab-column framing system should be included directly in the analysis and assessment of the building lateral and vertical force resisting systems.

9.4.2.2.1 Overall Geometry. The following three approaches to modeling slab-column frames are specifically recognized and are illustrated in Figure 9-5:

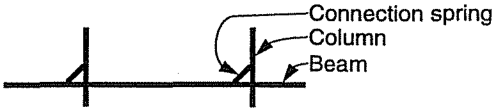
- *Effective Beam Width Model.* Columns and slabs are represented by frame elements that are rigidly interconnected at the slab-column joint.
- *Equivalent Frame Model.* Columns and slabs are represented by frame elements that are interconnected by connection springs.
- *Finite Element Model.* Columns are represented by frame elements and the slab is represented by plate-bending elements.



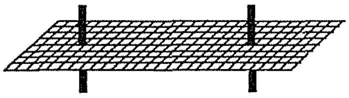
(a) Actual slab-column frame



(b) Effective beam width model



(c) Equivalent frame model



(d) Finite element model

Figure 9-5. Slab-Column Framing Models

In any model, the effects of changes in cross section, including slab openings, should be considered.

Commentary: The main structural components of slab-column frames are slabs, columns, slab-column joints, and the slab-column connection. In most cases, slab-column joints are not critical. Refer to relevant material on beam-column joints for special cases where slab-column joints may have high shear stresses. The slab-column connection commonly is a critical component in the system. It comprises the region of slab immediately adjacent to the column. Shear failure of the slab associated with shear and moment transfer can result in progressive collapse in cases where slab bottom reinforcement (or post tensioned strand) is not continuous through the column. See the report by ACI-ASCE Committee 352 (ACI 1988) for further information.

The effective beam width model, the equivalent frame model, and the finite-element model are illustrated in Figure 9-5. For each of these three models, the column is represented by a line element having stiffness properties defined by conventional methods. The effective beam width model represents the slab as a flexural member having stiffness reduced to represent the indirect framing between slab and column as well as slab cracking. The equivalent frame model represents the slab by a flexural member that connects to the column through a transverse torsional member. Finite-element models represent the flexural, shear, and torsional response of the slab directly. For each of the three models, the stiffness should be adjusted from theoretical values based on the gross cross section because of the significant effects of slab cracking on response (Vanderbilt and Corley 1983). Details on effective stiffnesses are in Section 9.5.3.

The effective beam width model, while simple to use, has a drawback in that there is no component to monitor directly the shear and moment transfer between slab and column, and this is an important aspect in checking performance. The finite-element model has certain advantages, but has relatively high computational cost. In most cases, it may be preferable to use an equivalent frame model because it is relatively simple to implement and it provides a component to directly monitor shear and moment transfer.

Other aspects of modeling are similar to those of beam-column frames, as discussed in Section 9.4.2.1. These aspects include the connection between the columns and foundation, the action of the slab as a diaphragm, and the interaction with nonstructural components.

Punching failure of slabs adjacent to columns must be considered, especially when slab bottom bars are not continuous through the joint.

9.4.2.2.2 Modeling Local Response. The analytical models for slabs, columns, and slab-column connections should be capable of representing the controlling deformation and failure modes.

The main deformations in the slab usually include flexure along the slab length and flexure and twisting action in the slab adjacent to the column (the connection region). Prominent failure modes include punching shear failure due to shear and moment transfer, flexural failure, and failure due to inadequate bar details. Furthermore, progressive collapse may result where one connection fails by punching. The analytical model should represent these effects as appropriate, depending on the details and proportions of the slab.

Column modeling should follow the guidelines for beam-column frames (Section 9.4.2.1).

Commentary: The general approach for modeling flexural, shear, and bond behaviors of slabs and columns is similar to that described for beam-column frames in Section 9.4.2.1.

The potential for slab-column connection failure should be investigated on a free-body diagram of the slab-column connection subjected to representative gravity and lateral load effects. Where adjacent component strengths are sufficient to induce failure in shear and moment transfer, this failure mode should be modeled. It is possible with most computer programs to represent connection failure directly by using a rotational joint spring element connecting the slab and the columns. The spring element is initially very stiff and yields at a moment corresponding to the development of the critical actions.

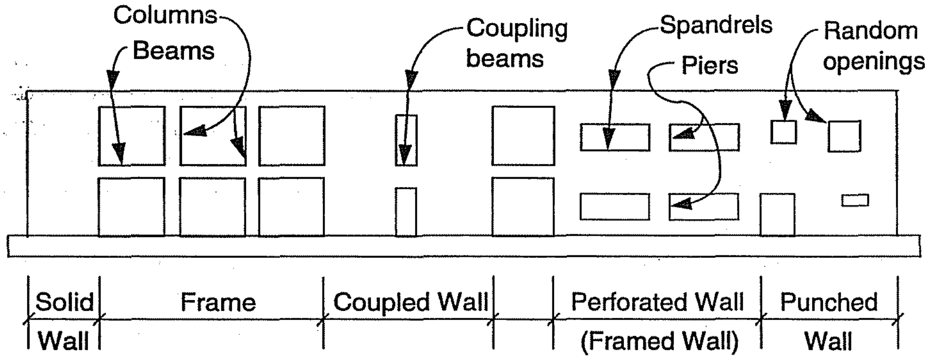
Failure of a slab-column connection may result in complete punch-through at that connection, *leading to gravity load transfer to adjacent connections, which subsequently may lead to progressive collapse. Progressive collapse is generally avoided if slab bottom bars are continuous through the connection in conventionally reinforced slabs, or if slab bottom bars or draped tendons are continuous through the connection in post-tensioned slabs. If these conditions are not met, the potential for progressive collapse should be modeled directly or investigated with a separate analysis.*

9.4.3 Concrete Shear Walls

Concrete wall elements should be classified as solid walls, punched walls, perforated walls, or coupled walls and should be classified further as continuous or discontinuous. Solid walls do not contain significant openings. Punched walls have significant openings that are not aligned vertically. Perforated walls are made of vertical and horizontal wall segments that are arranged in a regular pattern; these are sometimes referred to as framed walls. Coupled walls are a special case of perforated walls where two or more walls are interconnected by horizontal framing components. Discontinuous walls do not extend to the foundation but are supported instead on beams, columns, or other components. Where walls intersect to form flanged walls, the effect of flanges on stiffness and strength should be included according to the recommendations of Sections 9.5.3 and 9.5.4. Important wall interactions with adjacent wall and frame elements, especially discontinuous walls and elements aligned with other building axes, should be identified.

In coupled walls and perforated walls, the vertical components will often be referred to as wall piers. The horizontal components of coupled walls will be referred to as coupling beams, whereas those of perforated walls will be referred to as spandrels. Piers, coupling beams, and spandrels will also be referred to as wall segments, in which case the term refers to all these components.

Frame and Wall (Vertical) Elements of Buildings



9.4.3.1 Solid Walls

The analysis model for a solid wall element should represent the strength, stiffness, and deformation capacity of the wall for in-plane loading. Out-of-plane behavior need not be considered, except where the wall acts as a flange for an intersecting wall element. Solid walls may be considered "slender" if their aspect ratio (height/length) is equal to or exceeds 4 ($h_w/l_w \geq 4$). Solid walls may be considered "squat" if their aspect ratio is less than or equal to 2 ($h_w/l_w \leq 2$). Slender walls usually are controlled by flexural behavior, although shear strength may be a limiting factor in some cases. Squat walls usually are controlled by shear behavior, although flexure sometimes may be a limiting factor. The response of walls with intermediate aspect ratios usually is influenced by both flexure and shear. Potential failure of anchorages and splices may require modeling of these aspects as well. Interaction with other elements, including nonstructural elements, should be represented. Except for squat one- and two-story walls, sliding along construction joints need not be modeled.

> *"Equivalent columns" are usually acceptable for modeling walls, but other approaches may be more accurate.*

9.4.3.1.1 Overall Geometry. The analytical model can represent a solid wall with an equivalent wide column element located at the wall centerline, using multi, spring models, truss models, or planar finite elements. Where sliding shear strength at a horizontal construction joint limits the shear capacity of the wall, this behavior can be modeled with a yielding spring in series with the wall panel.

Commentary: *The equivalent column model is more appropriate for slender walls than for squat walls, although successful results have been obtained even for very low aspect ratio walls (Sozen and Moehle 1993). If a wall yields in flexure, or if the foundation yields, continued lateral deformations involve plastic rotations centered near the compression toe of the wall, with uplift occurring toward the tension side. The equivalent column model cannot represent this effect, as the equivalent column is located at the wall centerline rather than the toe. This can lead to inaccuracies in representing interactions with adjacent components that may be affected by uplift.*

Where interactions with adjacent elements are considered important, it may be preferable to represent the wall by using more-sophisticated techniques that represent the width of the wall. Multi-spring models (Otani 1980; Vulcano et al. 1989; Otani et al. 1985; Alami and Wight 1992; Charney 1991) may be considered. These models use two or three vertical springs to represent the axial and flexural stiffnesses and strengths of the wall, plus at least one horizontal or diagonal spring to represent the shear stiffness and strength of the wall. Other models that adequately account for flexural, shear, and rigid-body deformations also may be used.

The model of the connection between the wall and foundation will depend on details of the wall-foundation connection and the rigidity of the soil-foundation system.

9.4.3.1.2 Modeling Local Response. The analytical model should be capable of representing the controlling deformation and failure modes. The requirements for stiffness, strength, and deformability limits are in Section 9.5.

Walls can develop inelastic response associated with flexure, shear, development, splices, and foundation rotations. The analytical model should represent the likely modes of inelastic response.

Commentary: *Diagonal tension cracks can develop in walls at moderate levels of shear stress, and these can lead to deficiencies if horizontal reinforcement is inadequate. If the wall contains adequate horizontal reinforcement and shear* *stresses are high, the concrete may crush because of diagonal compression. When loading is reversed, crushing may occur at the other end of the wall, and after several cycles of loading, the crushed concrete can extend over a significant length of the wall. This can lead to a major loss of strength.*

Sliding shear failure can occur at a weak plane, such as a construction joint, or along flexural cracks that have interconnected after several cycles of loading to form a shear failure plane. At this point, shear is transmitted by shear friction and dowel action. After continued loading cycles, the shear friction resistance will deteriorate. Also, sliding along the joint may lead to kinking of the vertical bars and subsequently to bar fracture.

However, while limited sliding along construction joints is frequently observed after earthquakes, it is not expected to control building behavior except perhaps in long, squat low-rise walls whose nominal shear and flexural strength is exceptionally high by comparison. It is expected that the construction joint will be the weakest part of a typical wall and that some horizontal sliding might occur, but that the joint will then stiffen and allow the nominal wall strength to develop. Thus, unless sliding over many cycles is sufficient to fracture vertical wall reinforcing, the net effect of limited sliding is to dissipate energy without much damage or loss of capacity. Therefore, it is usually conservative to omit construction joints from the model. Additionally, rational construction joint models (based on dowel action and aggregate interlock) underestimate the capacity and post yield behavior observed after earthquakes; useful, reliable modeling rules are not available.

Lower standards for lapping reinforcement were customary when many older concrete buildings were constructed. In shear walls where flexural behavior predominates, insufficient boundary steel laps can limit the moment strength of the wall.

These aspects of wall behavior usually may be identified by examining a simple free-body diagram of the isolated wall subjected to gravity loads and likely lateral loads, as described previously for beams (Section 9.4.2.1). Flexural action should be checked at points of maximum moment (usually the base, and possibly at some intermediate levels where there is significant frame-wall interaction) and at points where concrete or reinforcement changes. Potential shear failure may be associated with the shear strength of the wall panel, where it may be due to diagonal compression or diagonal tension action, or with shear sliding at the base and other locations where construction joints are used. Splices of longitudinal reinforcement should be checked to determine whether lengths and confinement are adequate for the expected force and deformation demands. The wall model should be configured to represent these possible effects where they are important. Nodal points are required at the base, at floor levels (where adjacent elements interconnect and where loads are applied), and at intermediate locations where inelastic response is possible.

Preferably, the wall analytical model will directly incorporate interaction between axial load and flexural strength. Where the model does not account for this action directly, the analyst should manually modify flexural strengths to represent values corresponding to expected axial loads, and results should be verified. Axial loads should be determined by considering applicable gravity loads plus interaction effects with adjacent frames and walls.

9.4.3.2 Coupled, Perforated, or Punched Walls

The analysis model for coupled, perforated, or punched wall elements should represent the strength, stiffness, and deformation capacity of the wall, wall segments, and pier-spandrel connections. Considerations are generally similar to those for solid walls. Considerations for wall segments such as coupling beams, piers, and spandrels should include relevant aspects of walls and beams, depending on relative proportions and reinforcement details. Considerable judgment and detailed local analyses may be required to determine the nature of overall behavior.

***Commentary:** Modeling and evaluation procedures for perforated and punched walls must be established with considerable engineering judgment. Behavior and analysis requirements are likely to depend on the relative sizes of piers, spandrels, and openings. Perforated or punched walls may behave essentially as beam-column frames or as solid walls with many intermediate variations. Some insight into behavior often can be obtained by studying the results of linear elastic (or nonlinear if practical) finite-element models of portions of the wall element. It may be feasible to establish a strut and tie model to represent overall nonlinear behavior of a punched wall (Yanez et al. 1992).*

> Detailed subassembly models, as well as considerable engineering judgment, may be needed in developing appropriate models for punched and perforated walls.

9.4.3.2.1 Overall Geometry. The analytical model can represent walls and wall segments with equivalent beam and column line models, multi spring models, truss models, or planar finite elements. Line models of short columns and deep beams should incorporate both bending and shear stiffness. For perforated and punched walls, stiffness representations should be based on preliminary subassembly studies with planar finite elements. Simplified models with line elements may be inappropriate for some punched walls.

For common proportions (individual walls considerably stronger and stiffer than individual coupling beams), a coupled wall should have flexibility along its full height *without* rigid vertical segments within the depth of the coupling beams. Coupling beams should be modeled to connect to the boundary of the wall.

9.4.3.2.2 Modeling Local Response. The analytical model should be capable of representing the controlling deformation and failure modes. General modeling considerations are the same as those described in Section 9.4.3.1. Requirements for stiffness, strength, and deformability limits are given in Section 9.5.

Coupled walls (including the wall piers and coupling beams) may develop inelastic response associated with flexure, shear, development, splices, and foundation rotations. Perforated walls (including the wall piers, spandrels, and spandrel-pier connections) may develop inelastic response associated with flexure, shear, development, splices, and foundation rotations. Punched walls may develop inelastic response associated with flexure, shear, development, splices, and foundation rotations.

Commentary: Coupling between walls typically results in significant variations in wall axial force under lateral loading. This effect should be considered when defining stiffnesses and strengths. If the coupling beams have flanges (for example, as may occur if the floor slab frames into the beam), the effects of the slab on stiffness and strength should be included in the model.

9.4.3.3 Discontinuous Walls

The analysis model for a discontinuous wall element should represent the strength, stiffness, and deformation capacity of the wall and the supporting components.

***Commentary:** The supporting columns or beams may be subjected to significant forces and deformations. The potential for failure of these components should be carefully represented, as failure of these components has been the cause for complete building collapse in past earthquakes. In addition, at the level of the discontinuity, it may be* *necessary to model the flow of shear forces from one plane of vertical resistance to others through the floor diaphragm.*

9.4.3.3.1 Overall Geometry. Aspects of modeling the wall and the supporting components are covered elsewhere in this methodology. In addition, it is necessary to properly represent the flow of forces from one component to another.

Commentary: *Where discontinuous walls are supported on other components or elements, there may be considerable force and deformation demands on the supporting components, and there may also be considerable stress concentrations where the two intersect. These cases must be represented properly in the analytical model.*

9.4.3.3.2 Modeling Local Response. The analytical model should be capable of representing the controlling deformation and failure modes. The requirements for stiffness, strength, and deformability limits are in Section 9.5.

Discontinuous walls may develop inelastic response associated with flexure, shear, development, splices, and foundation rotations. The analytical model should represent the likely modes of inelastic response. Modeling considerations are the same as those described in Section 9.4.3.1.

9.4.4 Combined Frame-Wall Elements

The analysis model for a combined frame-wall element should represent the strength, stiffness, and deformation capacity of the wall, the frame, and the interconnections. Considerations for the walls and the frames are in Sections 9.4.2 and 9.4.3.

9.4.4.1 Overall Geometry

The analytical model should properly represent the interconnection between the frame and wall sub-elements. In most cases, beams will frame into the edge of the wall; where the wall is modeled by using a line element at the wall centerline, a rigid or nearly rigid offset should be provided to represent the fact that the beam connects to the wall edge. Refer to Sections 9.4.2 and 9.4.3 for details on frame and wall modeling.

9.4.4.2 Modeling Local Response

Refer to Sections 9.4.2 and 9.4.3 for details on frame and wall modeling.

9.4.5 Concrete Floor Diaphragms

The analysis model for a floor diaphragm should represent the strength, stiffness, and deformation capacity for in-plane loading. Diaphragm axial, shear, and flexural deformations should be modeled unless the diaphragms can be considered rigid and are strong enough to remain essentially elastic under the applicable earthquake loads. The model should allow assessment of diaphragm shear, flexure, anchorage, splicing, and connections to vertical components. In general, the evaluation or retrofit design must consider how the diaphragm connects vertical and lateral force resisting elements and how it braces elements subject to out-of-plane loads or deformations.

Commentary: *This methodology considers only cast-in-place concrete diaphragms; precast concrete diaphragms are not covered explicitly. Concrete floor diaphragms are composed of slabs, struts, collectors, and chords.*

Slabs commonly serve multiple purposes; they are a part of the floor or roof system to support gravity loads, they function as tension and compression flanges for floor beams, and they act as a part of the horizontal diaphragm. In its capacity as a part of a diaphragm, the floor slab may develop shear, flexural, and axial forces associated with the transmission of forces from one vertical lateral force resisting element to another, or with the slab action as a bracing element for portions of the building that are loaded out of plane.

Struts and collectors are built into diaphragms where the defined stress demand exceeds the capacity of the diaphragm without them. Typical locations include around openings, along defined load paths between lateral load resisting elements, and at intersections of portions of floors that have plan irregularities. They transmit primarily axial forces but may also carry shear and bending forces.

Diaphragm chords, usually at the edges of a horizontal diaphragm, function primarily to resist in-plane bending action of the diaphragm. Tensile forces typically are more critical, but compressive forces in thin slabs can be a problem. Exterior walls can serve the function of the diaphragm chord if there is adequate horizontal shear capacity between the slab and wall.

The analytical model often can represent the diaphragm as a continuous or simple-span horizontal beam that is supported by vertical elements of varying stiffness. Most computer programs assume a rigid diaphragm. The adequacy of this assumption should be checked.

Modeling rules should be determined by considering the relative flexibility of the diaphragm and vertical supporting elements. Where diaphragm nonlinearity is anticipated, this effect should be represented. The modeling procedures presented for frames and walls (Sections 9.4.2 and 9.4.3) provide general guidance on modeling issues for diaphragms.

9.4.6 Foundations

The analytical model should allow assessment of soil and structural foundation components and should represent the nonlinear response of the foundation system. The response of the foundation system can be represented with simple elasto plastic models. For simplicity, foundations may be represented as rigid footings, flexible strip footings, pile foundations, or drilled shafts. Appropriate models for equivalent linear stiffness and strength should be employed depending on the foundation type. The effects of foundation deformations on structure response should be taken into account. Chapter 10 presents details on foundation effects.

9.5 Component Models

9.5.1 General

This section applies to the reinforced concrete components of the structural model. While soil and nonstructural components must be considered for their effects on structural elements, specific modeling rules are not presented here. Refer to Chapters 10 and 12.

Section 9.5.2 discusses assumptions for material properties. Sections 9.5.3, 9.5.4, and 9.5.5 give guidelines for modeling component initial stiffnesses, ultimate strengths, and deformation capacities. In general, stiffnesses, strengths, and deformabilities of structural components may be calculated on the basis of the principles of mechanics of materials as verified by tests or may be calculated on the basis of the preset rules described in this methodology. In all cases, calculations for existing components should be based on the best available estimates of material properties and should use the best available analytical models, except where simplified models provide reasonable economy and accuracy. Calculations for new materials added as part of a retrofit may be based on nominal properties and calculation procedures contained in codes for the design of new construction.

Commentary: In general, the model must represent the stiffnesses, strengths, and deformabilities of structural components. Two specific approaches are presented. One approach is to calculate relevant properties directly by using basic principles of mechanics as verified by experimental results. The second approach is to use preset modeling rules described in detail in this chapter; these rules were derived by the project team on the basis of available test data, analytical methods, and engineering judgment. Some combination of the two approaches is permissible and is likely to be used in a typical building analysis.

The conclusion that an existing structure does not meet specified performance objectives can carry with it considerable consequences. Therefore, it is important that evaluation be based on the best available information on the properties of materials and components rather than on very conservative assumptions. In this document, for the evaluation of existing materials, the general approach is to use expected material and *component strengths as opposed to nominal design values from codes for new construction. Conventional methods for calculating stiffness, strength, and deformability are endorsed where these are deemed to provide a reasonably good estimate of actual component properties. Otherwise, alternative methods are presented. If the building is to be retrofit, new materials may be modeled with conventional design values specified in codes for new construction.*

Existing buildings often contain details and proportions that differ considerably from those permitted for new building designs. Many of these conditions have not been tested in the laboratory. Furthermore, many may involve brittle or degrading response modes whose behaviors are widely variable and difficult to predict with accuracy. The engineer should be aware that actual behavior may vary from calculated behavior. In critical cases, the engineer should investigate response for a range of likely component properties so that worst-case, but reasonably conceivable, building responses can be identified. In less critical cases, it is acceptable to assume single, best-estimate values for stiffness, strength, and deformability.

The following sections present procedures for modeling materials and components. In some cases, the procedures differ from more-conventional procedures such as those specified in ACI 318 (ACI 1995). The different procedures are intended to provide greater accuracy. In some cases, the improved accuracy requires additional computational effort. Where no guidance is given, the engineer should use the procedures specified in ACI 318, except that for existing components the expected materials strengths should be used as opposed to the design values specified in ACI 318.

Component behavior generally will be modeled using nonlinear load-deformation relations defined by a series of straight-line segments. Figure 9-6 illustrates a typical representation. In this figure, Q_c refers to the strength of the component and Q refers to the demand imposed by the earthquake. As shown in that figure, the response is linear to an effective yield point, B, followed by yielding (possibly with strain hardening) to point C, followed by strength degradation to point D, followed by final collapse and loss of gravity load capacity at point E. More-refined relations are acceptable but are not described in detail in this document. It is acceptable to use a simple bilinear model that includes only line segments A-B-C if the analysis ensures that response does not extend beyond point C for any of the components (Derivation of generalized load-deformation relations from cyclic test results is described in Section 9.5.4.1).

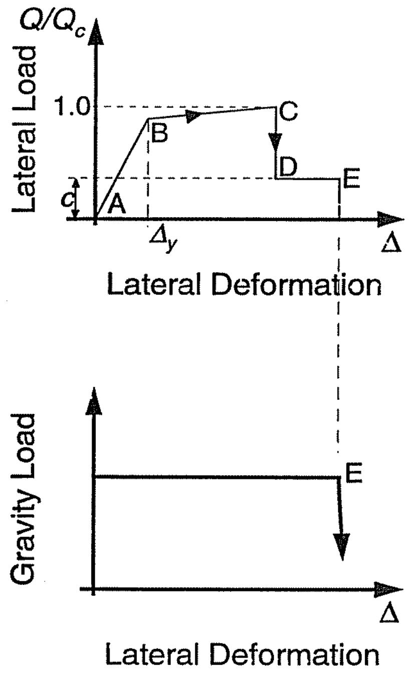


Figure 9-6. Generalized Load-Deformation Relations for Nondegrading Components

***Commentary:** Figure 9-6 illustrates a generalized load-deformation relation appropriate for most concrete components. The relation is described by linear response from A (unloaded component) to an effective yield point B, linear* *response at reduced stiffness from B to C, sudden reduction in lateral load resistance to D, response at reduced resistance to E, and final loss of resistance thereafter. The following main points relate to the depicted load-deformation relation:*

- *Point A corresponds to the unloaded condition. The analysis must recognize that gravity loads may induce initial forces and deformations that should be accounted for in the model. Therefore, lateral loading may commence at a point other than the origin of the load-deformation relation.*

Nondegrading concrete components follow the general relation of Figure 9-6. A simpler bilinear model is acceptable as long as ultimate deformations are carefully monitored by the engineer.

- *The slope from A to B should be according to the discussion in Section 9.5.3.*
- *Point B has resistance equal to the nominal yield strength. Usually, this value is less than the nominal strength.*
- *The slope from B to C, ignoring the effects of gravity loads acting through lateral displacements, is usually taken as between 5% and 10% of the initial slope. This strain hardening, which is observed for most reinforced concrete components, may have an important effect on the redistribution of internal forces among adjacent components.*
- *The ordinate at C corresponds to the nominal strength defined in Section 9.5.4. In some computer codes used for structural analysis, it is not possible to directly specify the value of resistance at point C. Rather, it is possible only to define the ordinate at B and the slope for loading after B. In such cases, results should be checked to ensure that final force levels following strain hardening are consistent with expected resistance for that deformation level. Strain hardening to values exceeding the nominal strength should be avoided.*
- *The abscissa at C corresponds to the deformation at which significant strength degradation begins. Beyond this deformation, continued resistance to reversed cyclic lateral forces can no longer be guaranteed. For brittle components, this deformation is the same as the deformation at which yield strength is reached. For ductile components, this deformation is larger than the yield deformation. Gravity load resistance may or may not continue to deformations larger than the abscissa at C.*
- *The drop in resistance from C to D represents initial failure of the component. It may be associated with phenomena such as fracture of longitudinal reinforcement, spalling of concrete, or sudden shear failure following initial yield. Resistance to lateral loads beyond point C usually is unreliable. Therefore, primary components of the lateral force resisting system should not be permitted to deform beyond this point.*
- *The residual resistance from D to E may be non-zero in some cases and may be effectively zero in others. Where specific information is not available, the residual resistance usually may be assumed to be equal to 20% of the nominal strength. The purpose of this segment is to allow modeling of components that have lost most of their lateral force resistance but that are still capable of sustaining gravity loads.*

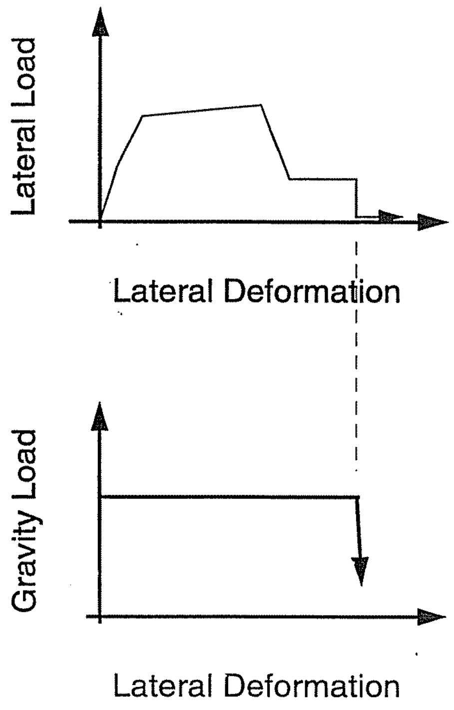


Figure 9-7. Alternative Idealized Load-Deformation Relation for a Component

- *Point E is a point defining the maximum deformation capacity. Deformations beyond this limit are not permitted because gravity load can no longer be sustained. In some cases, initial failure at C will result in loss of gravity load resistance, in which case E is a point having deformation equal to that at C and zero resistance. In other cases, deformations beyond C will be permitted even though the lateral load resistance is greatly reduced or even zero-valued.*

  Many currently available computer programs can only directly model a simple bilinear load-deformation relation. For this reason, it is acceptable to represent the load-deformation relation by lines connecting points A-B-C, provided response is not permitted to extend beyond C.

  Alternatively, it may be possible and desirable to use more-detailed load-deformation relations such as that illustrated in Figure 9-7. This relation has the following features:

- *The initial slope of the lateral load-displacement relation represents behavior before cracking. In using such a relation, the engineer should bear in mind that some initial cracking may have occurred because of restrained shrinkage and unknown loads.*
- *Lateral load resistance in some components may degrade in a more controlled manner than shown in Figure 9-6. The rate of degradation with increasing lateral deformation depends on the action being modeled and the number of loading cycles. In general, this aspect of behavior should be established on the basis of test data.*

Components subject to degrading during cyclic loading, such as shear-controlled beams, columns, and piers, show a loss of resistance between points B and C. (Refer to the discussion of Figure 9-11 in Section 9.5.4.1.) Degrading behavior must be modeled.

Commentary: Degradation is different from ductile yielding. Components that yield are able to maintain their strength over several cycles but cannot accept additional load. Degrading components do not maintain their strength; some of the load carried by a degrading component on one cycle must be carried by different components on subsequent cycles. Overall building response could be altered if enough elements or components experience this effective loss of strength. Consequently, widespread degradation must be modeled.

Degrading components are generally force-controlled, although some components may be able *to degrade slightly and still withstand additional deformations. In force-controlled primary components, inelasticity is not allowed, so deformation beyond point B in Figure 9-6 is unacceptable. This generally marks the end of reliable behavior and, consequently, the end of meaningful analysis. However, if the component can be designated as secondary (see Chapter 11), analysis may proceed, as long as the degrading behavior is accounted for, i.e., as long as the component's forces are redistributed to other elements.*

In a nonlinear static analysis such as a pushover, degrading is represented by a "shedding" of load from critical components without the application of additional load. If available computer programs are unable to represent this effect directly, degradation can be simulated with a series of analyses. In the first analysis, forces are applied until critical components reach a deformation level at which they would be expected to degrade. For the second analysis, the critical components are assigned fully or partially degraded strength and stiffness less than their original properties, and this second model is loaded to a point at which additional degradation is expected. A third model with degraded properties is analyzed, etc. The series of models and analyses yields a series of capacity curves; each curve is applicable over a range of displacements. Taken together, the applicable parts of each curve form an effective capacity curve for the degrading structure (Example building studies in Volume 2 illustrate this approach). Alternatively, in accordance with Section 9.3.1, if a component is expected to be fully degraded long before the performance point is reached, it may be acceptable to exclude it from the model altogether.

The point at which degrading occurs is not entirely predictable; it may depend on detailing, member proportions, and the relative magnitudes of moment, shear, and axial force. With reference to Figure 9-6, Figure 9-11, and Table 9-4, a rough approach is to assume that the component will be fully degraded at a ductility demand of 4. Fully degraded properties can be represented by lateral load resistance equal to about 20% of the undegraded strength. An intermediate partially degraded state, assigned at a ductility demand of 2, may also be assumed. Conclusions based on these analyses should be appropriate to the rough nature of analysis assumptions.

Widespread degradation can affect building performance and must be modeled. Example building studies in Volume 2 illustrate acceptable modeling procedures.

9.5.2 Material Models

9.5.2.1 General

The material models should consider all available information, including building plans, original calculations and design criteria, site observations, testing, and records of typical materials and construction practices prevalent at the time of construction. Chapter 5 describes procedures for identifying material properties. Default assumptions may be required in certain cases where information is unavailable.

Commentary: Successful application of the methodology requires good information about the building. In general, material properties should be established by inspection and testing.

9.5.2.2 Concrete

Evaluation of concrete material properties should involve determination of compressive strength, modulus of elasticity, aggregate density, and variability.

Commentary: *Compressive strength may be gauged by using the destructive and nondestructive methods identified in Chapter 5. Alternatively, concrete strength may be projected from early-age values by using conventional relations (ACI 1986). Note that some data indicate that the strength increase for air-cured concrete (as opposed to outdoor exposure) may cease at the age of about one year (Wood 1991). The projection of concrete strength should be accepted only where concrete design strength is known and quality control is believed to have been good, or where data are available on concrete strength at the time of construction. In addition, a visual inspection of the structure should verify that concrete quality appears reasonably uniform and that deterioration has not occurred. The modulus of elasticity may be gauged from nondestructive or destructive tests. Alternatively, it may be gauged indirectly from compression strength and density information by using conventional relations from ACI 318.*

Transverse reinforcement may be taken to enhance the strain capacity and compressive strength of concrete. Except where more-detailed models are used, compressive strain capacity may be defined by Equation 9-1 and compressive strength may be defined by Equation 9-2.

$$\varepsilon_{cu} = 0.005 + 0.1\rho'' f_y / f_c' \leq 0.02 \qquad (9\text{-}1)$$

$$f_{cc} = \left(1 + \rho'' f_y / f_c'\right) f_c' \qquad (9\text{-}2)$$

Commentary: *The strength and deformation capacities of confined concrete depend on aggregate density, the configuration and spacing of transverse reinforcement, and the strength of the concrete and reinforcement. The proposed relations are intended to apply to concrete confined by reasonably well detailed rectilinear hoops. Details of confinement models can be found in the technical literature (Sheikh 1982).*

Where a cross section relies on longitudinal reinforcement for strength, the compression strain in concrete surrounding the bar should not be taken to exceed the buckling strain. Except where more-refined models considering realistic strain histories are used to define buckling strain capacity, the maximum compressive strain of confined concrete, ε_{cu}, should be defined as follows: $\varepsilon_{cu} = 0.02$ for $s/d_b \leq 8$, where s = longitudinal spacing of confining transverse reinforcement and d_b = diameter of longitudinal reinforcement; $\varepsilon_{cu} = 0.005$ for $s/d_b \geq 16$; and ε_{cu} interpolated linearly between these values.

Commentary: *Many structural components, such as beams, columns, and walls, rely on longitudinal reinforcement to resist flexural and axial loads. If the longitudinal reinforcement is stressed in compression, it may buckle, in which case it cannot be relied on to continue resisting compressive forces. Under the action of reversing loads, reinforcement that buckles in compression with loading in one direction may be stressed in tension with loading in the opposite direction. This action may lead to low-cycle fatigue failure, so that the reinforcement can not continue to resist tensile forces. For this reason, it is necessary to ensure that this reinforcement does not buckle. The maximum strain limit of 0.02 is based on an evaluation of buckling data from tests on columns with closely-spaced hoops. The other limit is based on judgment.*

Unless specific data are available to indicate otherwise, deformabilities of components made with lightweight aggregate concrete should be assumed to be about 25% lower than those of equivalent components made with normal-weight aggregate concrete.

Commentary: *Lightweight aggregates in concrete often tend to result in poorer seismic behavior. Compared with normal-weight aggregate concrete of the same compressive strength, elastic modulus is reduced, behavior beyond the peak compressive stress may be more brittle, transverse reinforcement may be less effective as a confining*

Table 9-2. Minimum Tensile Properties of Concrete Reinforcing Bars (ATC 1996a)

		Plain Bars			Deformed Bars		
		Structural Steel Grade	**Intermediate Grade[1]**	**Hard Grade**	**Structural Steel Grade**	**Intermediate Grade[1]**	**Hard Grade**
1900 – 1919	Tensile strength, ksi				Proprietary shapes		
	Yield strength, ksi	33 to 35			Proprietary shapes, 33 to 55[2]		
1920 – 1949	Tensile strength, ksi	55 to 70	70 to 85	80 min	55 to 70	70 to 85	80 min
	Yield strength, ksi	33 ASTM A15-14	40 ASTM A15-14	50 ASTM A15-14	33 ASTM A15-14	40 ASTM A15-14	50 ASTM A15-14
1950 – 1969	Tensile strength, ksi				70 ASTM A-15	118 ASTM A-432	ASTM A-31
	Yield strength, ksi				40 to 45	60	75
1970 – 1996	Tensile strength, ksi				ASTM A-15[3]	ASTM 432	
	Yield strength, ksi				40 to 45	60	

[1] Intermediate-grade reinforcement established as the single standard for billet-steel in 1928 (approx.).

[2] Bend test determined that these early high-strength bars were often brittle.

[3] This grade has been generally phased out for use as primary tensile reinforcement but is often used for stirrups and ties.

agent, bond/anchorage/shear strengths are reduced, and overall reversed cyclic load behavior tends to be more pinched and to degrade more rapidly.

9.5.2.3 Reinforcement

Evaluation of reinforcement should consider grade; surface deformations; surface conditions (including corrosion); and bar placement and detailing. Grade can be established from the construction plans, from examination of grade markings in exposed bars, from sample tests, or from information on reinforcement commonly used during the construction era. Bar locations and details may be established from the plans or from testing and inspection of the building. See Chapter 5 for additional details.

Commentary: In some cases it will not be possible to establish the reinforcement grade with certainty. In these cases it may be necessary to rely on information about construction time and the reinforcement commonly available at that time. The engineer should be aware that the actual materials may differ from assumed ones, and the evaluation should take into account the possible errors in estimating both capacities and demands. In the absence of more-definitive data, the data in Table 9-2 may be used to guide selection of reinforcement properties. The table presents information on typical reinforcement grades used at various times in California (ATC 1996a).

Where theoretical or empirical models are used to calculate component strength and deformability, the potential for reinforcement buckling and subsequent fracture should be taken into account.

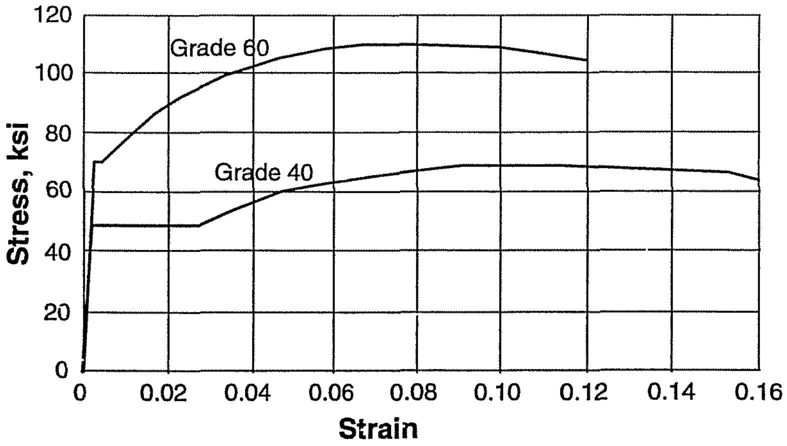


Figure 9-8. Reinforcing Steel Stress-Strain Relations

Maximum tensile strain in reinforcing steel should not be assumed to exceed 0.05.

Commentary: *Figure 9-8 plots typical stress-strain relations for reinforcement that may be used to help define default relations in cases where data are not available. These relations may be useful for calculating the strength and deformation properties of components.*

Reinforced concrete structural members subjected to deformation reversals may sustain reinforcement buckling, which usually is followed by the fracture of reinforcement in tension when the deformation is reversed. The tendency for buckling depends not only on the compressive strains but on previously developed plastic tensile strains in reinforcement, because plastic tension strain leaves an elongated bar exposed to develop compression in the initial stages of load reversal.

9.5.3 Component Initial Stiffness

Reinforced concrete component initial stiffness may be represented by a secant value defined by the effective yield point of the component, as shown by the initial slope in Figure 9-6. For flexure-dominated components, this stiffness corresponds approximately to the fully-cracked stiffness. For shear-dominated components, this stiffness corresponds approximately to the uncracked stiffness. The stiffness value may be determined as a function of material properties (considering current condition), component dimensions, reinforcement quantities, boundary conditions, and stress and deformation levels.

In many cases it will be impractical to calculate effective stiffnesses directly from basic mechanics principles. Instead, the effective initial stiffness may be based on the approximate values of Table 9-3.

As discussed in Section 9.4.2.2, slab-column frames can be modeled using the effective beam width model, the equivalent frame model, or finite-element plate-bending models. When these models are used, the effective stiffnesses of components should be established on the basis of experimental evidence to represent effective stiffnesses according to the general principles of this section. In particular, the effects of cracking on stiffness should be taken into account considering experimental evidence.

Commentary: *Reinforced concrete texts and design codes prescribe precise procedures for stiffness calculation. Most of these procedures were developed from tests of simply supported reinforced concrete flexural members loaded to relatively low stress levels.*

Table 9-3. Component Initial Stiffnesses

Component	*Flexural Rigidity*	*Shear Rigidity*[2]	*Axial Rigidity*
Beam, non-prestressed[1]	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Beam, prestressed[1]	E_cI_g	$0.4E_cA_w$	E_cA_g
Columns in compression	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns in tension	$0.5E_cI_g$	$0.4E_cA_w$	E_sA_s
Walls, uncracked	$0.8E_cI_g$	$0.4E_cA_w$	E_cA_g
Walls, cracked	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Flat slabs, non-prestressed	*See discussion*	$0.4E_cA_w$	E_cA_g
Flat slabs, prestressed	*in Section 9.5.3*	$0.4E_cA_w$	E_cA_g

[1] I_g for T-beams may be taken twice the I_g of the web alone, or may be based on the effective section as defined in Section 9.5.4.2.

[2] For shear stiffness, the quantity $0.4E_c$ has been used to represent the shear modulus, G.

[3] For shear-dominated components, see the discussion and commentary in Section 9.5.3.

The results often are not transferable to the effective stiffness of a reinforced concrete component that is interconnected with other components and subjected to high levels of lateral load. Actual boundary conditions and stress levels may result in significantly different effective stiffnesses.

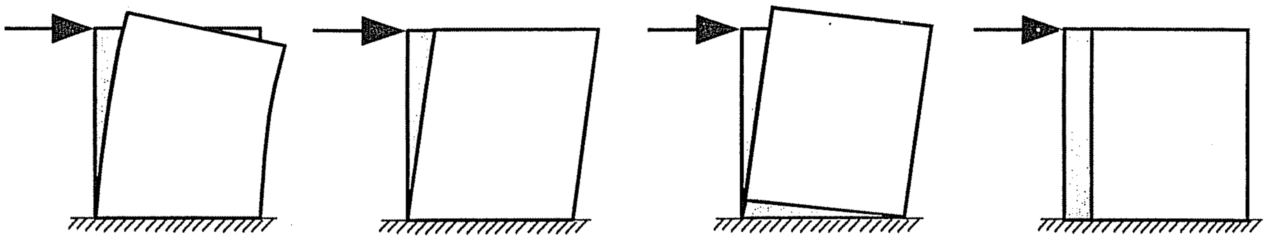
Experience in component testing suggests that important variations in effective stiffness can occur for nominally similar conditions (Aschheim and Moehle 1992; Otani et al. 1994). The engineer evaluating an existing building must be aware that a range of stiffnesses is possible for any set of nominal conditions and that variations within the range may affect the final performance assessment.

Figure 9-9 illustrates the typical sources of flexibility for a component subjected to lateral forces. These include flexure, shear, and partial reinforcement slip from adjacent connections (foundations, beam-column joints, walls, etc.). Flexure tends to dominate for relatively slender components (l/h exceeding about 5, where h is the section depth parallel the lateral load and l is the length from the point of maximum moment to the inflection point).

Shear and partial reinforcement slip tend to dominate for lower aspect ratios. For columns and shear walls subjected to appreciable axial stress variations under earthquake loading, it is important to model axial flexibility also.

The recommended initial stiffness, corresponding to stiffness near yield, in many cases will be considerably less than the gross-section stiffness commonly used in conventional design practice. The effective stiffness for a given component will depend somewhat on the sources of deformation and the anticipated stress levels, as suggested in the following paragraphs.

For a flexure-dominated component, effective stiffness can be calculated by considering well-developed flexural cracking, minimal shear



Flexure Shear Reinforcement Slip Sliding

Figure 9-9. Typical Idealized Sources of Deformation in a Component Subjected to Lateral Forces

cracking, and partial reinforcement slip from adjacent joints and foundation elements. Flexural stiffness can be calculated according to conventional procedures that take into consideration the variation of flexural moment and cracking along the component length. Shear stiffness may be approximated on the basis of the gross section. Where stress levels under applicable load combinations are certain to be less than levels corresponding to significant cracking, uncracked flexural stiffness may be appropriate. Note that flexural theory commonly assumes that concrete in the tension zone carries no tension stress. In reality, cracking in reinforced concrete components occurs at discrete locations, and significant tension stiffening can result from tension carried by concrete between the cracks (Park and Paulay 1974). Especially for lightly-reinforced components, the fully-cracked stiffness may grossly underestimate the actual stiffness.

For a shear-dominated component, the onset of shear cracking commonly results in a dramatic reduction in effective stiffness and may be considered to represent the end of elastic behavior for the component (Sozen and Moehle 1993). Therefore, for shear-dominated components the effective initial stiffness may be based on the gross-section properties considering flexure and shear. Stiffness reduction to account for reinforcement slip from foundation elements may be appropriate.

For an axial load-dominated component, the appropriate stiffness depends on whether the axial load is tensile or compressive under the applicable load combinations. Where it is compressive, the stiffness can be derived from the gross-section or uncracked transformed-section properties. Where it is tensile, and of sufficient magnitude to result in cracking, the stiffness should be based on the reinforcement only, although some adjustment to account for tension stiffening may be appropriate. However, note that tension stiffening tends to degrade under repeated loading.

The stiffness values given in Table 9-3 may be used instead of values calculated directly from principles of mechanics. The values were selected to represent values expected for typical proportions and reinforcement ratios. Some adjustment up or down depending on the actual proportions and reinforcement ratios is acceptable.

Default stiffness values in Table 9-3 are estimates only. Actual stiffness depends on stress level, deformation type, degree of fixity, crack patterns, etc.

Some of the stiffness values given in Table 9-3 depend on the level of axial load, where axial load is calculated considering gravity and lateral load effects. In statically indeterminate structures the calculated internal forces will depend on the assumed stiffness, and in certain cases it will not be possible to identify a stiffness from Table 9-3 that results in a force that is consistent with the assumed stiffness.

For example, a column may be assumed to be in compression, resulting in a flexural stiffness of $0.7E_cI_g$*, but the analysis with this stiffness produces column tension. On the other hand, if the same column is assumed to be in tension, resulting in a flexural stiffness of* $0.5E_cI_g$*, the analysis indicates that the same column is in compression. For this column, it is acceptable to assume an intermediate stiffness of* $0.6E_cI_g$ *and move on with the analysis, rather than trying to iterate an exact solution.*

Various approaches to representing the effects of cracking on stiffness of reinforced concrete slabs have been proposed and verified. Vanderbilt and Corley (1983) recommend modeling a slab-column frame with an equivalent frame in which the slab flexural stiffness is modeled as one-third of the gross-section value. Hwang and Moehle (1993) recommend an effective beam width model having an effective width for interior framing lines equal to $\beta(5c_1 + 0.25l_1)$*, where* β *represents cracking effects and ranges typically from one-third to one-half,* c_1 *= column dimension in the direction of framing, and* l_1 *= center-to-center span in the direction of framing.*

For exterior frame lines, use half this width. Use gross-section flexural stiffness properties for the effective width. Note that this effective width applies only where the analysis model represents the slab-column joints as having zero horizontal dimension (the effective width automatically corrects for the fact that the joint is nearly rigid). Alternate approaches may be used where verified by tests.

For prestressed slabs, less cracking is likely, so it is acceptable to model the framing using the equivalent frame model without the one-third factor or the effective beam width model with $\beta = 1.0$*.*

9.5.4 Component Strength

9.5.4.1 General

Actions (forces and associated deformations) in a structure are classified as either deformation-controlled or force-controlled. Components are similarly classified for each action they experience (e.g. columns in flexure, columns in shear). Thus, all components are classified as either primary or secondary (see Chapter 11) and as deformation- or force-controlled. Unless noted otherwise, the following discussion refers to primary elements and components only.

Deformation-controlled actions are permitted to exceed elastic limits under applicable earthquake loads. Strengths for deformation-controlled actions should be taken equal to expected strengths obtained experimentally or calculated by using accepted mechanics principles. Expected strength is defined as the mean maximum resistance expected over the range of deformations to which the component is likely to be subjected. When calculations are used to define mean expected strength, expected materials strengths including strain hardening are to be taken into account. The tensile stress in yielding longitudinal reinforcement should be assumed to be at least 1.25 times the nominal yield strength. Procedures specified in ACI 318 may be used to calculate strengths, except that strength reduction factors, ϕ, should be taken equal to 1.0, and other procedures specified in this document should govern where applicable.

Force-controlled actions are not permitted to exceed elastic limits under applicable earthquake loads. Strengths for force-controlled components should be taken equal to lower bound strengths obtained experimentally or calculated by using established mechanics principles. Lower-bound strength is defined generally as the lower 5 percentile of strengths expected. Where the strength degrades with continued cycling or increased lateral deformations, the lower-bound strength is defined as the expected minimum value within the range of deformations and loading cycles to which the component is likely to be subjected. When calculations are used to define lower-bound strengths, lower bound estimates of materials properties are to be assumed. Procedures specified in ACI 318 (with $\phi = 1.0$) may be used to calculate nominal strengths, except that other procedures recommended in this document should be used where appropriate.

For the structures covered by this methodology, deformation-controlled actions are limited to the following:

- Flexure (in beams, slabs, columns, and walls),
- Shear distortion in walls and wall segments,
- Connection rotation at slab-column connections

***Commentary:** This methodology is a displacement-based procedure, that is, its basis lies in estimating the expected lateral displacements and the resulting local deformations and internal force demands. For ductile components subject to deformation-controlled actions, performance is measured by the relation of deformation demand to deformation capacity. Force and stress levels are of lesser importance for these components. By contrast, for components subject to force-controlled actions, relatively brittle behavior is expected and the main measure of performance is the force or stress level. The force or stress levels in these components depend primarily on the forces that are delivered to them by yielding deformation-controlled components. Capacities of force-controlled components sometimes depend on ductility demand, as noted below.*

> For each action (e.g., flexure, shear), a component is either deformation-controlled or force-controlled. Inelasticity in force-controlled primary components is not acceptable. Strength should be modeled differently for deformation- and force-controlled components.

High strength estimates are desirable for deformation-controlled actions because in a yielding structure these determine the demands on the force-controlled actions. By contrast, low estimates are desirable for force-controlled actions because these actions may result in brittle failure, and a goal of the evaluation and retrofit design is to avoid this type of failure. There is one exception: where the same materials influence the strength of both deformation-controlled and force-controlled actions, it is reasonable to assume the same material properties rather than assuming upper-bound values for one and lower-bound values for the other. For example, consider a reinforced concrete beam where flexure is the deformation-controlled action and shear is the force-controlled action. In this case, both flexural and shear strength are affected by concrete and reinforcement properties. It would be reasonable to calculate flexural strength assuming estimated concrete strength and reinforcement stress equal to 1.25 times the nominal value. Shear strength would be calculated using the same assumed concrete strength and the same assumed nominal yield stress for the reinforcement, but without strain hardening. It would be unreasonable to assume a high compressive strength for flexure and a low compressive strength for shear because the same concrete resists both actions.

Deformation-controlled actions in reinforced concrete construction typically are limited to flexure and to shear in components with low aspect ratios. Flexure generally is the more ductile of the two, and resistance in flexure usually can be determined with greater accuracy. For this reason, *deformation-controlled actions are largely limited to flexure.*

As a flexure-dominated component is flexed into the inelastic range, the longitudinal reinforcement in tension may be stressed to yield and beyond. The actual yield stress of reinforcing steel typically ranges from the nominal yield value to about 1.3 times the nominal value, with average values about 1.15 times the nominal value (Mirza and MacGregor 1979). Tensile strength, which may be approached in components having high ductility demand, is typically about 1.5 times the actual yield value. Therefore, the minimum recommended tensile stress of 1.25 times the nominal yield value should be considered a low estimate suitable only for components with low and intermediate ductility demands.

In all cases, strengths should be determined with due consideration for co existing forces. For example, flexural strength and deformation capacity of columns need to be calculated considering the axial forces likely to co exist with the flexural demands. In general, for a column in compression, flexure is deformation-controlled and axial behavior is force-controlled. The column flexural moment strength and corresponding acceptance criteria are determined for the axial load expected to be acting on the column for the appropriate load combinations. Where lateral loading in different directions results in different axial loads, flexural strength and acceptability should be checked for both extremes and for critical cases in between. Special attention is required for corner columns, which may experience very high axial tension or compression under lateral loading along a diagonal building axis.

Component ductility demand is classified into three levels, as listed in Table 9-4. Some procedures for strength calculation, in particular for force-controlled actions, define the strength as a function of the component ductility demand, as defined in this table.

Table 9-4. Component Ductility Demand Classification

Maximum Value of Displacement Ductility	Classification
< 2	Low ductility demand
2 to 4	Moderate ductility demand
> 4	High ductility demand

Where strength and deformability capacities are to be derived from test data, the tests should be representative of the proportions, details, and expected stress levels of the components. In establishing design values from tests, the expected variability in test results must be taken into account. The loading history used in the test should be representative of cyclic response and damage accumulation expected for the critical loading.

Test data are used to define idealized multi linear load-deformation relations according to the following procedure:

- For deformation-controlled actions (Figure 9-10), a "backbone curve" should represent an upper bound to the forces and a drop in resistance when strength degradation becomes apparent in the cyclic data. A multi linear "idealized" load-deformation relation similar to that shown in Figure 9-6 approximates the backbone curve. For deformation-controlled actions, the multi linear load-deformation relation should display a ductility capacity of not less than 2. Otherwise, the action should be defined as force-controlled.
- For force-controlled actions (Figure 9-11), the backbone curve represents a *lower* bound to the forces, followed by a drop in resistance to match cyclic data. The idealized load-deformation relation should not show displacement ductility. (See Section 9.5.1.)

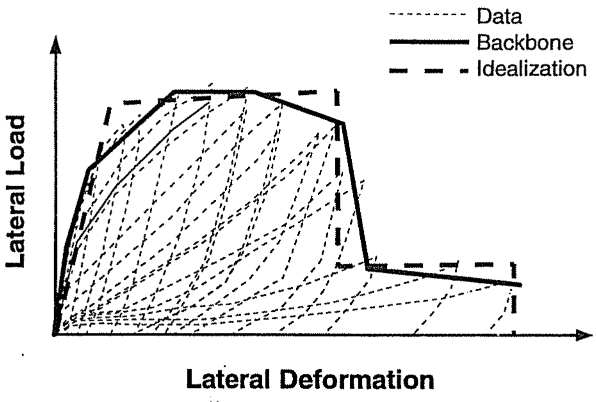


Figure 9-10. Construction of a Backbone Load-Deformation Relation for a Deformation-Controlled Action

Commentary: Strengths and deformation capacities given in this chapter are for earthquake loadings involving about three fully reversed cycles to the specified deformation level, in addition to similar cycles to lesser deformation levels. In some cases, including some short-period buildings and buildings subjected to long-duration earthquakes, a building may be subjected to more-numerous cycles at the specified deformation level. In other cases, such as where near-field ground motion effects are significant, a building may be subjected to a single impulsive deformation cycle. In general, for more numerous deformation cycles, the deformation capacity of deformation-controlled actions and strengths of force-controlled actions will be reduced. This effect should be considered in the evaluation or retrofit design. Where tests are conducted to determine modeling criteria, the loading program in the test can be adjusted to represent the history expected for the controlling loading.

9.5.4.2 Flexure and Axial Loads

It is generally acceptable to calculate flexural strength of members (with or without axial loads) on the basis of assumed monotonic load behavior, with strains assumed to vary linearly across the section and stresses assumed to be uniquely related to strains according to monotonic stress-strain relations. In squat wall sections, the effects of nonlinear strain variations may be included.

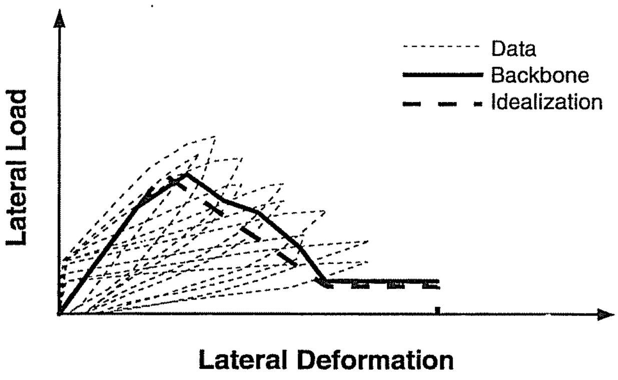


Figure 9-11. Construction of a Backbone Load-Deformation Relation for a Force-Controlled Action

In members consisting of a flange and web that act integrally, the combined stiffness and strength for flexural and axial loading should be calculated considering a width of effective flange on each side of the web equal to the smallest of (a) the provided flange width, (b) half the distance to the next web, and (c) one-eighth of the span for beams or one-quarter of the total height for walls.

When the flange is in compression, both the concrete and reinforcement within the effective width should be considered effective in resisting flexure and axial load. When the flange is in tension, longitudinal reinforcement within the effective width should be considered fully effective for resisting flexure and axial loads. The portion of the flange extending beyond the width of the web should be assumed ineffective in resisting shear within the plane of the web.

Flexural strength may be calculated directly by considering the stress-strain relations of Section 9.5.2 and conditions of equilibrium and compatibility. Alternatively, flexural strength may be calculated by using the rectangular stress block of ACI 318, with the following conditions:

- Maximum concrete compression strain ranging from 0.003 to 0.005

- Maximum stress in concrete compression zone taken equal to 85% of the expected compression strength
- Elasto plastic behavior of reinforcement with assumed yield stress equal to 1.4 times the nominal value

Shear strength degrades as a function of ductility demand. The degradation can change a component from deformation-controlled to force-controlled. This effect can be modeled explicitly by changing component strength as pushover analysis proceeds.

Flexural strength should be calculated considering the stress limitations imposed by available development and splicing of reinforcement, as described in Section 9.5.4.5.

Commentary: *Current U.S. codes of practice for new construction require that strength be calculated for a maximum usable concrete compression strain of 0.003. Tests of components subjected to flexure or combined flexure and axial load indicate that larger strains usually can be reached when the component has moderate strain gradients and confinement provided by adjacent sections or transverse reinforcement. For this reason, it is permitted to calculate flexural and combined flexural and axial load strengths by assuming maximum usable strain of 0.005 at the extreme concrete compression fiber. Larger strains may be used where allowed by the confining action of transverse reinforcement. In no case should compression strain capacity be assumed to exceed 0.02. For unconfined components subjected to nearly uniaxial compression (that is, without significant bending), the maximum usable compression strain at strength may be as low as 0.002, rather than the limit of 0.005 recommended where significant flexure is present.*

The recommendation for effective flange width is based on data from lateral load tests of frame connections and flanged walls (French and Moehle 1991; Thomsen and Wallace 1995). The recommendation applies to beams, walls, and other similar components. When the flange is in compression, it is acceptable for computational convenience to ignore the flange reinforcement. When the flange is in tension, the reinforcement within the effective width should be assumed to be fully effective in carrying flexural tension forces. For walls with significant boundary reinforcement located just outside the designated effective width, that reinforcement should be considered at least partly effective in resisting flexure and axial loads.

Flexural strength and deformation capacity of columns must be calculated with due consideration of co existing axial force. This subject is discussed in greater detail in the commentary to Section 9.5.4.1.

9.5.4.3 Shear and Torsion

Strengths in shear and torsion should be calculated according to ACI 318 except as noted below.

Commentary: *Shear and torsion resistance is known to degrade with increasing number of loading cycles and increasing ductility demands. This effect must be recognized in establishing shear strengths for evaluation or retrofit design. The expressions presented here are intended for structures in which only a few deformation cycles at the peak displacement level occur during the applicable earthquake. Where long-duration motions are expected, or where the structure has a very short period resulting in more-numerous cycles, some downward adjustment in shear strength may be appropriate.*

Within yielding regions of components with moderate or high ductility demands, shear and torsion strength should be calculated according to accepted procedures for ductile components (e.g., the provisions of Chapter 21 of ACI 318). Within yielding regions of components with low ductility demands, and outside yielding regions, shear strength may be calculated using accepted procedures normally used for elastic response (e.g., the provisions of Chapter 11 of ACI 318).

Within yielding regions of components with moderate or high ductility demands, transverse reinforcement should be assumed ineffective in resisting shear or torsion where the longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, or where perimeter hoops either are lap-spliced or have hooks that are not adequately anchored in the concrete core. Within yielding regions of components with low ductility demands, and outside yielding regions, transverse reinforcement should be assumed ineffective in resisting shear or torsion where the longitudinal spacing of transverse reinforcement exceeds the component effective depth measured in the direction of shear.

Commentary: *The recommendations for transverse reinforcement spacing are based on the understanding that unless there is a minimum amount of transverse reinforcement, spaced so that at least one well-anchored stirrup or hoop intersects each inclined shear crack, the shear contribution of transverse reinforcement should be ignored. The requirements for details in yielding regions of components with moderate and high ductility demands are based on the understanding that cover spalling may occur in these regions, and therefore the transverse reinforcement must be adequately anchored into the core concrete. For the purposes of this section, the length of the yielding region can be assumed to be equal to the largest of the following: the member dimension in the direction of the loading, one-sixth of the clear span or clear height, and 18 inches.*

Column shear strength in existing construction may be calculated by the following expressions:

$$V_n = V_c + V_s \tag{9-3}$$

where

$$V_c = 3.5\lambda\left(k + \frac{N}{2000A_g}\right)\sqrt{f'_c}\,b_w d \tag{9-4}$$

$$V_s = \frac{A_v f_y d}{0.6s} \tag{9-5}$$

and $k = 1$ in regions of low ductility and 0 in regions of moderate and high ductility, $\lambda = 0.75$ for lightweight aggregate concrete and 1 for normal-weight aggregate concrete, and N = axial compression force in pounds (zero for tension force). All units are expressed in pounds and inches. Note that column shear strength needs to be checked within yielding regions at the column ends and near midheight. There may be less reinforcement at midheight than at the ends, and ductility demands will be lower.

Commentary: *Experiments on columns subjected to axial load and reversed cyclic lateral displacements indicate that ACI 318 design strength equations may be excessively conservative for older existing columns, especially those with low ductility demands (Lynn et al. 1995; Aschheim and Moehle 1992; Priestley et al. 1994). The recommended column shear strength equation is based on a review of the available test data. The available strength in older columns appears to be related to ductility demand; therefore, conservative procedures should be used to determine whether ductility demands will reach critical levels.*

The column axial load should include both gravity and seismic contributions.

Where ductility levels reach intermediate or high levels, the methodology recommends that the shear strength contribution assigned to the concrete be reduced dramatically. For columns with low amounts of transverse reinforcement, the assigned strength drops to near zero. This severe recommendation is made with the understanding *that shear failure of poorly confined columns commonly is a cause of column failure and subsequent structure collapse. Engineering judgment—as well as the specifications of this methodology—should be applied to determine the proper course of action for buildings with columns having widely spaced ties and moderately high shear stresses.*

Wall shear strength in existing construction may be calculated by the following procedures.

$$V_n = V_c + V_s \qquad (9\text{-}6)$$

where

$$V_c = 2\lambda\sqrt{f_c'}\,t_w l_w \qquad (9\text{-}7)$$

$$V_s = \frac{A_v \beta_n f_y l_w}{s} \qquad (9\text{-}8)$$

except the shear strength V_n need not be taken to be less than $4\lambda\sqrt{f_c'}\,t_w l_w$ and should not be taken to exceed $10\lambda\sqrt{f_c'}\,t_w l_w$, where V_n is the nominal wall shear strength, $\lambda = 1.0$ for normal-weight aggregate and $\lambda = 0.75$ for lightweight aggregate concrete, t_w is wall web thickness (in inches), and l_w is wall length (in inches).

When a wall has a horizontal reinforcement percentage, ρ_n, less than 0.0025, the reinforcement contribution is reduced with the β_n factor. The value of β_n should decrease linearly from a value of 1.0 for $\rho_n = 0.0025$ to a value of 0.0 for $\rho_n = 0.0015$. For $\rho_n \leq 0.0015$, there will be no contribution from the wall reinforcement to the shear strength of the wall.

In walls with flanges and webs acting integrally, the portion of the flange extending beyond the width of the web should be assumed ineffective in resisting shear within the plane of the web.

***Commentary:** The lower bound for walls is supported by tests on walls (Sozen and Moehle 1993; Wood 1990). The addition of the variable λ to the expressions is based on judgment. For shear walls, in contrast to columns, the available data do not support reducing the concrete contribution with increasing ductility demand (ATC 1983). The reduction in shear strength for low horizontal reinforcement ratios is based on the expectation that wall reinforcement may fracture and be ineffective once shear cracking occurs if the volume ratio of reinforcement is very low.*

9.5.4.4 Shear Friction

Shear friction strength should be calculated according to ACI 318, taking into consideration the expected axial load due to gravity and earthquake effects. For shear walls, the axial load may be assumed to act uniformly across the length of the element at the level being checked. Where retrofitting involves the addition of concrete requiring overhead work with dry-pack, the shear friction coefficient, μ, should be taken to be equal to 70% of the value specified by ACI 318.

***Commentary:** The recommendation for shear friction strength is based on research results reported by Bass et al. (1989). The reduced friction coefficient for overhead work is because of the likelihood of having poorer quality of the interface at this joint.*

For reinforced concrete walls subjected to significant lateral forces, it is likely that portions of the wall length will be in tension because of flexure and axial load effects. For isolated cantilever walls, the compression zone may be relatively short, with the majority of the wall length subjected to flexural tension. For coupled walls, one of the wall piers may be in tension along its entire length because of coupling action. In the opinion of the writers, it is acceptable in cases such as these to assume that the entire length of the wall is effective in resisting shear along critical planes, with the axial load stress taken equal to the total axial load divided by the total wall length.

Shear friction provides most or all of the shear resistance across typical horizontal construction joints. Section 9.4.3.1 discusses the need to model construction joints.

9.5.4.5 Development, Splicing, and Anchorage

Development requirements for straight bars, hooked bars, lap-splices, and embedments may be calculated according to the general provisions of ACI 318. Where the ACI 318 provisions are not satisfied, it will be necessary to estimate the maximum stress that can be developed in the bar under stress reversals. The following general approaches may be used.

Within yielding regions of components with moderate or high ductility demands, detail requirements and strength provisions for straight, hooked, and lap-spliced bars should be according to Chapter 21 of ACI 318. Within yielding regions of components with low ductility demands, and outside yielding regions, details and strength may be calculated according to Chapter 12 of ACI 318, except that the detail requirements and strength of lap-splices may be taken to be equal to those for development of straight bars in tension without consideration of lap-splice classifications.

Where the development, hook, and lap-splice length and detailing requirements of ACI 318 are not satisfied in existing construction, the maximum stress capacity of reinforcement may be calculated according to Equation 9-9.

$$f_s = \frac{l_b}{l_d} f_y \qquad (9\text{-}9)$$

where f_s = bar stress capacity for the development, hook, or lap-splice length (l_b) provided; l_d = length required by Chapter 12 or Chapter 21 (as appropriate) of ACI 318 for development, hook, or lap-splice length, except that splices may be assumed to be equivalent to straight bar development in tension; and f_y = yield strength of reinforcement. Where transverse reinforcement is distributed along the development length with spacing not exceeding one-third of the effective depth, the developed reinforcement may be assumed to retain the calculated stress capacity to large ductility levels. For larger spacings of transverse reinforcement, the developed stress should be assumed to degrade from f_s to $0.2f_s$ at ductility demand equal to 2.0.

Strength of straight, discontinuous bars embedded in concrete sections (including beam-column joints) with clear cover over the embedded bar not less than $3d_b$ may be calculated by the following equation:

$$f_s = \frac{2500}{d_b} l_e \le f_y \qquad (9\text{-}10)$$

where f_s = maximum stress (in psi) that can be developed in embedded bar having embedment length l_e (in inches), d_b = diameter of embedded bar (in inches), and f_y = bar yield stress (in psi). When the expected stress equals or exceeds f_s as calculated above, and f_s is less than f_y, the developed stress should be assumed to degrade from f_s to $0.2f_s$ at ductility demand equal to 2.0. In beams with short bottom bar embedments into beam-column joints, flexural strength should be calculated considering the stress limitations of Equation 9-10.

Doweled bars added in seismic retrofit may be assumed to develop yield stress when all the following conditions are satisfied: drilled holes for dowel bars are cleaned with a stiff brush that extends the length of the hole; embedment length, l_e, is not less than $10d_b$; and minimum spacing of dowel bars is not less than $4l_e$ and minimum edge distance is not less than $2l_e$. Other values for dowel bars should be verified by test data. Field samples should be tested to ensure that design strengths are developed.

***Commentary:** It is well accepted in the technical literature that strength for a given length of a lap-splice is essentially equivalent to that for the same length of straight bar development. Current codes for new construction require longer lengths for lap-splices than for straight bar development, in part to discourage the use of laps in regions of high stress. Where existing construction is retrofitted, lap-splices similarly should be located away from regions of high stress and moderate or high ductility demand.*

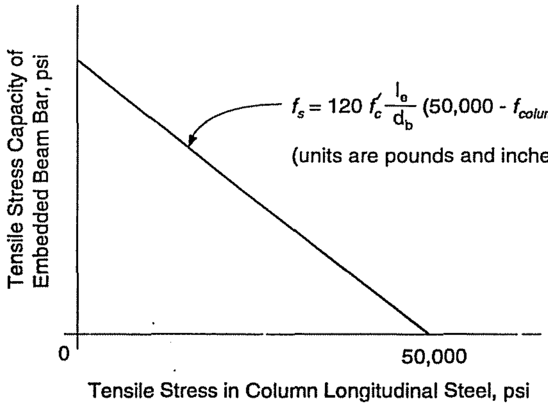


Figure 9-12. Relation Between Beam Embedded Bar Stress Capacity and Co-Existing Tensile Stress in Adjacent Column Longitudinal Reinforcement

The embedment length used in Equation 9-10 was derived from design equations in ACI 318 that relate to pullout of bars having sufficient cover or transverse reinforcement so that splitting of cover concrete cannot occur. The expressions may be applied to bottom beam reinforcement embedded a short distance into a beam-column joint. For an embedment of six inches into a joint, which is common for frames designed for gravity loads only, Equation 9-10 typically produces values of f_s *= 20 ksi or lower. Experimental research on beam-column connections (CSSC 1994b) indicates that higher stress capacities may be available when flexural tension stresses in adjacent column longitudinal reinforcement (which acts to clamp the embedded bar) are low. The available data support the use of Figure 9-12 to estimate the stress capacity of embedded bars. In this figure, the column longitudinal reinforcement stress is calculated on the basis of column actions coexisting with the embedded bar tensile force.*

The specification for doweled bars is based on tests (Luke et al. 1985). Other suitable methods of anchoring new concrete to existing concrete are acceptable.

9.5.4.6 Beam-Column Connections

Shear strength in beam-column joints can be calculated according to the general procedures of ACI 352 (ACI 1985), with appropriate modifications to reflect the differences in detailing and configuration between older existing construction and new construction. Specific recommendations are provided in the following paragraphs.

A joint is defined as the volume of the column within the depth of the beam framing into the column. For nominal joint shear stress calculations, the effective horizontal joint area, A_j, is defined by a joint depth equal to the column dimension in the direction of framing and a joint width equal to the width of the smallest of the following: column width, beam width plus joint depth, and twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side. Forces are to be calculated on the basis of the development of flexural plastic hinges in adjacent framing members, including effective slab width, but need not exceed values calculated from appropriate gravity and earthquake load combinations. Nominal joint shear strength, V_n, may be calculated as follows:

$$V_n = \lambda \gamma \sqrt{f_c'} A_j, psi \qquad (9\text{-}11)$$

in which $\lambda = 0.75$ for lightweight aggregate concrete and 1.0 for normal-weight aggregate concrete, A_j is effective horizontal joint area, and γ is defined in Table 9-5.

In Table 9-5, ρ'' is the volumetric ratio of horizontal confinement reinforcement in the joint.

Where joint reinforcement is unknown, worst-case reinforcement should be assumed.

Commentary: The specification for beam-column joint shear strength is developed from various sources (CSSC 1994b). Otani (1991) and Kitayama et al. (1991) present data indicating that joint shear strength is relatively insensitive to the amount of joint transverse reinforcement, provided there is a minimum amount (a transverse steel ratio equal to about 0.003). Beres et al. (1992) and Pessiki et al. (1990) present supporting data for joints representative of those used in older frame construction. Although some researchers report that increased column axial load results in

Table 9-5. Values of γ for Use in Equation 9-11

ρ''	Value of γ				
	Interior Joint With Transverse Beams	Interior Joint Without Transverse Beams	Exterior Joint With Transverse Beams	Exterior Joint Without Transverse Beams	Knee Joint
<0.003	12	10	8	6	4
≥0.003	20	15	15	12	8

increased shear strength, the data on the whole do not show a significant trend.

The procedures for estimating joint shear are the same as those specified in ACI 318 and ACI 352.

9.5.4.7 Slab-Column Connection Strength

The shear and moment transfer strength of the slab-column connection should be calculated by considering the combined action of flexure, shear, and torsion in the slab at the connection with the column.

The flexural strength of a slab resisting moment due to lateral deformations should be calculated as $M_{nCS} - M_{gCS}$, where M_{nCS} is the flexural strength of the column strip and M_{gCS} is the column strip moment due to gravity loads. M_{gCS} is calculated according to the procedures of ACI 318 with applicable gravity loads.

An acceptable procedure is to calculate the shear and moment transfer strength as described below.

For interior connections without transverse beams, the shear and moment transfer strength may be taken to be equal to the lesser of two strengths: the strength calculated by considering the eccentricity of shear on a slab critical section due to combined shear and moment, as prescribed in ACI 318; and the moment transfer strength, equal to $\Sigma M_n/\gamma_f$. Here, ΣM_n is the sum of positive and negative flexural strengths of a section of slab between lines that are two and one-half slab or drop-panel thicknesses ($2.5h$) outside opposite faces of the column or capital, γ_f is the portion of moment transferred by flexure per the specifications of ACI 318, and h is the slab thickness.

For moment about an axis parallel to the slab edge at exterior connections without transverse beams, where the shear on the slab critical section due to gravity loads does not exceed $0.75V_c$, or at a corner support does not exceed $0.5V_c$, the moment transfer strength may be taken equal to the flexural strength of a section of slab between lines that are a distance, c_1, outside opposite faces of the column or capital. V_c is the direct punching shear strength defined by ACI 318.

Commentary: *The flexural action of a slab connecting to a column is nonuniform, as illustrated in Figure 9-13. Portions of the slab nearest the column yield first, followed by a gradual spread of yielding as deformations increase. The actual flexural strength developed in the slab will depend on the degree to which lateral spread of yielding can occur.*

The recommendation to limit effective width to the column strip is the same as the design requirement of ACI 318 and represents a lower bound to expected flexural strength. In some cases the full width of the slab will yield. If a greater portion of the slab yields than is assumed, the demand on the slab-column connection and the columns will be increased. Nonductile failure modes can result.

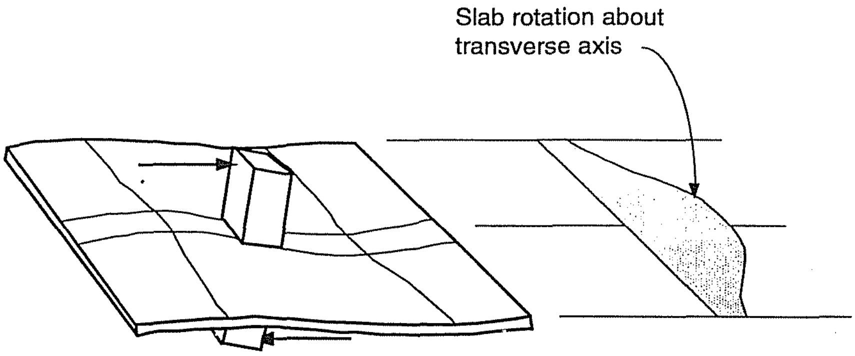


Figure 9-13. Nonuniform Flexural Action of a Slab-Column Connection Under Lateral Deformations

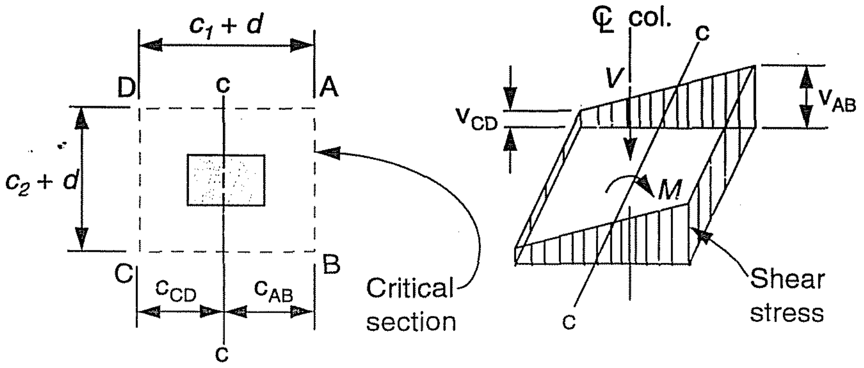


Figure 9-14. Nominal Shear Stresses Acting on a Slab Critical Section (ACI 1995)

Shear and moment transfer strength of interior slab-column connections may be calculated using any models verified by experimental evidence (Hwang and Moehle 1993; Hawkins 1980). A simplified approach that follows the concepts of ACI 318 is acceptable. According to this approach, connection strength is the minimum of two calculated strengths. One is the strength corresponding to development of a nominal shear stress capacity on a slab critical section surrounding the column (Figure 9-14). All definitions are according to ACI 318. In applying this procedure, tests indicate that biaxial moment transfer need not be considered (Pan and Moehle 1992; Martinez et al. 1994). The second strength corresponds to developing flexural capacity of an effective slab width. The effective width is modified from ACI 318 on the basis of results reported by Hwang and Moehle (1993). Both top and bottom reinforcement are included in the calculated strength.

Shear and moment transfer strength for exterior connections without beams is calculated using the same procedure as specified in ACI 318. Where spandrel beams exist, the strength should be modified to account for the torsional stiffness and strength of the spandrel beam.

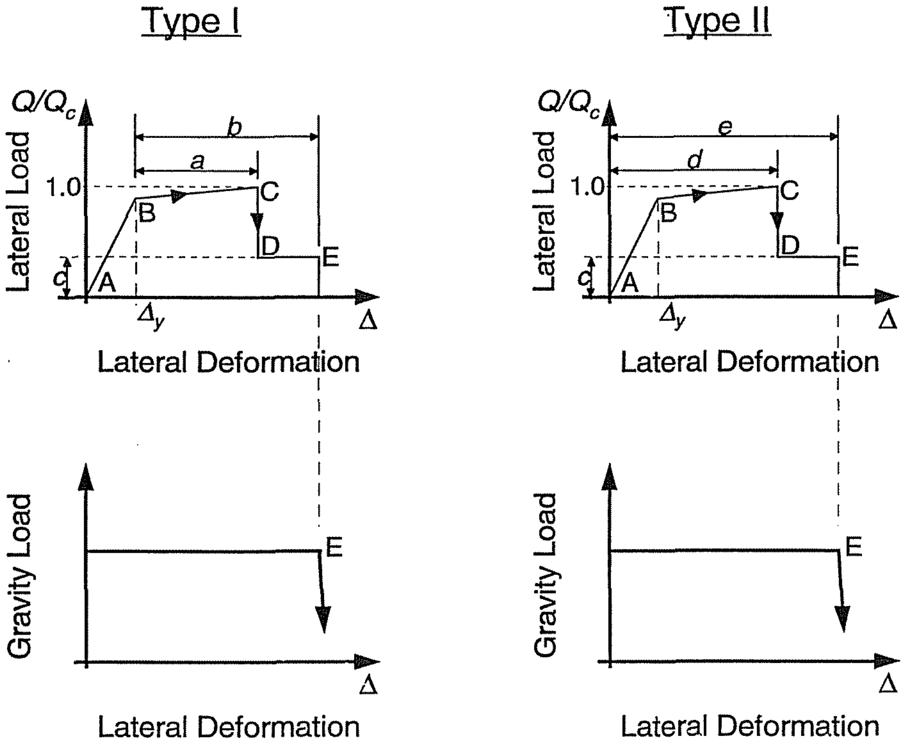


Figure 9-15. Generalized Load-Deformation Relations for Components

9.5.5 Component Deformability

9.5.5.1 General

The analysis should be capable of tracking the nonlinear load-deformation relation of components. Component load-deformation relations are generally composed of continuous linear segments. The general form of the load-deformation relation is discussed in Section 9.5.1. Deformation limits corresponding to loss of lateral load resistance and corresponding to loss of gravity load resistance should be defined.

Figure 9-15 illustrates a generalized load-deformation relation applicable to most concrete components. As shown, there are two ways to define deformations:

Type I: In this curve, deformations are expressed directly using terms such as strain, curvature, rotation, or elongation. The parameters a and b refer to those portions of the deformation that occur after yield, that is, the plastic deformations. Parameters a, b, and c are defined numerically in Tables 9-6 through 9-12 at the end of this chapter.

Type II: In this curve, deformations are expressed in terms such as shear angle and tangential drift ratio. The parameters d and e refer to total deformations measured from the origin. Parameters c, d, and e are defined numerically in Tables 9-6 through 9-12 at the end of this chapter.

Commentary: *Curve type I is convenient to use when the deformation is a flexural plastic hinge. Most computer programs for inelastic analysis will directly report the flexural plastic hinge rotation in this format, so that results can be compared readily with response limits (acceptance criteria). Curve type II is convenient to use when the deformation is interstory drift, shear angle, sliding shear displacement, or beam-column joint rotation. Both types are used in this methodology.*

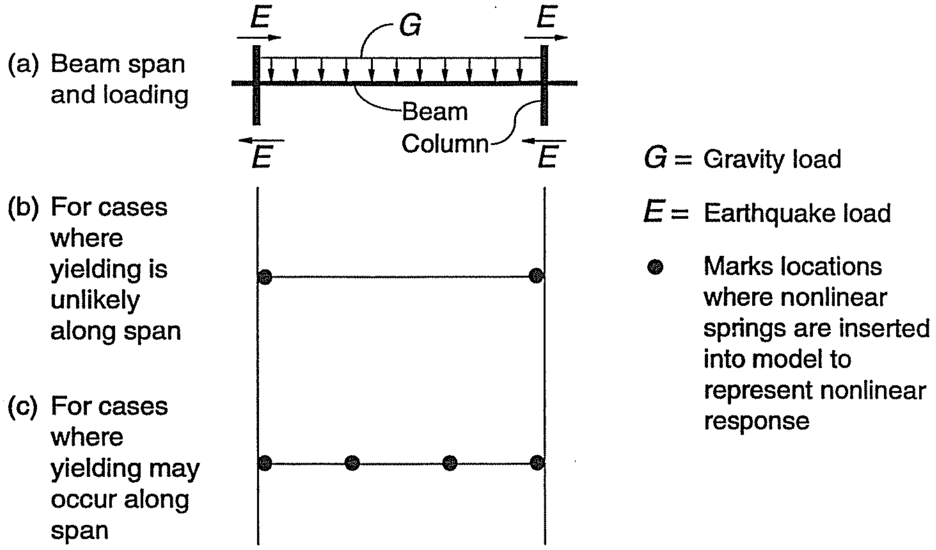


Figure 9-16. Simple Plastic Hinge Models for Beams

9.5.5.2 Beams

Beams may be modeled with concentrated plastic hinge models, distributed plastic hinge models, or other models whose behavior has been demonstrated to adequately represent important characteristics of reinforced concrete beam components subjected to lateral loading. The model should be capable of representing inelastic response along the component length, except where it is shown by equilibrium that yielding is restricted to the component ends. Where nonlinear response is expected in a mode other than flexure, the model should be able to represent that mode.

Monotonic load-deformation relations should be according to the generalized relation shown in Figure 9-15. The generalized deformation may be either the chord rotation or the plastic hinge rotation. Values of generalized deformations at points *B*, *C*, and *E* may be derived from experiments or rational analyses and should take into account the interactions between flexure and shear. Alternatively, where the generalized deformation is taken as rotation in the flexural plastic hinge zone, the plastic hinge rotation capacities may be as defined by Table 9-6 (at end of this chapter). In this table, the parameters *a*, *b*, and *c* refer to the measurements in Figure 9-15, type I.

Commentary: *Probably the most direct way to model a beam with currently available computer programs is as a line element having linearly elastic properties along the length with bilinear rotational springs at the ends of the line element (Figure 9-16b). Where yielding can occur along the span as well, rotational springs must be added to the model at critical points to capture potential yielding (Figure 9-16c). These springs may be assigned to have a rigid stiffness to the yield moment, followed by a reduced stiffness to represent post yield response. The post yield stiffness should result in a reasonable overall* *strain hardening of the beam load-deformation relation.*

Plastic hinge rotation capacities can be obtained from tests or can be calculated using principles of structural mechanics. One method for estimating plastic hinge rotation capacity by calculations is as follows. First, the moment-curvature relation is calculated using realistic estimates of material stress-strain relations. From these relations, the yield and ultimate curvatures, ϕ_y *and* ϕ_u*, are determinated. Next, the plastic hinge length,* l_p ($l_p = h/2$ *is an acceptable value that usually gives conservative results, where h is the section depth in the direction of loading) is estimated. Finally, the plastic hinge rotation capacity is estimated as* $\theta_p = (\phi_u - \phi_y)l_p$.

Alternatively, plastic hinge rotation capacities suitable for use with the methodology can be read directly from Table 9-6, where plastic rotation represented by the quantity a corresponds to the point where significant degradation in the moment-rotation relation occurs, and the plastic rotation represented by the quantity b corresponds to the point where loss in gravity load capacity is assumed. In the table, the plastic rotation represented by a is based on available test data (Aycardi et al. 1992,; Beres et al. 1992; CSSC 1994b; Pessiki et al. 1990; and Qi and Moehle 1991), supplemented by plastic rotation angle calculations and judgment. The plastic rotation represented by b is based primarily on the judgment of the project team, as supplemented by test results.

9.5.5.3 Columns

Columns may be modeled with concentrated plastic hinge models, distributed plastic hinge models, or other models whose behavior has been demonstrated to adequately represent important characteristics of reinforced concrete column components subjected to axial and lateral loading. Where nonlinear response is expected in a mode other than flexure, the model should be able to represent that mode. Where there are significant axial force variations under the action of earthquake loading, the model should also represent the effects of the variation on stiffness and strength properties. This can be achieved by using interaction surfaces for plastic hinge models. Fiber models usually can represent this effect directly.

Monotonic load-deformation relations should be according to the generalized relation shown in Figure 9-15. The generalized deformation may be either the chord rotation or the plastic hinge rotation. Values of the generalized deformation at points *B*, *C*, and *E* may be derived from experiments or rational analyses and should take into account the interactions between flexure, axial force, and shear. Alternatively, where the generalized deformation is taken as rotation in the flexural plastic hinge zone, the plastic hinge rotation capacities may be as defined by Table 9-7 (end of chapter). In this table, the parameters *a*, *b*, and *c* refer to the measurements in Figure 9-15, type I.

Commentary: *As with beams, probably the most direct way to model a column with currently available computer programs is as a line element having linear-elastic properties along its length with bilinear rotational springs at its ends. The general guidelines provided in the commentary to Section 9.5.5.2, including the guidelines on calculation of plastic hinge rotation capacity, apply for columns as well.*

Instead of calculating plastic hinge rotation capacities, the values provided in Table 9-7 may be used. In this table, the quantity a represents plastic rotation corresponding to the point where significant degradation in the moment-rotation relation occurs, and the quantity b corresponds to the point where loss in gravity load capacity is assumed. In the table, the plastic rotations were based on available test data (Lynn et al. 1995; Qi and Moehle 1991; CSSC 1994b), supplemented by plastic rotation angle calculations and judgment.

9.5.5.4 Beam-Column Joints

Better performance is expected when beam-column joints are stronger than adjacent framing components. If joints are stronger than the adjacent components, the joint region may be modeled as a stiff or rigid zone. If joints are not stronger than the adjacent components, the analytical model will have to represent the nonlinear load-deformation response. Joints may be modeled by using concentrated spring elements connecting beams to columns, or other models whose behavior has been demonstrated to adequately represent important characteristics of reinforced concrete beam-column joints subjected to lateral loading.

Monotonic load-deformation relations should be according to the generalized relation shown in Figure 9-15. Values of the generalized deformation at points *B*, *C*, and *E* may be derived from experiments or rational analyses. Alternatively, where the generalized deformation is taken as total shear angle in the joint, the total rotation capacities may be as defined by Table 9-8 (end of chapter). In this table, the parameters *c*, *d*, and *e*, refer to the measurements in Figure 9-15, type II.

Commentary: Probably the most direct way to model a joint with currently available computer programs is as a concentrated spring with nonlinear properties. The spring may be assigned a rigid stiffness to the yield point, with nonlinear response thereafter.

Joint shear-rotation capacities can be obtained from tests. Alternatively, shear distortion capacities suitable for use with the methodology can be read directly from Table 9-8, where the quantity d corresponds to the total shear angle at which significant degradation occurs, and the quantity e corresponds to the total shear angle where gravity load capacity should be assumed to be lost.

9.5.5.5 One-Way Slabs

One-way slabs may be modeled with the general procedures for beams identified in Section 9.5.5.2. Where the slab is part of a two-way slab system, the recommendations of Section 9.5.5.6 may be used.

9.5.5.6 Two-Way Slabs and Slab-Column Connections

Two-way slabs and slab-column connections may be modeled as described in Section 9.4.2.2. Where the frame is modeled using the effective beam width model, the slab may be modeled by using concentrated plastic hinge models, distributed plastic hinge models, or other models. Where the frame is modeled using the equivalent frame model, the slab may be modeled as above, and the connection with the column may be modeled as a bilinear spring. The model should be capable of representing inelastic response along the component length, except where it is shown by equilibrium that yielding is restricted to the component ends. Where nonlinear response is expected in a mode other than flexure or slab-column connection rotation, the model should be able to represent that mode.

Monotonic load-deformation relations should be according to the generalized relation shown in Figure 9-15. The generalized deformation may be either the chord rotation or the plastic hinge rotation. Values of the generalized deformation at points *B*, *C*, and *E* may be derived from experiments or rational analyses and should take into account the interactions between flexure and shear. Alternatively, where the generalized deformation is taken as rotation in the flexural plastic hinge zone, or rotation of the spring connecting the slab and column, the plastic hinge rotation capacities may be as defined by Table 9-9 (end of chapter). In this table, the parameters *a*, *b*, and *c* refer to the measurements in Figure 9-15, type I.

Commentary: *The slab span generally can be treated as a flexural framing member, having properties defined by those of the slab cross section. Flexural strengths of the slab should be represented as defined in Section 9.5.4.7. When modeled in this fashion, the plastic rotation capacities should be according to Table 9-9; the*

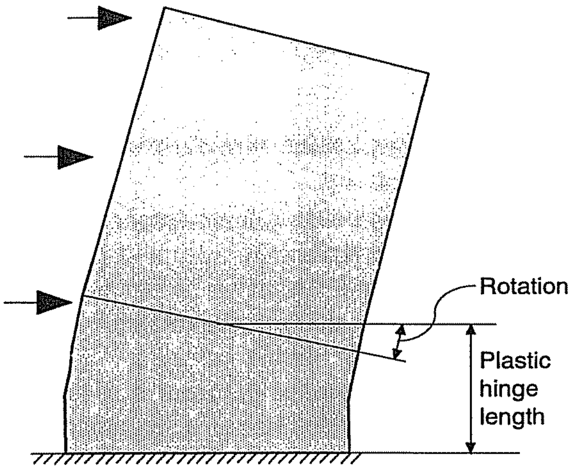


Figure 9-17. Plastic Hinge Rotation in a Wall or Wall Segment Governed by Flexure

values in this table are based on results from tests on slab-column connections and represent rotations at which failures have been observed. It is essential that the nonlinear analysis model represent the behavior of the slab-column connection in addition to the slab and column components. Nonlinear response of slab-column connections is a complex function of flexure, shear, torsion, and bond actions. Some detailed models have been reported (Hawkins 1980; Luo et al. 1994). As an alternative, two simplified approaches are suggested in this methodology. In the first, the slab-column frame is modeled using the effective beam width model, in which case the slab rotation capacities are limited to the values in Table 9-9. In the second approach, the slab-column frame is modeled using the equivalent frame model, in which case both the slab and the connection have plastic rotation capacities according to Table 9-9. The limiting rotation values in Table 9-9 are from various tests (Pan and Moehle 1989; Martinez et al. 1994; Hwang and Moehle 1993;. Graf and Mehrain 1992; Durrani et al. 1995), supplemented by judgment of the project team.

9.5.5.7 Walls, Wall Segments, and Wall Coupling Beams

Walls may be modeled using any procedures that satisfy the requirements of equilibrium and kinematics where these models are verified by tests. Alternatively, the simplified approach presented in this section may be used. In this approach, the selection of a model for walls and wall segments depends on whether the wall or wall segment is governed by flexure or by shear, as described below.

Where the wall or wall segment is governed by flexure, the load-deformation relation should be of the type shown in Figure 9-15, type I. In this figure, the generalized deformation is to be taken as the rotation over the plastic hinging region (Figure 9-17). The rotation at point B of Figure 9-15 corresponds to the yield point, θ_y, as given by the following expression:

$$\theta_y = \left(\frac{M_y}{E_c I}\right)\left(l_p\right) \qquad (9\text{-}12)$$

where M_y is the yield moment of the wall or wall segment, which may be calculated as the moment at which reinforcement in the boundary zone (or outer 25% of the wall length) yields, E_cI is the flexural rigidity according to Section 9.5.3, and l_p represents the assumed plastic hinge length, which may be taken equal to 0.5 times the flexural depth of the component, but less than 50% of the segment length for wall segments. The plastic hinge rotation capacities within the same length l_p are defined by Table 9-10 (end of chapter). In this table, the parameters a, b, and c refer to the measurements in Figure 9-15, type I.

Where the wall or wall segment is governed by shear, it is more appropriate to use shear drift ratio (Figure 9-18) as the deformation measure. Shear drift ratio capacities are defined in Table 9-11. In this table, the parameters c, d, and e refer to the measurements in Figure 9-15, type II. Where sliding along a construction joint controls overall performance (see Section 9.4.3.1), the parameters in Table 9-11 may still be applied, but the story

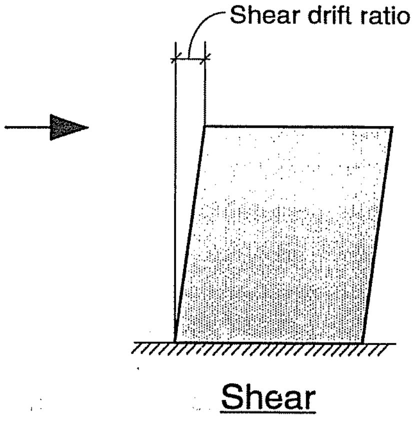


Figure 9-18. Shear Drift Ratio for Walls and Wall Segments Governed by Shear

height over which the shear drift ratio is calculated should be assumed to be equal to 50 inches regardless of the actual story height.

Coupling beam models should be established considering the aspect ratio of the beams, and model development should be guided by judgment. Where the beam is relatively slender ($l/h \geq 5$ or so), it should be modeled as described in Section 9.5.5.2. Where the beam is less slender ($l/h < 5$ or so), and where the beam is detailed essentially as a segment of the wall, use the guidelines presented above for wall segments. Where the beam is less slender ($l/h < 5$ or so), and is reinforced as a beam distinct from the wall, the following approach should be considered. Model the beam considering both shear and flexural deformations. Use chord rotation (Figure 9-19) as the relevant deformation measure. The chord rotation capacities are defined by Table 9-12. In this table, the parameters c, d, and e refer to the measurements in Figure 9-15, type II.

Commentary: Walls are generally primary lateral force resisting elements of a building. The engineer should attempt to establish a realistic model for the wall on the basis of available test data and advanced analysis techniques. The wide variety of wall geometries precludes definition of response parameters except in a relatively rudimentary sense. The quantities presented in this section are presented to help guide selection of overall modeling parameters. Alternative parameters may be appropriate in individual cases.

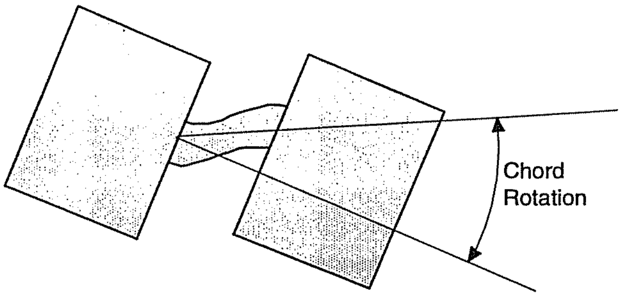


Figure 9-19. Chord Rotation for Coupling Beams

Where a wall is governed by flexural response, it is expected that the analysis model will represent this behavior directly. In tests of walls, the plastic hinge length typically ranges between about 0.5 and 1.0 times the flexural depth (flexural depth is measured in the direction of the corresponding shear force), with values tending toward the higher range for high levels of shear. Study of several cantilever walls tested in laboratories indicates that it is appropriate to assume a length of $0.5l_w$ (Wallace and Moehle 1992). For wall segments, this length may extend beyond the midlength, in which case it is appropriate to limit the plastic hinge length to half the member length.

For walls or wall segments controlled by shear, shear drift ratio is used as the deformation measure. As shown in Figure 9-18, the shear drift is measured as the overall distortion of the component; where the boundaries are not rotated significantly (as may be the case for piers between very stiff spandrels), the shear drift ratio may be taken equal to the interstory drift ratio. One exception is where sliding occurs along a construction joint. In this case, the total absolute slip along the joint needs to be controlled. For this purpose, the effective height is assumed to be 50 inches. Therefore, for example, the shear drift ratio of 0.0075 in Table 9-11 corresponds to a slip at the joint equal to 0.0075 x 50 inches = 0.375 inches. See Section 9.4.3.1 regarding the need to model construction joints.

Symmetrical walls can be expected to have similar hysteretic behavior with either end in compression. Such behavior would lead to symmetric hysteresis loops (Ali and Wight 1991). By contrast, T-shaped or other flanged walls may show asymmetric load-deformation relations (Thomsen and Wallace 1995). Modeling should account for critical cases. Data from Ali and Wight (1991), Thomsen and Wallace (1995), and Paulay (1986) were used with judgment to establish the parameters in Table 9-10.

Low-rise walls governed by shear, especially those subject to sliding along the main flexural crack at the interface with the foundation, can be expected to have severely pinched and slightly degrading hysteresis loops (Saatcioglu 1995). Additional data on low-rise walls is found in Sozen and Moehle (1993) and Wood (1990).

The behavior of reinforced concrete coupling beams depends on aspect ratio and reinforcement details. Measured relations between load and chord rotation for wall segments with $l/h \approx 2$ *are given in Paulay (1971a, 1971b). For coupling beams with conventional longitudinal reinforcement, non conforming transverse reinforcement results in pinched hysteresis loops compared with loops for conforming transverse reinforcement. Coupling beams with diagonal reinforcement showed the most stable loops of all.*

9.6 Notations

a = parameter to measure plastic deformation capacity at which lateral force resistance degrades

A_g = gross cross-sectional area

A_j = effective horizontal joint cross-sectional area

A_s = cross-sectional area of reinforcement

A'_s = the cross-sectional area of longitudinal reinforcement in compression

A_v = cross-sectional area of transverse reinforcement resisting shear

A_w = cross-sectional area of web

b = parameter to measure plastic deformation capacity at which gravity load resistance degrades

b_w = web width

c = residual strength ratio

c_1 = column cross-sectional dimension in the direction of framing

c_2 = column cross-sectional dimension transverse to direction of framing

d = parameter to measure total deformation capacity at which lateral force resistance degrades

d = effective depth of flexural component

d_b = diameter of longitudinal reinforcement

e = parameter to measure total deformation capacity at which gravity load resistance degrades

E_c = Young's modulus for concrete

f_c' = unconfined concrete compressive strength

f_{cc} = concrete compressive strength

f_s = stress in reinforcement

f_y = reinforcement yield stress

G = shear modulus for concrete

h = flexural depth of cross section

h = slab thickness

h_c = cross sectional dimension of column core

h_w = wall height

I_g = moment of inertia of gross concrete section

k = coefficient in Equation 9-4

K = rotational spring constant

l = length from the point of maximum moment to the point of contraflexure

l_1 = center-to-center span in the direction of framing

l_b = development, hook, or lap-splice length provided

l_d = development, hook, or lap-splice length required by ACI 318
l_e = embedment length of reinforcement
l_p = plastic hinge length used for calculation of deformation capacity
l_w = wall length, measured horizontally in the direction of applied shear
M = flexural moment

M_{gCS} = column strip moment due to gravity loads in a two-way slab
M_n = nominal flexural strength of a component
M_{nCS} = design flexural strength of the column strip in a two-way slab
M_p = plastic moment strength
M_y = yield moment of a wall or wall segment
N = axial compression force in pounds (zero for tension force)
P = design axial load
P_o = nominal axial load strength at zero eccentricity
Q = external load effect (e.g., moment, shear, axial force)
Q_c = strength to resist external load effect, Q
s = longitudinal spacing of transverse reinforcement
t_w = web thickness of shear wall
V = design shear force
V_c = shear strength contribution attributed to concrete
V_g = gravity shear acting on the slab critical section as defined by ACI 318
V_o = the direct punching shear strength as defined by ACI 318
V_n = nominal shear strength
V_s = shear strength contribution attributed to reinforcement

β = coefficient to represent stiffness reduction in slab-column framing owing to cracking
β_n = coefficient to modify effectiveness of horizontal reinforcement in resisting shear in walls
ϕ = strength reduction factor from ACI 318
ϕ = curvature
ϕ_u = ultimate curvature capacity
ϕ_y = yield curvature
Δ = deformation,
Δ_y = yield deformation
ε_{cu} = maximum compressive strain capacity of concrete
γ = coefficient used to define joint shear strength
γ_f = portion of moment transferred by flexure per the specifications of ACI 318
λ = coefficient to modify strength on the basis of aggregate density
μ = shear friction coefficient
θ = hinge rotation
θ_y = yield rotation of plastic hinge in a flexural component
θ_p = plastic hinge rotation capacity in a flexural component
ρ = ratio of nonprestressed tension reinforcement
ρ' = ratio of nonprestressed compression reinforcement
ρ'' = volumetric ratio of transverse reinforcement
ρ_{bal} = reinforcement ratio producing balanced strain conditions
ρ_n = volumetric ratio of horizontal reinforcement resisting shear in a wall.

Table 9-6. Modeling Parameters for Nonlinear Procedures—Reinforced Concrete Beams

			Modeling Parameters[3]		
			Plastic Rotation Angle, rad		*Residual Strength Ratio*
Component Type			*a*	*b*	*c*
1. Beams controlled by flexure[1]					
$\frac{\rho-\rho'}{\rho_{bal}}$	*Transverse Reinforcement*[2]	$\frac{V}{b_w d\sqrt{f_c'}}$[4]			
≤ 0.0	C	≤ 3	0.025	0.05	0.2
≤ 0.0	C	≥ 6	0.02	0.04	0.2
≥ 0.5	C	≤ 3	0.02	0.03	0.2
≥ 0.5	C	≥ 6	0.015	0.02	0.2
≤ 0.0	NC	≤ 3	0.02	0.03	0.2
≤ 0.0	NC	≥ 6	0.01	0.015	0.2
≥ 0.5	NC	≤ 3	0.01	0.015	0.2
≥ 0.5	NC	≥ 6	0.005	0.01	0.2
2. Beams controlled by shear[1]					
stirrup spacing $\le d/2$			0.0	0.02	0.2
stirrup spacing $> d/2$			0.0	0.01	0.2
3. Beams controlled by inadequate development or splicing along the span[1]					
stirrup spacing $\le d/2$			0.0	0.02	0.0
stirrup spacing $> d/2$			0.0	0.01	0.0
4. Beams controlled by inadequate embedment into beam-column joint[1]					
			0.015	0.03	0.2

1. When more than one of the conditions 1, 2, 3, and 4 occur for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "transverse reinforcement," "C" and "NC" are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic region: 1) closed stirrups are spaced at $\le d/3$, and 2) for components of moderate and high ductility demand the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.
3. Linear interpolation between values listed in the table is permitted.
4. V = design shear force
5. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).

Table 9-7. Modeling Parameters for Nonlinear Procedures—Reinforced Concrete Columns

			Modeling Parameters[4]		
			Plastic Rotation Angle, rad		Residual Strength Ratio
Component Type			a	b	c
1. Columns controlled by flexure[1]					
$\frac{P}{A_g f_c'}$ [5]	Transverse Reinforcement[2]	$\frac{V}{b_w d\sqrt{f_c'}}$ [6]			
≤ 0.1	C	≤ 3	0.02	0.03	0.2
≤ 0.1	C	≥ 6	0.015	0.025	0.2
≥ 0.4	C	≤ 3	0.015	0.025	0.2
≥ 0.4	C	≥ 6	0.01	0.015	0.2
≤ 0.1	NC	≤ 3	0.01	0.015	0.2
≤ 0.1	NC	≥ 6	0.005	0.005	-
≥ 0.4	NC	≤ 3	0.005	0.005	-
≥ 0.4	NC	≥ 6	0.0	0.0	-
2. Columns controlled by shear[1,3]					
Hoop spacing ≤ $d/2$, or $\frac{P}{A_g f_c'}$ [5] ≤ 0.1			0.0	0.015	0.2
other cases			0.0	0.0	0.0
3. Columns controlled by inadequate development or splicing along the clear height[1,3]					
Hoop spacing ≤ $d/2$			0.01	0.02	0.4
Hoop spacing > $d/2$			0.0	0.01	0.2
4. Columns with axial loads exceeding $0.70P_o$[1,3]					
Conforming reinforcement over the entire length			0.015	0.025	0.02
All other cases			0.0	0.0	0.0

1. When more than one of the conditions 1, 2, 3, and 4 occur for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "transverse reinforcement," "C" and "NC" are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic hinge region: 1) closed hoops are spaced at ≤ $d/3$, and 2) for components of moderate and high ductility demand the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.
3. To qualify, 1) hoops must not be lap spliced in the cover concrete, and 2) hoops must have hooks embedded in the core or must have other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
4. Linear interpolation between values listed in the table is permitted.
5. P = Design axial load
6. V = Design shear force
7. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).

Table 9-8. Modeling Parameters for Nonlinear Procedures—Reinforced Concrete Beam-Column Joints

			Modeling Parameters[4]		
			Shear Angle, rad		Residual Strength Ratio
Component Type			d	e	c
1. Interior joints					
$\frac{P}{A_g f_c'}$ [2]	Transverse Reinforcement[1]	$\frac{V}{V_n}$ [3]			
≤ 0.1	C	≤ 1.2	0.015	0.03	0.2
≤ 0.1	C	≥ 1.5	0.015	0.03	0.2
≥ 0.4	C	≤ 1.2	0.015	0.025	0.2
≥ 0.4	C	≥ 1.5	0.015	0.02	0.2
≤ 0.1	NC	≤ 1.2	0.005	0.02	0.2
≤ 0.1	NC	≥ 1.5	0.005	0.015	0.2
≥ 0.4	NC	≤ 1.2	0.005	0.015	0.2
≥ 0.4	NC	≥ 1.5	0.005	0.015	0.2
2. Other joints					
$\frac{P}{A_g f_c'}$ [2]	Transverse Reinforcement[1]	$\frac{V}{V_n}$ [3]			
≤ 0.1	C	≤ 1.2	0.01	0.02	0.2
≤ 0.1	C	≥ 1.5	0.01	0.015	0.2
≥ 0.4	C	≤ 1.2	0.01	0.02	0.2
≥ 0.4	C	≥ 1.5	0.01	0.015	0.2
≤ 0.1	NC	≤ 1.2	0.005	0.01	0.2
≤ 0.1	NC	≥ 1.5	0.005	0.01	0.2
≥ 0.4	NC	≤ 1.2	0.0	0.0	-
≥ 0.4	NC	≥ 1.5	0.0	0.0	-

1. Under the heading "transverse reinforcement," "C" and "NC" are abbreviations for conforming and non-conforming details, respectively. A joint is conforming if closed hoops are spaced at ≤ $h_c/3$ within the joint. Otherwise, the component is considered non-conforming. Also, to qualify as conforming details under condition 2, 1) hoops must not be lap spliced in the cover concrete, and 2) hoops must have hooks embedded in the core or must have other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
2. The ratio $\frac{P}{A_g f_c'}$ is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional area of the joint and the concrete compressive strength. The design axial force is to be calculated considering design gravity and lateral forces.
3. The ratio $\frac{V}{V_n}$ is the ratio of the design shear force to the shear strength for the joint.
4. Linear interpolation between values listed in the table is permitted.
5. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).

Table 9-9. Modeling Parameters for Nonlinear Procedures—Two-Way Slabs and Slab-Column Connections

		Modeling Parameters[4]		
		Plastic Rotation Angle, rad		Residual Strength Ratio
Component Type		a	b	c
1. Slabs controlled by flexure, and slab-column connections[1]				
$\frac{V_g}{V_o}$ [2]	Continuity Reinforcement[3]			
≤ 0.2	Yes	0.02	0.05	0.2
0.2	Yes	0.0	0.04	
≤ 0.2	No	0.02	0.02	-
≥ 0.4	No	0.0	0.0	-
2. Slabs controlled by inadequate development or splicing along the span[1]				
		0.0	0.02	0.0
3. Slabs controlled by inadequate embedment into slab-column joint[1]				
		0.015	0.03	0.2

1. When more than one of the conditions 1, 2, and 3 occur for a given component, use the minimum appropriate numerical value from the table.
2. V_g = the gravity shear acting on the slab critical section as defined by ACI 318, V_o = the direct punching shear strength as defined by ACI 318.
3. Under the heading "Continuity Reinforcement," assume "Yes" where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, assume "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, assume "No."
4. Linear interpolation between values listed in the table is permitted.
5. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).

Table 9-10. Modeling Parameters for Nonlinear Procedures—Walls and Wall Segments Controlled by Flexure

			Modeling Parameters[4]		
			Plastic Rotation Angle, rad		Residual Strength Ratio
Component Type			a	b	c
1. Walls and wall segments controlled by flexure					
$\frac{(A_s - A_s')f_y + P}{t_w l_w f_c'}$ [1]	$\frac{V}{t_w l_w \sqrt{f_c'}}$ [2]	Boundary Element[3]			
≤ 0.1	≤ 3	C	0.015	0.020	0.75
≤ 0.1	≥ 6	C	0.010	0.015	0.40
≥ 0.25	≤ 3	C	0.009	0.012	0.60
≥ 0.25	≥ 6	C	0.005	0.010	0.30
≤ 0.1	≤ 3	NC	0.008	0.015	0.60
≤ 0.1	≥ 6	NC	0.006	0.010	0.30
≥ 0.25	≤ 3	NC	0.003	0.005	0.25
≥ 0.25	≥ 6	NC	0.002	0.004	0.20

1. A_s = the cross-sectional area of longitudinal reinforcement in tension, A_s' = the cross-sectional area of longitudinal reinforcement in compression, f_y = yield stress of longitudinal reinforcement, P = axial force acting on the wall considering design load combinations, t_w = wall web thickness, l_w = wall length, and f_c' = concrete compressive strength.
2. V = the design shear force acting on the wall, and other variables are as defined above.
3. The term "C" indicates the boundary reinforcement effectively satisfies requirements of ACI 318. The term "NC" indicates the boundary requirements do not satisfy requirements of ACI 318.
4. Linear interpolation between values listed in the table is permitted.
5. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).

Table 9-11. Modeling Parameters for Nonlinear Procedures—Walls and Wall Segments Controlled by Shear

	Modeling Parameters		
	Shear Drift Ratio, rad		Residual Strength Ratio
Component Type Conditions	d	e	c
1. Walls and wall segments			
All walls and wall segments controlled by shear	0.0075	0.02	0.4

1. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).

Table 9-12. Modeling Parameters for Nonlinear Procedures—Coupling Beams

		Modeling Parameters[3]		
		Chord Rotation, rad		*Residual Strength Ratio*
Component Type		*d*	*e*	*c*
1. Coupling beams controlled by flexure				
Longitudinal reinforcement and transverse reinforcement[1]	$\frac{V}{b_w d \sqrt{f_c'}}$ [2]			
Conventional longitudinal reinforcement with	≤ 3	0.025	0.040	0.75
conforming transverse reinforcement	≥ 6	0.015	0.030	0.50
Conventional longitudinal reinforcement with non-	≤ 3	0.020	0.035	0.50
Conforming transverse reinforcement	≥ 6	0.010	0.025	0.25
Diagonal reinforcement	N/A.	0.030	0.050	0.80
2. Coupling beams controlled by shear				
Longitudinal reinforcement and transverse reinforcement[1]	$\frac{V}{b_w d \sqrt{f_c'}}$ [2]			
Conventional longitudinal reinforcement with	≤ 3	0.018	0.030	0.60
conforming transverse reinforcement	≥ 6	0.012	0.020	0.30
Conventional longitudinal reinforcement with non-	≤ 3	0.012	0.025	0.40
Conforming transverse reinforcement	≥ 6	0.008	0.014	0.20

1. Conventional longitudinal steel consists of top and bottom steel parallel to the longitudinal axis of the beam. The requirements for conforming transverse reinforcement are: 1) closed stirrups are to be provided over the entire length of the beam at spacing not exceeding $d/3$; and 2) the strength provided by the stirrups (V_s) should be at least three-fourths of the design shear.
2. V = the design shear force on the coupling beam in pounds, b_w = the web width of the beam, d = the effective depth of the beam, and f_c' = concrete compressive strength in psi.
3. Linear interpolation between values listed in the table is permitted.
4. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).

Chapter 10

Foundation Effects

Audience Interest Spectrum

Owner	*Architect*	*Bldg. Official*	*Engineer*	*Analyst*
░	░	▒	▒	█

10.1 General

Deformation and movement of foundations can significantly affect the seismic response and performance of structures. Central to the general structural analysis methodology is a nonlinear structural analysis technique whereby the behavior of structural components is represented by nonlinear load-displacement relationships. This chapter formulates analogous techniques, compatible and consistent with the general methodology, to include the effects of foundations in the overall procedure. By utilizing these techniques, it is possible to expand the structural model with representations of the foundations to be included directly in the analysis. General guidance on when and how to use these techniques is also included.

The general methodology consists of the formulation of an analytical model of the features of a building that are pertinent to its performance during earthquakes. This model is an assembly of systems. The lateral forces induced by seismic shaking are imposed on the lateral load system. This system may overlap and include portions of the vertical load carrying system. Systems comprise structural elements—concrete moment frame elements acting in conjunction with floor diaphragm elements to form a lateral load system. Individual beams and columns are the components of the moment frame element. Behavior parameters (e.g., strength, stiffness) and acceptability criteria (e.g., ductility, drift) may exist at the system, element, or component level. For example, the maximum roof displacement for the lateral load system might be limited to an acceptable value so long as the story drifts within the moment frame elements are limited and the rotational ductility demand for the column components is below certain limits.

The effects of foundations can be included in the analysis by extending the basic model to include the foundation system in a directly analogous manner, as discussed in Section 10.2. The response parameters of foundation elements are dependent upon properties of structural and geotechnical components. Spread footing 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. Section 10.3 formulates some generic models for typical foundation elements. Modeling rules and acceptance criteria for structural components of foundations are in Chapters 9 and 11. In many respects the modeling of geotechnical components is similar to that of structural ones. The goal is to formulate a relationship between the force applied to a component and the corresponding displacement. Because of the properties of soil materials, some uncertainties and approximations are necessary, as reviewed in Section 10.4. The force-displacement relationships for geotechnical components depend upon the strength and stiffness properties of soil materials. Section 10.5 provides some typical values for pertinent properties and guidance on when and how to pursue data specific to a particular building.

The basic analysis methodology of Chapter 8 implies several assumptions common to conventional structural modeling. The approach is essentially a finite-element analysis where continuous properties are concentrated at discrete points to simplify the procedure. Hysteretic behavior is included directly by the inelastic action of the individual finite elements. Viscous damping within the soil material is neglected. Kinematic effects of soil-structure interaction are not included in the methodology. These limitations are acceptably conservative for the large majority of structures and foundations when the simplified inelastic procedures of the methodology are used. The analysis results in a prediction of the displacements that might occur in the structure and foundation for a given earthquake demand. Section 10.6 discusses the acceptability of these displacements for geotechnical components. In some cases, modifications to existing foundations are warranted. Section 10.7 is an overview of foundation retrofit measures.

10.2 Foundation System and Global Structural Model

The analysis of the seismic performance of a building can include the effects of foundations directly by including foundations in the structural model. The nature and extent of the structural model, including the foundation system, depends on a number of interrelated factors. The structural engineer makes judgments as to the level of refinement necessary to capture the important modes of behavior for a given building. The appropriate considerations for foundation systems are similar.

10.2.1 Factors Affecting Foundation Models

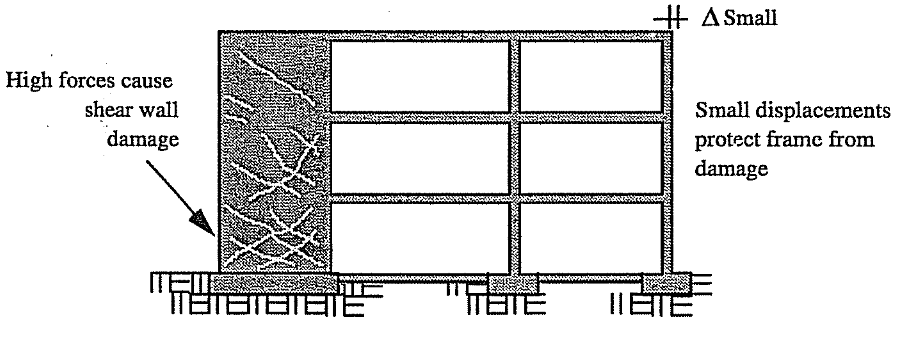
Foundation stiffness and strength influence the seismic performance of a structure. The structural engineer determines how foundation effects are included in the analysis model for the evaluation and retrofit of an existing building. In many instances the expert assistance of a geotechnical engineer is essential. Geotechnical engineers must keep in mind that "stiff and strong" is not necessarily better than "flexible and weak". Soft-weak assumptions for soils properties are not always conservative for the structure. The best information is a range of values to envelope possible conditions. Highly accurate estimates of soil properties can be expensive to generate. Decisions often can be made with relatively crude information by utilizing simplified parametric studies to get an approximate idea of the importance of individual structural and foundation characteristics. Many of the factors influencing foundation modeling are reviewed qualitatively below. In some instances, approximate "rules of thumb" are offered. Ultimately, however, the structural and geotechnical engineer must make judgments based heavily upon experience.

10.2.1.1 Geotechnical Conditions

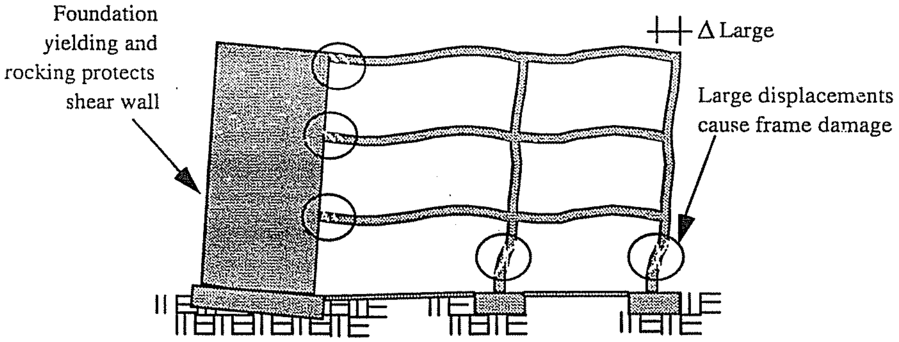
Softer, weaker soils are likely to influence seismic response. This is particularly true if there are observable signs of foundation distress, such as excessive displacements or settlement related to structural distress in the building itself. This generalization about soft soils should not imply that foundation effects are never a factor with hard sites. A common foundation effect is the uplifting of spread footings, particularly beneath tall, stiff lateral load elements. Stiff, strong soil materials at the toe of these elements normally do not restrict rocking.

Foundations for which existing dead and live loads are a large percentage of their ultimate capacity are more likely to affect structural behavior than others. If existing loads are over approximately 50 to 67% of capacity, large seismic

Foundation stiffness and strength affect various structural components differently.



Stiff/Strong Foundation



Flexible/Weak Foundation

Stiff/strong is not always favorable; nor is flexible/weak always conservative.

forces can result in significant permanent displacements at the foundation. A large variation in relative ratios among existing foundation element dead and live loads to the ultimate capacity of each element can imply a potential for significant redistribution of load in the inelastic range. This occurs when relatively highly loaded footings yield in advance of others. Inelastic redistribution of forces due to foundation effects can pose problems by inducing torsion in an apparently regular building.

10.2.1.2 Basic Structural System

In general, relatively slender shear wall buildings are the most sensitive to foundation effects under seismic loading. Short, multiple-bay moment frames can often be evaluated by neglecting foundation effects entirely. However, most buildings fall somewhere in between. Structures with periods ranging between 0.3 to 1.0 second are more sensitive than others to foundation effects. Table 10-1 provides a qualitative summary of basic systems and their relative sensitivity to foundation effects.

In Table 10-1 there is a distinction made between load-bearing shear walls and those acting in conjunction with a frame. Even if the frame is considered to be secondary (vertical load carrying), foundation rotation at the base of a shear wall can impose large displacement demand on the frame elements. Note also that, although complete frame buildings are generally less sensitive to foundation effects, tall, narrow frames can be sensitive to uplift of foundations due to large overturning forces.

Although long frames are not particularly sensitive to overall foundation rotation, the fixity of column bases can be an important consideration. With sufficient data, it is possible to model the stiffness and strength of the foundation elements for an explicit solution. Often, however, it is simpler and sufficient to bracket the solution with a fixed and pinned assumption for column fixity. In most instances the detail at the base of the column is the most influential factor affecting fixity.

Table 10-1. Sensitivity of Structural Systems to Foundation Effects

Structural Description	Aspect Ratio[1]	Relative Sensitivity to Foundation Movement
Slender shear wall-frames	$h/l > 2\pm$	High
Slender bearing shear walls		
Narrow frames		
Short shear wall-frames	$h/l < 2\pm$	Moderate
Short bearing shear walls		
Long frames		Low

1. where h = height of building and l = width of lateral load elements

10.2.1.3 Foundation Systems

Foundation systems for concrete structures consist typically of either shallow or deep elements or, less frequently, a combination of both. Shallow foundations normally are isolated or continuous spread footings, or large mats that are vertically supported by bearing directly on soil. Compared with deep foundations, they are relatively flexible in resisting vertical and/or rotational actions. Resistance to uplift is restricted to superimposed existing loads.

Most deep foundation elements are driven piles of steel or concrete or drilled cast-in-place concrete piers. These components rely on friction and/or end bearing for downward vertical support. Piers and piles are capable of significant resistance to uplift provided that they are adequately tied to the structure. Although deep foundations are relatively stiff and strong, this does not mean that foundation movements will not affect the structural response. In one of the example building analyses (Barrington Medical Office Building), very small drilled pier movements changed the ultimate limit state of the shear walls above from shear to flexure.

Combined systems of shallow and deep elements can be sensitive to foundation effects because of the inherent differences in strength and stiffness, particularly in the inelastic range. When a shallow footing beneath a shear wall begins to rock, a significant redistribution of load can ensue if other walls are supported on deep elements.

Often basic foundation elements are interconnected by pile caps, grade beams, basement walls, slabs, or other structural elements to form the entire foundation system (see Figure 10-1). These can influence the relative sensitivity of the performance to foundation effects. This is particularly true of basement walls, which can spread overturning forces over large distances and increase rotational resistance.

Many foundation systems are relatively stiff and strong in the horizontal direction. Passive pressures against pile caps or footings, and friction under slabs and footings, act simultaneously to transfer loads from the structure. Approximate comparison of the horizontal stiffness and strength of the foundation with those of the structure can provide insight into the necessity of including the horizontal degree of freedom in the analysis.

10.2.1.4 Performance Objectives

Seismic performance objectives beyond Life Safety are sensitive to the degree of inelastic demand throughout the structure. Permanent ground displacements related to foundation movements can impede the post-earthquake serviceability of a building. In general these are very difficult to predict. Greater refinement and accuracy in foundation modeling based on geotechnical investigations, tests, and analyses can help in these instances.

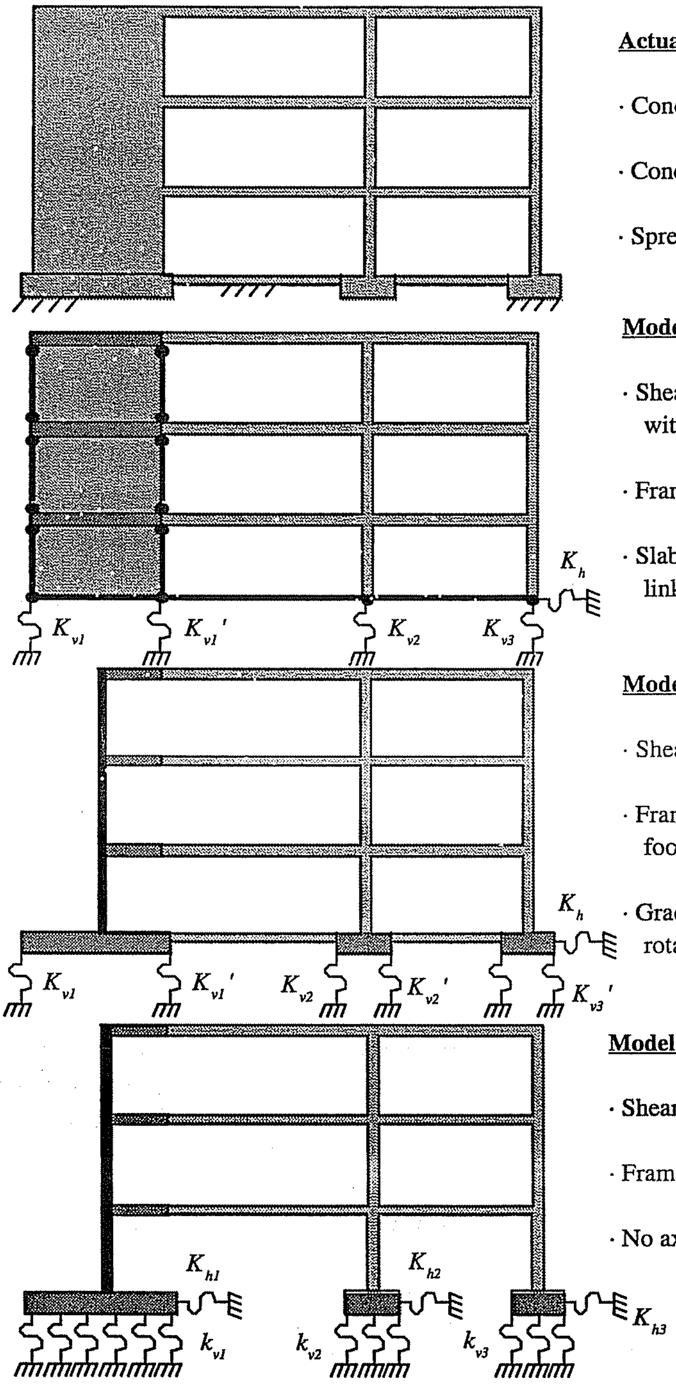
10.2.2 Assembling a Coordinated Global Structural Model

The structural engineer can arrange individual elements with appropriate component material properties geometrically to form a model of the foundation system. This process is similar to that for other structural systems and requires judgment and experience to strike a balance between accuracy and simplicity. A model that omits an important physical characteristic of a structure might yield unreliable results. At the other extreme, a model that is unnecessarily complex increases the chance for undetected errors and can obscure a basic understanding of the behavior of the building. When including the foundation system in the global structural model, it is important to coordinate the formation of the model with that of the structure above. In this way the behavioral characteristics important to the specific seismic performance of the building are effectively represented.

Figure 10-1 depicts a simple structure consisting of a shear wall and frame, which might act together as a lateral load carrying system for a building. The foundation system consists of conventional spread footings which might be interconnected by a grade beam or a slab on grade. The various two-dimensional models shown below the actual structure illustrate different modeling assumptions depending upon the actual characteristics of the systems, elements, and components of the entire structure, including the foundation. Each model represents the supporting soil by components with properties K. These are situated in the models to represent the spread footing elements.

In Model A, panel components represent the shear distortion properties of the wall and act in conjunction with axial link components, concentrating the bending properties at the ends of the wall.

The soil components beneath the first-floor columns represent both the vertical and the rotational stiffness of the spread footing element. In Models B and C, a column component represents both the shear and the bending properties of the wall. In these cases, the rigid beam element at the base of the wall transfers the rotational restraint of the footing element to the shear wall column.



Actual Structure

- Concrete shear wall
- Concrete frame
- Spread footings

Model A

- Shear walls modeled as shear panels with axial links at boundaries
- Frame columns pinned at base
- Slab provides axial but not rotational linkage of footings

Model B

- Shear walls modeled as columns
- Frame columns fixed at base to footings
- Grade beam provides axial and rotational linkage of footings

Model C

- Shear walls modeled as columns
- Frame columns fixed at base to footings
- No axial or rotational linkage of footings

Figure 10-1. Global and Foundation Modeling Alternatives

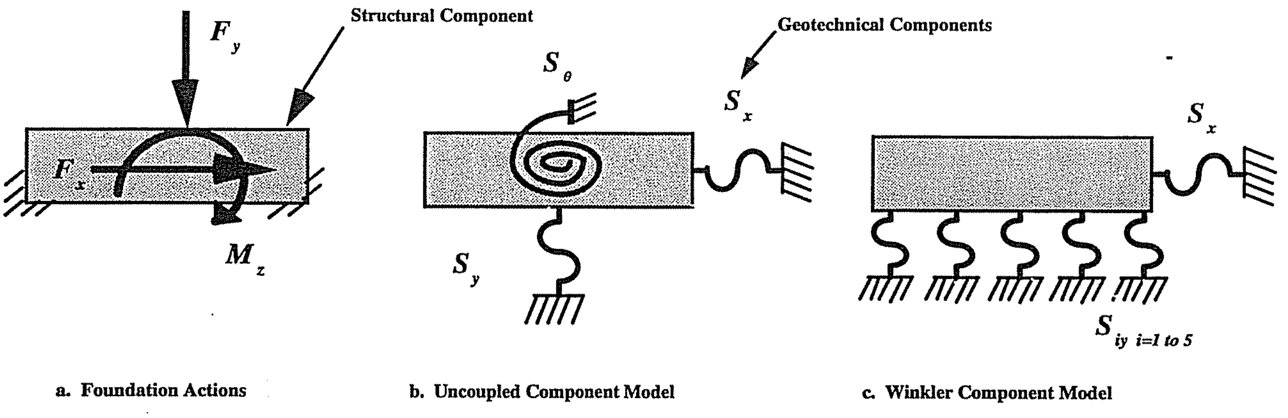


Figure 10-2. General Foundation Element Models

In some cases, grade beams, slabs, or basement walls provide rigid and strong horizontal linkage of foundation elements. This condition might allow the representation of the horizontal strength and stiffness of the foundation, K_h, at concentrated points, as in Models A and B. Model C reflects the situation when the lateral capacities and stiffnesses of the individual footing elements greatly exceed those of any linkage elements. This might be the case for thin, poorly reinforced slabs on grade, for example. In some cases, the horizontal foundation stiffness may be so large that full fixity may be assumed.

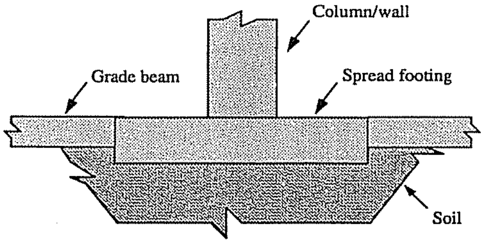
Column components of frames may not have sufficient strength to mobilize the rotational resistance of a supporting footing element. Model A represents this condition with a pin at the column base. The footing element provides only local vertical restraint but can work in combination with other, similar elements to form couples to resist global frame overturning. Models B and C illustrate the condition when the frame columns are fixed at the base. The restraint of the footing element comes from the separation of the soil components by a rigid element to form a local couple. In Model B, the rotational restraint is enhanced by the participation of the grade beams.

10.3 Foundation Elements

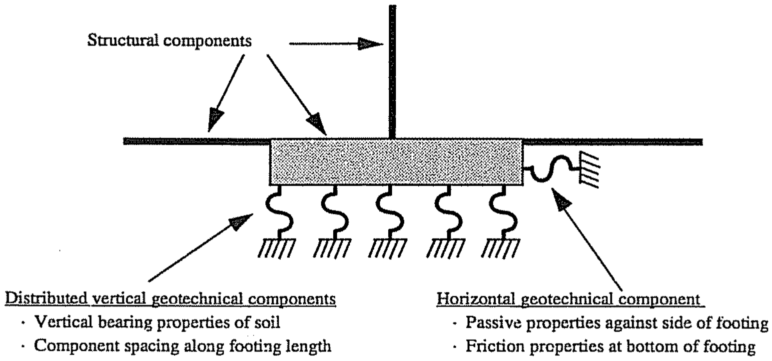
This section develops a generic model for typical foundation elements. Specific models used for analyses normally consist of structural and geotechnical components. Appropriate modeling depends upon the physical properties and configuration of the structural components and their interaction with the soil components. Chapter 9 contains modeling rules for the structural components.

The procedure is to model the nonlinear properties of the foundation element to reflect the possibility of soil yielding, sliding, or uplift, as well as inelastic structural behavior where appropriate. Section 10.4 provides guidance on the formulation of properties for geotechnical components. The combined component model represents the behavior of the foundation element.

The model of a foundation element represents its force-displacement behavior for the actions imposed upon it. This is illustrated in Figure 10-2. The element shown in Figure 10-2 (a) might be a spread footing or a pile group and cap. Vertical force, F_y, lateral force, F_x, and moment, M_z, act upon the element causing it to translate (Δ_x, Δ_y) and rotate (θ_z). An uncoupled, single node model of the element is shown in Figure 10-2 (b). The



a. Spread Footing Foundation



b. Element Model for Analysis

Figure 10-4. Basic Shallow Foundation Model

geotechnical components (S_x, S_y, S_θ) represent the stiffness and strength in each of the independent degrees of displacement freedom, as discussed in Section 10.4. The single-node representation is appropriate when the structural components are relatively rigid and do not interact significantly with the soil. Alternatively, the load-displacement behavior of a foundation element may be represented by a coupled Winkler component model, as shown in Figure 10-2 (c). The Winkler component models can capture more accurately the theoretical plastic capacity for interrelated actions. It is also appropriate when the structural components are relatively flexible and there may be significant interaction with soil material.

Figure 10-3 illustrates the theoretical inelastic interaction of vertical and rotational actions for a spread footing element beneath a shear wall (Bartlett 1976). The lateral action is normally uncoupled from the vertical and rotational action and is not included in Figure 10-3 for simplicity. The maximum rotational restraint and the nonlinear rotational stiffness are a function of the vertical load on the foundation element. The assumed theoretical elastic/plastic distribution of contact surface stress and its general relationship to the ultimate bearing capacity of the soil material are also illustrated.

10.3.1 Shallow Bearing Foundations

Rectangular isolated and continuous spread footings normally consist of flat plate and/or beam components bearing vertically directly against the underlying soil component to resist vertical, horizontal, and rotational loads (see Figure 10-4a).

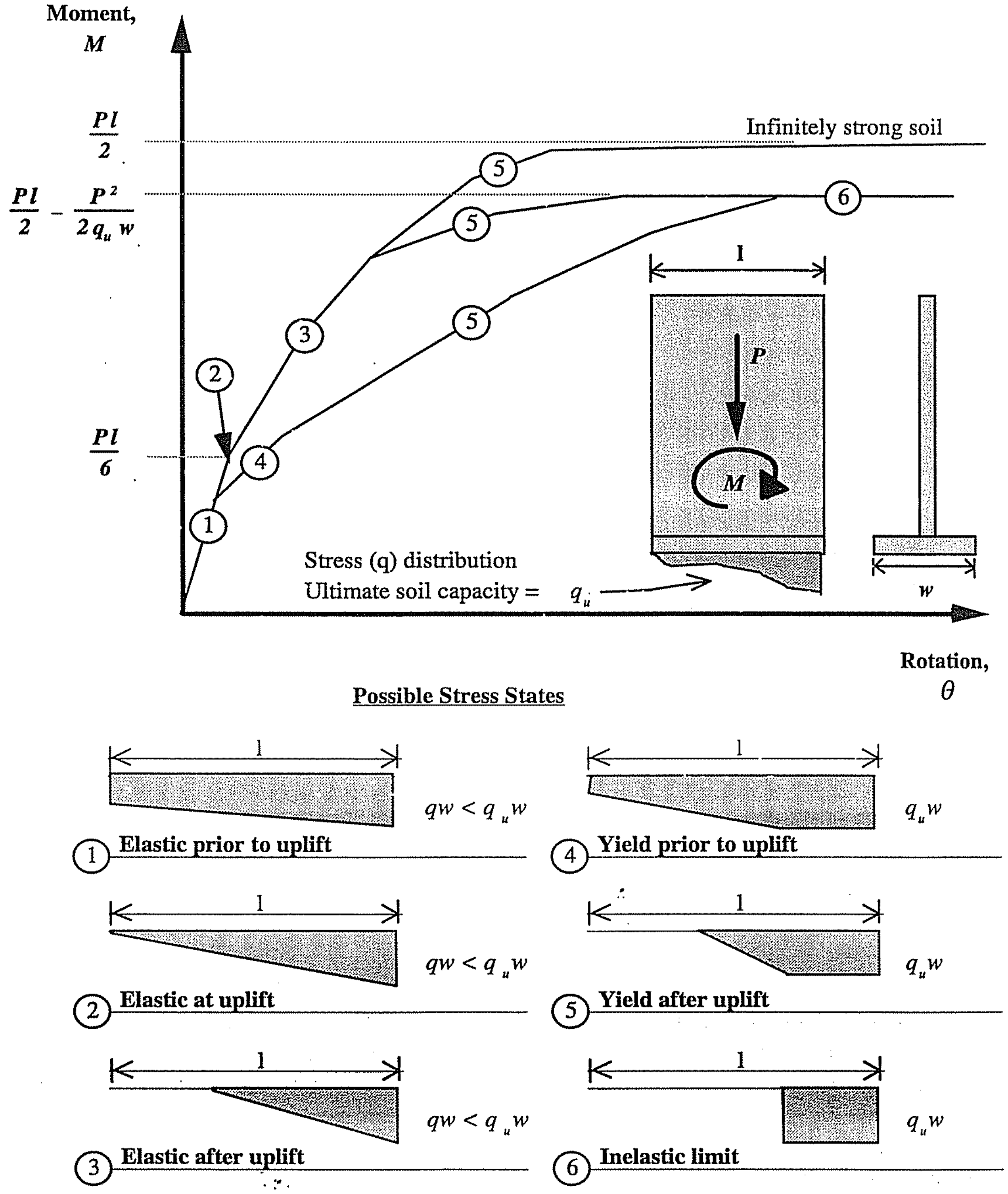
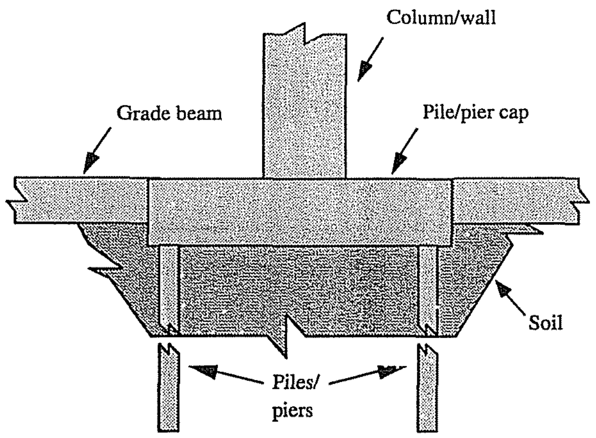
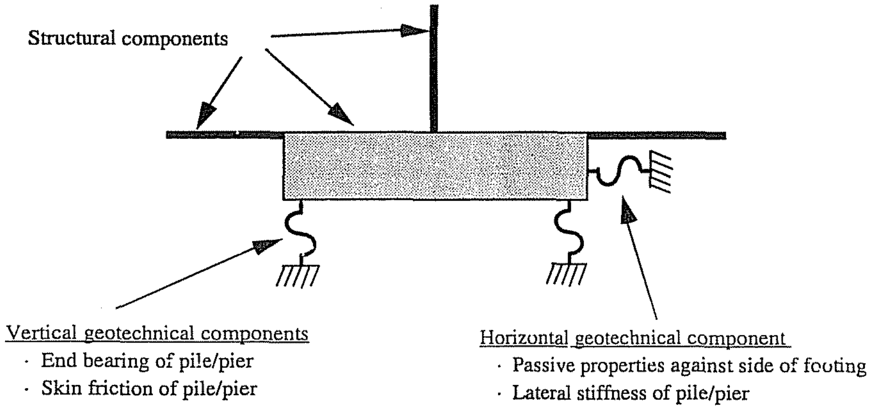


Figure 10-3. Theoretical Elastic-Plastic Foundation Behavior



a. Deep Pile/Pier Foundation



b. Element Model for Analysis

Figure 10-5. Basic Deep Foundation Model

Conventional structural finite-element components represent the beams, columns, and walls in the generalized model in Figure 10-4b. In some cases, particularly beneath shear walls, the structural footing component may be essentially rigid, compared to the supporting soil. The vertical and rotational resistance is the result of direct bearing on the supporting soil. Vertical geotechnical components represent both the stiffness and the strength of these actions. Grade beams might also have vertical geotechnical components beneath them. The smaller the spacing of the geotechnical components along the footing or grade beams, the greater the theoretical accuracy of the solution will be. Parametric studies of the relative stiffness of the structural and geotechnical components provide quick and simple insight in this regard. The inherent uncertainty of geotechnical material properties usually does not warrant excessive refinement.

Lateral resistance, represented by the horizontal geotechnical components, is the result of friction between the bottom and sides of the concrete and the contact surfaces of the soil, as

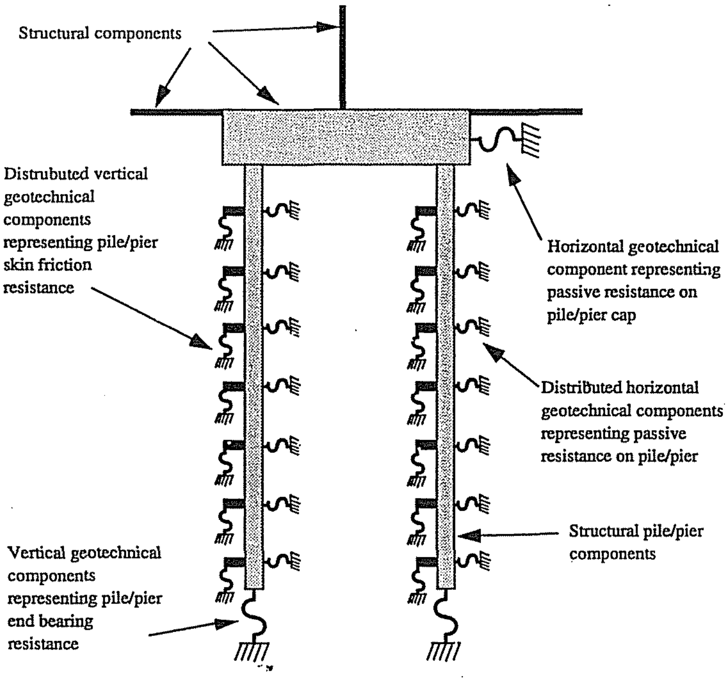


Figure 10-6. Refined Deep Foundation Model

well as any passive pressure against the sides of the footing in the direction of loading. The grade beam elements in Figure 10-4b might also be capable of resisting lateral load by friction and/or by transfer load to other elements axially.

10.3.2 Deep End Bearing and Friction Foundations

A basic model for deep foundation elements is shown in Figure 10-5. Deep foundation elements normally consist of flat plate and/or beam components supported upon driven piles of wood, concrete, or steel, or drilled cast-in-place concrete piers. The pile/pier components provide axial resistance through both skin friction and end bearing. Conventional structural finite-element components represent the beams, columns, and walls in the generalized model in Figure 10-5b. The vertical and rotational resistance of the foundation element is the result of axial resistance of the pile/piers, represented in the model by vertical geotechnical components. Grade beams might also have vertical geotechnical components beneath them representing bearing resistance of the supporting soil.

Lateral resistance, represented by the horizontal geotechnical components, is the result of friction between and passive pressure against the sides of the pile/pier cap in the direction of loading and the horizontal resistance of the piles. The grade beam elements in Figure 10-5b might also be capable of resisting lateral load through axial force transfer and/or friction.

An alternative, refined, deep foundation model is shown in Figure 10-6. In this representation the interaction of the pile/pier with the soil is modeled directly. This approach is appropriate when the stiffness of the pile/pier is relatively large. Piers greater than 2 feet in diameter might be an example. Often parametric studies can provide guidance on when such refinement is necessary. In most instances, refined substructure models may be used to determine horizontal component properties for pile groups for use in the basic model of Figure 10-5.

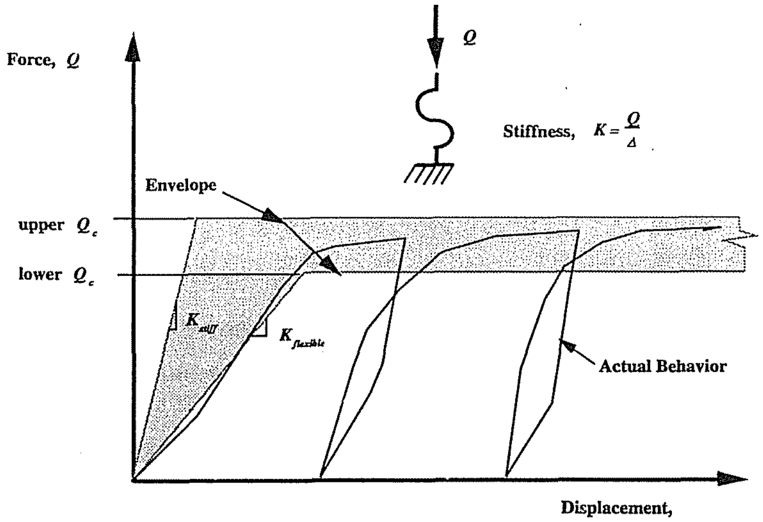


Figure 10-7. Basic Force-Displacement Envelope for Soils Components

10.4 Properties of Geotechnical Components

This section defines basic theoretical properties for geotechnical materials. These properties define the stiffness and strength behavior of geotechnical components for use in models of foundation elements. These properties are the basis for the generalized nonlinear force-displacement envelopes shown in Figure 10-7. The component shown in Figure 10-7 might represent a section of soil material beneath a shallow footing or the lateral resistance of a pile cap in a deep foundation model.

Upon initial loading, the example component may be relatively stiff until, for example, a preconsolidation pressure due to previous overburden or drying shrinkage might be reached. At this point the material may soften progressively until a capacity plateau is reached. If the footing is unloaded, the rebound is not usually complete, and permanent displacement occurs. For repeated cyclic loading, the permanent displacement can accumulate. When reloaded, the soil beneath the footing can be substantially stiffer than for previous cycles. This behavior is simplified and generalized for use in a structural analysis model using the strength and stiffness envelope shown in Figure 10-7.

This envelope allows the structural engineer to investigate the sensitivity of the analysis to the soils parameters. It may be that the stiff-strong assumption will give critical results for some structural elements while the flexible-weak will more adversely affect others. As a general rule of thumb, the initial range of the envelope should reflect a factor of 4 between minimum and maximum values. The procedure is to make a best estimate of component stiffness and strength, then divide and multiply by 2 to generate lower and upper bounds, respectively. The envelope might be wider for very sensitive solutions or highly uncertain geotechnical data. Conversely, if detailed geotechnical investigation results in reliable and accurate properties, then the uncertainty envelope might be narrowed.

The implied unidirectional relationship of Figure 10-7 reflects the characteristics of many geotechnical components. A spread footing, for example, cannot resist tension or uplift forces once dead loads are overcome by seismic overturning. One exception is deep pile/pier components which may resist tension by mobilizing skin friction during uplift. In this instance or other similar instances the force-displacement envelope extends below the x-axis into the tension zone. The

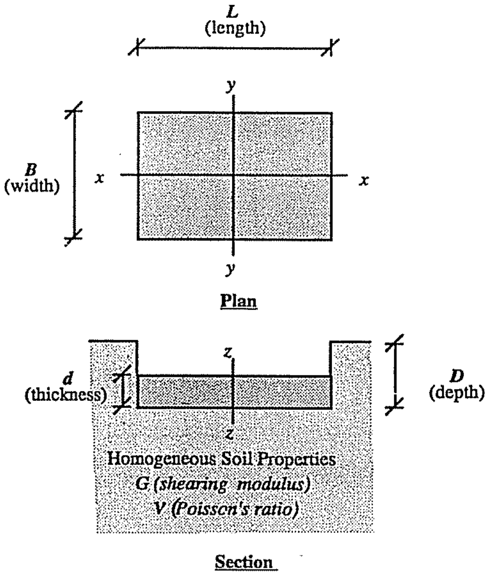


Figure 10-8. Properties of a Rigid Plate on a Semi-infinite Homogeneous Elastic Half-Space for Stiffness Calculations

envelope may not be symmetrical since, in the example of a deep pile/pier, end bearing stiffness and strength are not mobilized during uplift.

The elastic, perfectly plastic, generalized force-displacement relationship is the simplest to use with most structural analysis programs. If more-refined properties are available and the computational effort is tolerable, other, more sophisticated representations are certainly acceptable. As discussed previously, however, the inherent uncertainty of geotechnical material properties usually does not warrant excessive refinement.

Another property of the large majority of soil components is evident in the generalized relationship. Geotechnical components do not tend to degrade or shed load with large displacements. Most geotechnical components possess virtually unlimited inelastic displacement capacities. Exceptions include piles in rare, sensitive clays, liquefied soils, and steep hillside conditions. The implications of this characteristic are discussed further in Section 10.6.

10.4.1 Bearing Stiffness Parameters

10.4.1.1 Basic Steps

The basic steps for determining the stiffness properties of shallow bearing geotechnical components are as follows (see Figures 10-2b and 10-8):

1. Determine the uncoupled total surface stiffnesses, $K_{i'}$, of the foundation element by assuming it to be a rigid plate bearing at the surface of a semi-infinite elastic half-space (see Table 10-2).
2. Adjust the uncoupled total surface stiffnesses, $K_{i'}$, or the effects of the depth of bearing by multiplying by embedment factors (see Table 10-3), e_i, to generate the uncoupled total embedded stiffnesses, K_i.
3. Calculate individual distributed stiffness intensities, k_i, by dividing the uncoupled total embedded stiffnesses, K_i, by the corresponding area of contact or moment of inertia.

$$k_z = \frac{K_z}{L\,B}$$

$$k_y = \frac{K_y}{L\,d}$$

$$k_x = \frac{K_x}{B\,d}$$

$$k_{\theta x} = \frac{K_{\theta x}}{I_x}$$

$$k_{\theta y} = \frac{K_{\theta y}}{I_y}$$

4. Compare the vertical stiffness intensities k_z, $k_{\theta y}$, and $k_{\theta x}$. In general, these will not be equal. In a two-dimensional analysis, one of the rotational intensities normally will not be used. If the difference between the vertical translational stiffness intensity, k_z, and the vertical rotational stiffness intensity, k_θ, is small, then either, or a representative average, may be used in Step 6, below. If the difference is large, then one or the other may be used in Step 6 if the footing is acting primarily in either vertical translation or rotation. If the difference is large and the actions are highly coupled, then the approximate procedure in Step 5 may be used to refine the component stiffnesses.

Table 10-2. Surface Stiffnesses for a Rigid Plate on a Semi-infinite Homogeneous Elastic Half-Space (adapted from Gazetas 1991) [1]

Stiffness Parameter	*Rigid Plate Stiffness at Surface, Ki'*
Vertical Translation, K_Z'	$\frac{GL}{1-v}\left[0.73+1.54\left(\frac{B}{L}\right)^{0.75}\right]$
Horizontal Translation, K_Y' (toward long side)	$\frac{GL}{2-v}\left[2+2.5\left(\frac{B}{L}\right)^{0.85}\right]$
Horizontal Translation, K_X' (toward short side)	$\frac{GL}{2-v}\left[2+2.5\left(\frac{B}{L}\right)^{0.85}\right]-\frac{GL}{0.75-v}\left[0.1\left(1-\frac{B}{L}\right)\right]$
Rotation, $K_{\theta X}'$ (about x axis)	$\frac{G}{1-v}\,I_X^{0.75}\left(\frac{L}{B}\right)^{0.25}\left(2.4+0.5\frac{B}{L}\right)$
Rotation, $K_{\theta Y}'$ (about y axis)	$\frac{G}{1-v}\,I_Y^{0.75}\left[3\left(\frac{L}{B}\right)^{0.15}\right]$

1. See Figure 10-8 for definitions of terms

Table 10-3. Stiffness Embedment Factors for a Rigid Plate on a Semi-infinite Homogeneous Elastic Half-Space (adapted from Gazetas 1991) [1]

Stiffness Parameter	*Embedment Factors, ei*
Vertical Translation, e_Z	$\left[1+0.095\frac{D}{B}\left(1+1.3\frac{B}{L}\right)\right]\left[1+0.2\left(\frac{(2L+2B)}{LB}d\right)^{0.67}\right]$
Horizontal Translation, e_Y (toward long side)	$\left[1+0.15\left(\frac{2D}{B}\right)^{0.5}\right]\left\{1+0.52\left[\frac{\left(D-\frac{d}{2}\right)16\,(L+B)\,d}{BL^2}\right]^{0.4}\right\}$
Horizontal Translation, e_X (toward short side)	$\left[1+0.15\left(\frac{2D}{L}\right)^{0.5}\right]\left\{1+0.52\left[\frac{\left(D-\frac{d}{2}\right)16\,(L+B)\,d}{LB^2}\right]^{0.4}\right\}$
Rotation, $e_{\theta X}$ (about x axis)	$1+2.52\frac{d}{B}\left(1+\frac{2d}{B}\left(\frac{d}{D}\right)^{-0.20}\left(\frac{B}{L}\right)^{0.50}\right)$
Rotation, $e_{\theta Y}$ (about y axis)	$1+0.92\left(\frac{2d}{L}\right)^{0.60}\left(1.5+\left(\frac{2d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.60}\right)$

1 See Figure 10-8 for definitions of terms

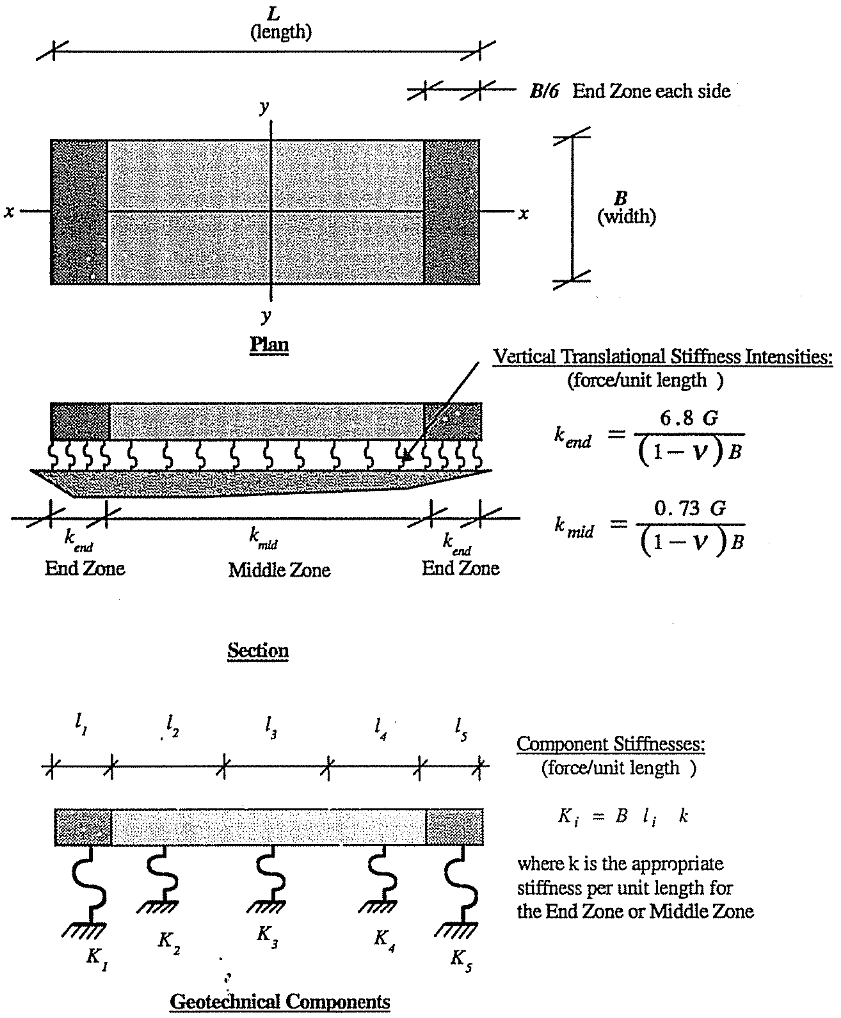


Figure 10-9. Winkler Component Model of Rectangular Spread Footing

5. Represent the ends of the rectangular footing with end zones of a length of approximately one-sixth of the footing width, as in Figure 10-9. Base the vertical stiffness intensity in the end zones on the vertical translational stiffness of a B x $B/6$ isolated plate. The vertical stiffness intensity in the middle zone is equivalent to that for vertical translation of an infinitely long strip footing ($L/B = \infty$).
6. Calculate individual geotechnical component stiffnesses by multiplying the appropriate stiffness intensity by the spacing of the components in each direction in the Winkler component model.

Table 10-4. Poisson's Ratio for Typical Soil Materials

Soil Description	*Approximate Value of Poisson's Ratio*, ν
Saturated clays and sands, beneath the water table	0.50
Nearly saturated clays, above the water table	0.40
Wet, silty sands (S_r = 50 to 90%)	0.35
Nearly dry sands, stiff clays, rock	0.25

10.4.1.2 Shear Modulus

The bearing stiffness of a vertically loaded plate on soil materials is a function of the dimensions of the plate, the depth of the bearing plane beneath the surface, and the properties of the soil materials. The shear modulus, G, for a soil is related to the modulus of elasticity, E, and Poisson's ratio, ν, by the relationship

$$G = \frac{E}{2\,(1+\nu)}$$

Poisson's ratio for typical soils is shown in Table 10-4.

The initial shear modulus, G_0 , is related to the shear wave velocity at low strains, v_s, and the mass density of the soil, ρ, by the relationship

$$G_0 = \rho\, v_s^{\,2}$$

Converting mass density to unit weight, γ, gives an alternative expression

$$G_0 = \frac{\gamma}{g}\, v_s^{\,2}$$

where g is acceleration due to gravity. Some typical values of G_0 for commonly encountered soils are tabulated in Section 10.5.

Most soils are intrinsically nonlinear, and the shear wave modulus decreases with increasing shear strain. The large strain shear wave velocity, v'_s, and the effective shear modulus, G, can be estimated on the basis of the anticipated maximum ground acceleration in accordance with Table 10-5, adapted from the NEHRP Provisions (BSSC, 1995). To reflect the upper- and lower-bound concept illustrated in Figure 10-7 in the absence of a detailed geotechnical site study, the upper-bound stiffness of rectangular footings should be based on twice the effective shear modulus, G, determined in accordance with the above procedure. The lower-bound stiffness should be based on one-half the effective shear modulus. Thus the range of stiffness should incorporate a factor of 4 from lower- to upper-bound.

Table 10-5. Effective Shear Moduli and Shear Wave Velocity as Determined by Shaking Intensity

	Seismic Shaking Intensity, ZEN ^{1,2}			
	0.10	*0.15*	*0.20*	*≥ 0.30*
Ratio of effective to initial shear modulus (G/G_0)	0.81	0.64	0.49	0.42
Ratio of effective to initial shear wave velocity (v'_s/v_s)	0.90	0.80	0.70	0.65

Notes:

1. Site specific values may be substituted if documented in a detailed geotechnical site investigation.
2. The value of E used to determine the product, ZEN, should be taken as equal to 0.5 for the Serviceability Earthquake, 1.0 for the Design Earthquake, and 1.25 for the Maximum Earthquake.
3. Linear interpolation may be used for intermediate values.

10.4.1.3 Flexible Structural Components

In some instances, the stiffness of the structural components of the footing may be relatively flexible compared with the soil material. A slender grade beam resting on stiff soil is an example. Classical solutions for beams on elastic supports can provide guidance on when such effects are important. For example, a grade beam supporting point loads spaced at a distance of L might be considered flexible if:

$$\frac{E\,I}{L^4} < 10\,k_{SV}\,B$$

where E is the effective modulus of elasticity, I is the moment of inertia, and B is the width of the grade beam. For most flexible foundation systems, the unit subgrade spring coefficient, k_{SV}, may be taken as:

$$k_{SV} = \frac{1.3\,G}{B(1\text{-}\nu)}$$

10.4.2 Bearing Capacity Parameters

10.4.2.1 Vertical Components

The classical general expression for ultimate vertical soil bearing stress capacity (Bowles 1982 and Scott 1981) of a shallow footing is:

$$q_{ult} = c\,N_c\,\zeta_c + \gamma D\,N_q\,\zeta_q + 1/2\,\gamma B\,N_\gamma\,\zeta_\gamma$$

In this expression:

c = cohesion property of the soil

N_c = cohesion bearing capacity factor depending on angle of internal friction, ϕ, for the soil (see Figure 10-10)

N_q = surcharge bearing capacity factor depending on angle of internal friction, ϕ, for the soil (see Figure 10-10)

N_γ = density bearing capacity factor depending on angle of internal friction, ϕ, for the soil (see Figure 10-10)

ζ_c, ζ_q, ζ_γ = footing shape factors (see Table 10-6)

γ = soil total unit weight

D = depth of footing

B = width of footing

An experienced geotechnical engineer normally should prepare these calculations on the basis of the results of specific field and laboratory tests for a given site. In the initial stages of a seismic evaluation, approximate values of foundation capacities may be used by the structural engineer to determine the sensitivity of the subject analysis to foundation effects and the need for refined capacities. Guidance on the selection of approximate capacities is offered in Section 10.5.

Table 10-6: Bearing Shape Factors

	Bearing Shape Factors		
Shape of Footing	**Cohesion** ζ_c	**Surcharge** ζ_q	**Density** ζ_γ
Strip	1.0	1.0	1.0
Rectangle	$1+\frac{B}{L}\frac{N_q}{N_c}$	$1+\frac{B}{L}\tan\phi$	$1-0.4\frac{B}{L}$
Circle or square	$1+\frac{N_q}{N_c}$	$1+\tan\phi$	0.6

It is important for the structural engineer to note that the maximum bearing stress capacity is dependent upon the width and bearing depth of an individual footing. Wider footings are capable of sustaining larger unit loads before deforming plastically. The overburden pressure on deeper footings enhances their ultimate capacity. Thus the procedure for determining the capacity of individual vertical geotechnical components includes consideration of the dimensions and depth of the entire footing element to estimate its total capacity. The total capacity can then be divided by the total area to convert to stress capacity. The individual capacities of geotechnical components are simply the product of the stress capacity times the spacing of the components in each direction.

10.4.2.2 Horizontal Components

The total horizontal capacity of foundation elements can be estimated from their stiffness and displacement as a percentage of their thickness, *d*, shown in Figure 10-8 (Martin and Yan 1995). The maximum lateral capacity can be estimated as that associated with a displacement of from 2 to 4% of

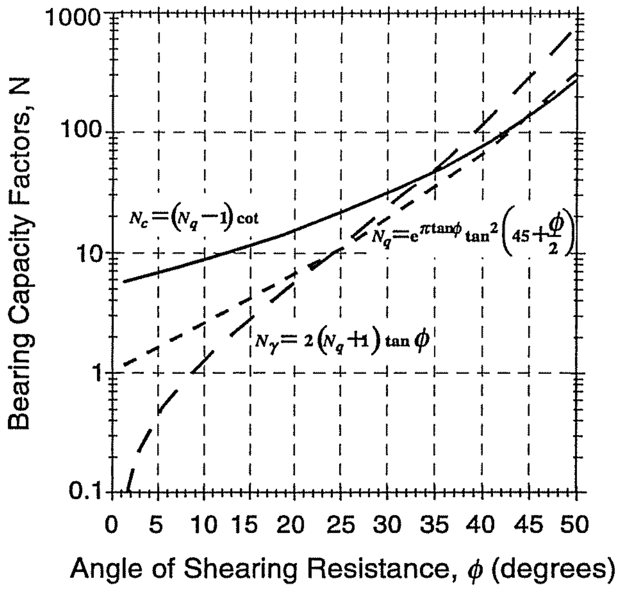


Figure 10-10. Bearing Capacity Factors

the thickness. The total capacity can be converted to stress capacity by dividing by the area of horizontal contact. Component capacity is the product of the stress capacity and the spacing of the components in each direction. The recent California Department of Transportation criteria for bridges gives further guidance on lateral characteristics of footings and pile caps (ATC 1996b).

10.4.3 Deep Component Stiffness Parameters

10.4.3.1 Vertical Components

Axially loaded piles or piers transfer loads to the soil through a combination of end bearing and skin friction (see Figure 10-11). Their true axial stiffness is a complex nonlinear interaction of the structural properties of the pile/pier and the load displacement behavior of the soil for friction and end bearing (Martin and Lam 1995). The stiffness at the top of the pile/pier can be expressed as:

$$K_z = \frac{Q_{total}}{\Delta_z}$$

In most cases, the tip displacement is relatively small. This can be understood by considering two extreme cases. If the pile/pier is purely end bearing, the tip bearing stiffness, by definition, must be relatively large compared with the friction stiffness of the soil and the axial stiffness properties of the pile/pier. If the tip displacement is assumed to be zero, the resulting axial stiffness of the pile/pier is:

$$K_z = \frac{(EA)_{pile}}{L}$$

At the other extreme, a purely friction pile/pier implies that the force at the tip is zero. This is also consistent with the assumption of a very small tip displacement. For zero tip displacement and a uniform total transfer to the soil by skin friction, the axial stiffness of the pile/pier approximates:

$$K_z = \frac{2(EA)_{pile}}{L}$$

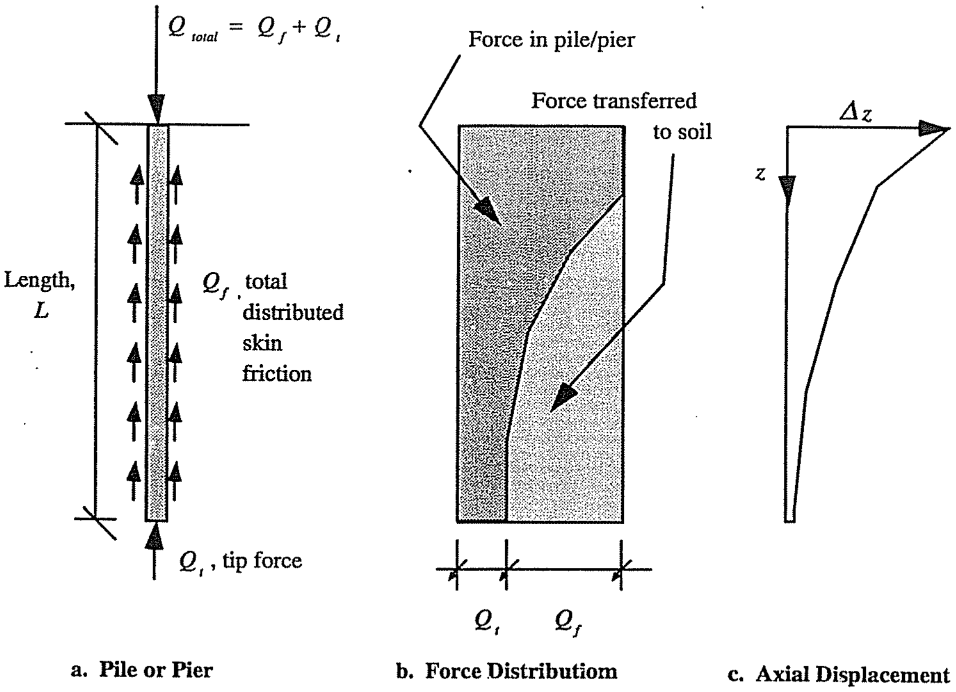


Figure 10-11. Forces and Displacements of Axially Loaded Pile or Pier

Allowing for some tip displacement and recognizing the inherent complexity of the problem, a reasonable initial range of stiffness for a vertical pile/pier component is:

$$\frac{0.5(EA)_{pile}}{L} < K_z < \frac{2(EA)_{pile}}{L}$$

This range of stiffness is consistent with the force-displacement envelope recommended for geotechnical components. If the effect of pile stiffness variation is significant for structural behavior, refined geotechnical analysis can justify a more narrow range.

10.4.3.2 Horizontal Components

In most circumstances, the horizontal stiffness and strength of a pile/pier group is controlled by the passive pressure and frictional resistance at the cap (see Section 10.3.2). These horizontal components would have stiffness parameters consistent with those developed in Section 10.4.1. For deep foundation elements with large-diameter piers (> 24 inches), the more refined model of Figure 10-6 can be developed to evaluate the effects of pier/soil interaction on horizontal stiffness. Research, including computer programs, by Reese et al. (1994) and others provides guidance on modeling parameters, as do California bridge criteria (ATC 1996b). Parametric studies of typical deep foundation elements with the refined model can be used to develop approximate equivalent stiffnesses for simplified horizontal components to restrain the cap without including refined models of each element in the full structural analysis.

10.4.4 Deep Component Capacity Parameters

As shown in Figure 10-11, the downward capacity of a pile/pier is the sum of the capacity of the tip in end bearing and of the total friction along the shaft. There are three general types of pile/pier behavior pertinent to vertical load capacity:

1. **Predominantly End Bearing:** Some pile/piers are installed primarily to advance through poor material to underlying bearing strata. For example, piles might be driven through saturated weak clay into firm alluvium. Although the pile might penetrate a depth of a few diameters into the alluvium, it is predominantly point bearing and its capacity is largely dependent on the bearing capacity of the material at its tip.
2. **Intermediate:** Pile/piers installed into granular or well-graded soils generally rely on both friction and end bearing resistance. Friction normally increases with depth, primarily reflecting increases in overburden pressure. Tests indicate that the frictional resistance of these types of piles typically represents more than half the total capacity for downward load. For uplift, however, the frictional resistance is less than for downward load.
3. **Predominantly Friction:** Pile/piers installed in fine-grained silts and clays often rely primarily on skin friction for vertical resistance. Although the tip offers some resistance, the strength and stiffness of these materials in bearing is usually small compared with the friction. Frictional resistance to uplift in silts and clays is nearly the same as for downward loads.

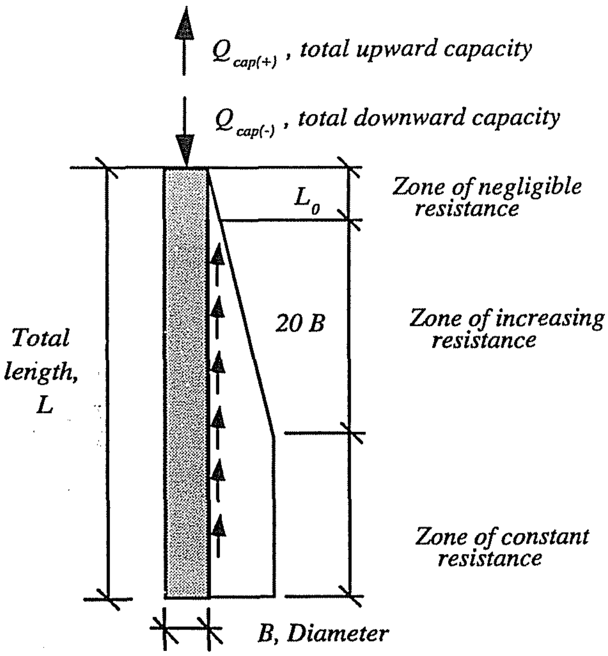
The capacities of pile/piers are difficult to determine even in the best of circumstances. In the absence of specific geotechnical data, the procedures illustrated in Figures 10-12 and 10-13 may be used, for granular or cohesive materials respectively, to calculate preliminary estimates of capacity. Ranges of typical values of parameters for use with these procedures are presented in Section 10.5. These all have been adapted from NAVFAC (1986). For friction resistance in granular materials the top 3 to 5 diameters of pile/pier length are often neglected. The upward frictional capacity of pile/piers in cohesive materials is assumed to be equal to the downward frictional capacity, neglecting end bearing, using the procedures in Figure 10-13. This is acceptable using the relatively conservative NAVFAC values for adhesion strength; but the necessity for reduction of upward capacity should be a consideration in any project specific geotechnical investigation.

10.5 Characterization of Site Soils

The scope of required investigation of geotechnical and foundation conditions depends upon the nature of the deficiencies of the building, the desired performance goal for the building, and the selected strategy for the rehabilitation.

The nature and extent of foundation and geotechnical information required for the seismic evaluation and retrofit of concrete buildings depends upon several considerations on both sides of the capacity vs. performance equation. As discussed in Chapter 4, the properties of site soils affect the intensity of shaking that can be expected at the building site. In addition to the shaking hazard, the potential for ground displacement also may present a need for geotechnical data. The scope of investigations for these hazards should be coordinated with that discussed here for structural modeling properties.

The foregoing sections of this chapter address the force-displacement characteristics of foundation elements and geotechnical components to resist vertical, rotational, and horizontal actions induced by ground shaking. These are principally strength and stiffness properties utilized in the global modeling of structural behavior. Some generic ranges of values for these parameters for typical soils are presented in Tables 10-7 and 10-8. This level of information can provide a starting point for the structural engineer to begin to determine the overall sensitivity of the building to foundation effects during earthquakes. On the basis of preliminary analyses, the structural engineer might conclude that it is unnecessary to refine the



Soil Profile

Thickness	Wt. Density	Angle of Shearing Resistance
L_0	γ_0	ϕ_0
L_1	γ_1	ϕ_1
L_3	γ_3	ϕ_3
etc.	etc.	etc.
etc.	etc.	etc.
L_t	γ_t	ϕ_t

Downward Capacity $$Q_{cap\,(-)} = P_t\, N_q\, A_t + \sum_{i=1}^{t-1} F_{di}\, P_i \tan \delta_i\, a_s\, L_i$$

Where P_t = Effective vert. stress at tip

$$P_t = \sum_{i=0}^{t-1} L_i\, \gamma_i \le P \text{ @ } L_0 + 20B$$

N_q = Bearing capacity factor (see Table 10-8a)
A_t = Bearing area at tip
F_{di} = Effective horiz. stress factor for downward load (see Table 10-8b)
P_i = Effective vert. stress at depth i

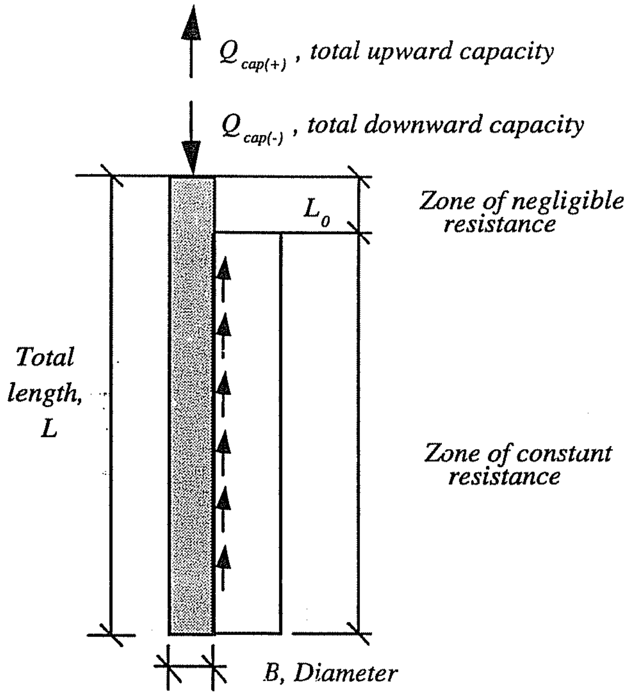
$$P_i = \sum_{j=0}^{i} L_j\, \gamma_j \le P \text{ @ } L_0 + 20B$$

δ_i = Friction angle between pile/pier and soil at depth i (see Table 10-8c)
a_s = Surface area of pile/pier per unit length

Upward Capacity $$Q_{cap\,(+)} = \sum_{i=1}^{t-1} F_{ui}\, P_i \tan \delta_i\, a_s\, L_i$$

Where F_{ui} = Effective horiz. stress factor for upward load (see Table 10-8b)
other parameters as for downward capacity

Figure 10-12. Pile/Pier Capacities for Granular Soils (adapted from NAVFAC, 1986)



Soil Profile

Thickness	Wt. Density	Cohesion
L_0	γ_0	c_0
L_1	γ_1	c_1
L_3	γ_3	c_3
etc.	etc.	etc.
etc.	etc.	etc.
L_t	γ_t	c_t

Downward Capacity $$Q_{cap\,(-)} = c_t\, N_c\, A_t + \sum_{i=1}^{t-1} c_{ai}\, a_s\, L_i$$

Where c_t = Cohesion strength of soil (see Table 10-8d) at tip
N_c = Bearing capacity factor 9.0 for depths greater than 4B
A_t = Bearing area at tip
c_{ai} = Cohesion strength of soil (see Table 10-8d) at depth i
a_s = Surface area of pile/pier per unit length

Upward Capacity $$Q_{cap\,(+)} = \sum_{i=1}^{t-1} c_{ai}\, a_s\, L_i$$

Where parameters are as for downward capacity

Figure 10-13. Pile/Pier Capacities for Cohesive Soils (adapted from NAVFAC, 1986)

Table 10-7. Typical Shallow Bearing Soil Material Properties

			Average Properties in Zone of Influence								
Soil Profile[1] Type	Description	Typ. Unified Classif. of Soil Material	Shear Wave Velocity V_S (ft/sec)	SPT N (blows/ft)	Undrained Shear Strength, s, (psf)	Weight Density γ, (pcf)	Angle of Shearing Resistance φ (degrees)	Range of Initial Shear[3] Modulus, G_0 (psf x 10^6)		Range of Maximum Ultimate Bearing Stress q_{ult} (psf)[3,4]	
								Low	High	Low	High
S_A	Hard Rock		> 5000					120		20,000	50,000+
S_B	Rock		2500 to 5000			140+		25	120	15,000	40,000
S_C	Dense Soil - Soft Rock	GW,GP	1200 to 2500	> 50	> 2000	120 - 140	> 40	5	25	8,000	32,000
S_D	Stiff Soil	SW,SP,SM , SC, GM,GC	600 to 1200	15 to 50	1000 to 2000	100 - 130	33 - 40	1	5	5,000	20,000
S_E	Soft Soil	CL,ML,MH ,CH	< 600	< 15	< 1000	90 - 120	< 33		< 1	2,000	15,000
S_F	Special Study	OL,OH, PT									

1. Soil Profile Type in Zone of Influence may differ from that used to determine ground shaking parameters.
2. Zone of Influence extends below a shallow footing to approximately three times its width.
3. Ranges of values are provided for use in initial parametric studies. Site-specific geotechnical investigations are recommended for any structure sensitive to foundation effects.
4. Maximum capacities assume bearing at a minimum depth of 1 foot below adjacent grade and a minimum width of footing of 3 feet.

Table 10-8a. Typical Pile/Pier Capacity Parameters: Bearing Capacity Factors, N_q (adapted from NAVFAC, 1986)

Placement	Angle of Shearing Resistance for Soil, φ (degrees)												
	26	28	30	31	32	33	34	35	36	37	38	39	40+
Driven Pile	10	15	21	24	29	35	42	50	62	77	86	120	145
Drilled Pier	5	8	10	12	14	17	21	25	30	38	43	60	72

Table 10-8b. Typical Pile/Pier Capacity Parameters: Effective Horizontal Stress Factors, F_{di} and F_{ui} (adapted from NAVFAC, 1986)

Pile/Pier Type	Downward F_{di}		Upward F_{ui}	
	low	high	low	high
Driven H-pile	0.5	1.0	0.3	0.5
Drive straight prismatic pile	1.0	1.5	0.6	1.0
Drive tapered pile	1.5	2.0	1.0	1.3
Driven jetted pile	0.4	0.9	0.3	0.6
Drilled pier	0.7		0.4	

Table 10-8c. Typical Pile/Pier Capacity Parameters: Friction Angle, δ (degrees) (adapted from NAVFAC, 1986)

Pile/pier Material	δ
Steel	20
Concrete	$0.75\ \phi$
Timber	$0.75\ \phi$

Table 10-8d. Typical Pile/Pier Capacity Parameters: Cohesion, c_t and Adhesion, c_a (psf) (adapted from NAVFAC, 1986)

Pile Material	Consistency of Soil	Cohesion, c_t		Adhesion, c_a	
		low	high	low	high
Timber and Concrete	Very soft	0	250	0	250
	Soft	250	500	250	480
	Med. stiff	500	1000	480	750
	Stiff	1000	2000	750	950
	Very Stiff	2000	4000	950	1300
Steel	Very soft	0	250	0	250
	Soft	250	500	250	460
	Med. stiff	500	1000	460	700
	Stiff	1000	2000	700	720
	Very Stiff	2000	4000	720	750

foundation model properties for some buildings. In other cases, the performance might be highly sensitive to soil properties. This is particularly true for performance objectives beyond Life Safety, where permanent base displacements are important.

On the other hand, the structural and geotechnical engineer should carefully consider the results of the preliminary analyses to assess both the sensitivity of the results to soils properties and the probable outcome of further investigations. The extent of site investigations for foundation and geotechnical data is largely a benefit and cost issue. If detailed information is likely to confirm beneficial results, then the cost may be worthwhile. For this reason, a progressive approach to the investigations is appropriate. This is summarized in Table 10-9.

10.5.1 Initial Investigation for Minimum Information

The primary objective of the first stage of geotechnical investigation for a building evaluation is to gather enough information to generate a basic model of the foundation system for analysis. This includes classification of site soil materials to allow selection of initial analysis properties in conjunction with Tables 10-7, 10-8a, 10-8b, 10-8c and 10-8d. These tables use the NEHRP (BSSC 1996) soil profile classifications as examples of material types. In some cases, conditions differ from these typical profiles and site specific data is required. Potential initial resources for documenting existing geotechnical conditions and material properties include the following:

- Review of geotechnical reports, drawings, test results, and other available documents related directly to the building or to nearby sites
- Review of regional or local reports related to geologic and seismic hazards and subsurface conditions
- Visual inspection of the structure and its foundation

Site-specific information may be obtained from geotechnical reports and foundation drawings. Relevant site information that would be obtained from geotechnical reports includes logs of borings and/or cone penetrometer tests and laboratory tests to determine shear strengths of the subsurface materials, and engineering assessments that may have been conducted addressing geologic hazards, such as faulting, liquefaction, and landsliding. If geotechnical reports are not available for the subject facility, geotechnical reports for adjacent buildings may provide a basis for assignment of initial properties. Regional geotechnical and geological reports and other studies often provide general information on subsurface soil materials, depth of the groundwater table, seismicity, and site hazards.

Information contained on existing building drawings should be thoroughly reviewed for relevant foundation data. These data include the type, size, and location of all footings and footing design loads. Information on the size of the foundation element, the locations of the bases of the footings or the tips of the piles, and the pile cap elevations may be necessary to formulate a model of the foundation system. Other relevant information includes material composition (i.e., wood, steel, or concrete piles) and pile installation methods (open- and closed- end piles that are driven or jetted). The design drawings may also indicate information regarding the allowable bearing capacity of the foundation elements. This information can be used directly to estimate foundation capacity. Typically, allowable loads for spread footings are approximately one-third to one-half of the ultimate capacity (factor of safety = 2 to 3). Allowable design loads for piles and piers typically are about one-half to two-thirds of the ultimate capacity (factor of safety = 1.5 to 2).

Table 10-9. Progressive Scope of Geotechnical Site Investigation

Stage	Relative Cost of Investigation	Sources	Objectives	Sensitivity of Building to Foundation Effects	Performance Objectives
1. Initial (minimum)	Inexpensive	Existing reports Drawings Construction records Regional maps Existing load data Visual inspections	Soil profile selection (S_A, S_B . . .) Type and condition of existing footings Soils classification Generic stiffness and capacity (see Tables 10.7 & 10.8)	Low	Modest Structural Stability, Life Safety (LS) for simple, foundation insensitive, buildings
2. Supplemental (as needed)	Inexpensive to moderate	Shallow test pits Standard penetration test (SPT blow counts) Torvane shear tests	Moisture contents Approximate densities Cohesion Angle of shearing resistance	Moderate	Average LS for buildings with some sensitivity to foundation effects Damage control for simple insensitive buildings
3. Advanced (as needed)	Moderate to expensive	Borings Cone penetrometer tests(CPT/SCPT) Undisturbed samples Laboratory tests Theoretical analyses In situ load tests	Refined strength and stiffness	High	Enhanced LS for very foundation sensitive buildings Damage control for moderate to sensitive buildings

Records may also be available indicating ultimate pile capacities if load tests were performed. Construction records can be important, since footing characteristics are often changed in the field by the geotechnical engineer on the basis of actual subsurface conditions. Finally, information on the existing loads on the structure is needed to determine the amount of existing dead load on foundation elements to be included in the analysis.

In addition to reviewing existing documents, the structural engineer, with the possible assistance of a geotechnical counterpart, should conduct a visual site reconnaissance to observe the performance of the site and building foundation to gather information for several purposes. First, the reconnaissance would confirm that the actual site conditions agree with information obtained from the building drawings. Variances from the building drawings should be noted and considered in the evaluation. Such variances would include building additions or foundation modifications that are not shown on the existing documentation. The second purpose of the site reconnaissance is to document off-site development that may have a potential impact on the building. Such off-site development could include building grading activities that may impose loads or reduce the level of lateral support to the structure under consideration. The final aspect of the visual site reconnaissance is to document the performance of the existing building and the adjacent area to record signs of poor foundation performance, such as settlement of floor slabs, foundations, or sidewalks. These indicators may suggest structural distress that could affect performance during a future earthquake.

10.5.2 Supplemental Information

The existing site data and information gained from the initial geotechnical investigation may need to be supplemented by additional site explorations and geotechnical analysis. The structural engineer, in consultation with a geotechnical engineer, should formulate a scope of further investigation after preliminary structural analysis to assess the sensitivity of the building performance to foundation effects. The purpose of the supplemental investigation normally would be to refine the estimates of the stiffness and strength of geotechnical components. Supplemental data may also be necessary for sites where there is a significant potential for liquefaction, lateral spreading, or landsliding.

It is sometimes possible to excavate relatively shallow test pits at the exterior or interior of the building to examine typical footings and soils materials. The geotechnical engineer can do some simple field tests to get a better idea of the soil properties, particularly for spread footing foundations. Fairly inexpensive laboratory tests of samples taken from the site can provide additional useful information on moisture content, density, and other basic properties. On the basis of this information, the geotechnical engineer can make a judgment as to a refined range of stiffness and strength parameters for analysis. Upon this advice, the structural engineer can decide to utilize the refined range of properties for further analysis or to proceed with more-detailed geotechnical investigation.

10.5.3 Advanced Investigations

The purpose of advanced geotechnical investigations is to reduce the range of geotechnical component properties to the minimum necessary for an acceptable level of confidence in the behavior of the existing and/or retrofitted building (see Section 10.6). The scope of advanced investigations depends on the specific uncertainties regarding the performance of the building and their implications for potential costs. On the basis of knowledge gained from analysis, the structural engineer estimates the degree of confidence required for an effective decision on performance. This implies a definable range of stiffness and strength parameters for geotechnical components. The geotechnical engineer advises the structural engineer as to the type of investigation necessary. The geotechnical engineer might also speculate on the likelihood of specific results for certain critical parameters. This speculation is important in some cases to avoid investigation costs that are unlikely to support assumptions necessary for compensating construction cost savings.

By definition, the advanced investigations are relatively expensive. There is, however, a fairly broad range of costs within this general category. Exploration borings, cone penetrometer tests, and conventional laboratory tests are generally at the less expensive end of the spectrum. Theoretical analyses using linear and nonlinear finite-element techniques can be utilized to generate refined estimates of the force-displacement characteristics of foundation elements. The associated costs range from minor to significant, depending on the complexity of the problem. In situ, large-scale load tests on real or prototype foundation elements provide a high degree of reliability. The costs of such tests must be considered carefully in light of potential savings.

10.6 Response Limits and Acceptability Criteria

The recommendation for a force-displacement envelope for geotechnical components in Section 10.4 recognizes the inherent uncertainty in the properties of soil materials. These uncertainties are generally greater than those associated with structural materials when it comes to strength and stiffness. In contrast with geotechnical components, the greatest uncertainty with structural components often is in their ductility and post-elastic behavior characteristics. As illustrated in Figure 10-7, most individual geotechnical components have very large ductilities. For the Life Safety or Structural Stability performance objectives, individual geotechnical components are acceptable regardless of their maximum displacements. This, of course, does not mean that the effects of these displacements on the structure are acceptable. These effects are reflected in the global structural model automatically if the foundation elements are included. For performance objectives beyond Life Safety for damage control, the effects of permanent displacements of geotechnical components warrant consideration.

10.6.1 Foundation Influence on Structural Response

The results of preliminary analyses using the force-displacement envelopes of Section 10.4 indicate the sensitivity of a building to foundation effects. Generally two analyses are required—one assuming stiff/strong geotechnical components and the other assuming flexible/weak. If the results of each analysis indicate that performance objectives are met for both assumptions, then further refinement of geotechnical properties is unnecessary. Similarly, if both assumptions lead to the conclusion that retrofitting is required for essentially the same deficiencies, then it is normally sufficient to correct the deficiencies without reducing the uncertainty in geotechnical parameters. If the two assumptions lead to different conclusions, then their specific results need to be carefully compared.

Flexible/weak foundation assumptions tend to lead to solutions that indicate lower forces and larger displacements in structures when compared with stiff/strong assumptions. This may lead to a more favorable prediction of performance overall if, for example, the lower forces are in shear critical members and the other elements can sustain the larger displacements. The reliability of the more favorable prediction of performance may be highly dependent on the geotechnical parameters. If this is the case, then investigations should be considered to refine these properties. Alternatively, the more conservative prediction of performance can be assumed and addressed accordingly by retrofit measures.

10.6.2 Permanent Foundation Displacements

Accurate predictions of permanent foundation displacements due to earthquakes are not possible. For performance objectives beyond Life Safety, nonetheless, these permanent displacements and their influence throughout the building may be significant. As a general-order-of-magnitude estimate of permanent foundation displacements, it seems reasonable to focus on the portion of the total foundation movement attributable to inelastic effects. These displacements are those that occur after the component capacity has been reached. By definition they are the maximum component displacements less the elastic displacements. It is probably overly conservative to assume that these are the permanent displacements in all cases, since load reversals would probably recover some of the inelastic movement. However, depending on specific foundation characteristics and the duration of intense shaking, the ratcheting effect illustrated in Figure 10-7 can cause an accumulation on inelastic displacement. For this reason, the inelastic portion of foundation displacement and its pro rata effect on the building should be assumed to be permanent.

10.7 Modifications to Foundation Systems

Foundation modifications might be required because of inadequate capacity of existing foundations to resist the effects of seismic shaking. New structural improvements, such as new shear walls or the strengthening of existing shear walls, might also require foundation modifications or additions. Improvements might also address site hazards other than shaking, such as liquefaction or landslides. These objectives might be accomplished by one or a combination of the basic construction techniques as summarized in Table 10-10.

10.7.1 Soil Removal and Replacement

Soil removal and replacement can increase strength and stiffness locally or over large areas. The technique is to excavate material and then backfill with compacted suitable material. Cement or other stabilizing agents can improve the effectiveness of the fill material. This approach is not effective in most circumstances to correct deficiencies beneath existing buildings, since extensive shoring is normally required for excavation. In some instances, local areas supporting existing or new foundation elements can be improved. For example, removal and recompaction of material adjacent to spread footings or pile/pier caps can improve lateral resistance.

Removal and replacement is often an alternative for global site stabilization. Transverse buttresses can improve resistance to landsliding or block sliding induced by liquefaction. Often improved drainage is installed in conjunction with soil removal and replacement.

10.7.2 In Situ Soil Densification

Densification to improve stiffness and strength of soils can sometimes be accomplished without removal. Impact and vibration are often effective, especially for granular materials. The direct effect of vibration and the settlement resulting from volume change pose a problem near existing buildings, however. Different grouting techniques can densify some soils in a large- or small-scale application. Compaction grouting consists of the injection under pressure of a mixture of soil, cement, and water into a soil mass. The grout forms a bulb that displaces and densifies the soil. Care must be taken near existing improvements because of the potential for heaving and lateral displacement. Chemical, or permeation, grouting replaces the pore water in soil materials to improve stability. Jet grouting is a technique that uses high-velocity jets to inject and mix a stabilizing agent into a soil mass. Field testing usually is required for in situ grouting to verify the quality and extent of the process. Also preconstruction testing to determine the effectiveness of the proposed method is common.

10.7.3 Underpinning

Underpinning is a common solution to increase the stiffness and strength of existing foundation elements. The objective of underpinning is to transfer the foundation loads to a deeper, more competent bearing zone. This can be accomplished in a number of different ways. Pits can be excavated beneath existing footings and then filled with concrete. This is done segmentally to minimize temporary shoring. Piles, or more commonly drilled piers, can be installed adjacent to and tied to existing footings or pile caps. Drilling equipment for large-diameter piers often cannot access the interior of existing buildings. Micropiles ranging in size from 3 to 8 inches in diameter can be installed with portable equipment. Depending on the specific soil conditions, these small-diameter shafts can carry fairly large loads (100 to 200 kips).

Underpinning can be designed to resist both compression and tension loads. The design and construction of underpinning should also address effective load transfer. In some cases, preloading using jacks ensures that loads are transferred to the underpinning without excessive displacement. Often it is necessary only to install nonshrink grout between the existing footing and the new elements.

Table 10-10. Techniques for and Objectives of Foundation Modifications

Construction Technique	General Objective of Modifications		
	Increased Stiffness/Strength of Existing Elements	Support for New Structural Elements	Global Site Stabilization
Soil Removal and Replacement	Can be effective locally; large scale use usually impractical		Common alternative
In Situ Soil Densification			
Impact and Vibration	Normally impractical near existing improvements		Common alternative
Grouting	Can be effective locally and on a larger scale; most effective for granular materials		Common alternative
Underpinning			
Conventional block	Common alternative	Not applicable	Not applicable
Piles or Piers	Piers more common because of vibration	Not applicable	Not applicable
Structural Additions or Modifications			
Slabs	Can be effective in mobilizing more elements for horizontal resistance		Not applicable
Grade Beams/Walls	Often used to spread loads to increased number of elements		Can be effective for horizontal stabilization
Spread Footings	Existing components can be enlarged	New components can be installed	Not applicable
Piers/Piles	Components can be added to existing groups/elements; large diameters diffcult in interior	New elements can be installed	Can be used for vertical and horizontal stbilization
Base Isolation	Requires evaluation of benefits/costs; more applicable for damage control objectives		Not applicable

10.7.4 Structural Additions or Modifications

Foundation improvements can include the addition of new elements to support new shear walls or columns for the structural retrofitting and modification of existing foundation elements.

Where the potential for differential lateral displacement of building foundations exists, the provision of interconnecting grade beams or a well-reinforced slab on grade can provide effective mitigation of these effects. Grade beams and walls can also augment the rotational resistance of the foundation system by spreading overturning forces to a larger number of resisting elements.

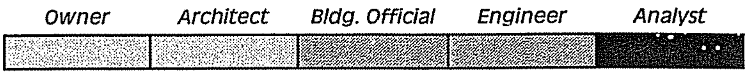
Driven piles made of steel, concrete, or wood or cast-in-place concrete piers may be used to supplement the vertical and lateral capacities of existing foundation elements or to support new elements. Similarly, existing spread footings may be enlarged or new footings added to improve foundation performance. Capacities and stiffnesses of new or supplemental foundation elements and components should be determined in accordance with the appropriate procedures of Section 10.4. The potential for differential strengths and stiffnesses among individual elements and components should be included. In some cases, jacking and shoring can be incorporated in the construction of new foundation elements to mitigate these effects.

10.7.5 Base Isolation

Base isolation and energy dissipation systems are retrofit strategies that have been used for a number of existing buildings. Conceptually, base isolation is an attempt to reduce the response of the building by decoupling it from the ground. Since this decoupling occurs at the base, it may be viewed as a foundation modification. But the inability of existing systems to meet performance objectives need not be confined to, or necessarily even include, the foundation in order for base isolation to be considered as a strategy for a given building. The practicality of the use of this technology for existing buildings is highly dependent on the characteristics of the specific building. Basements or crawl spaces facilitate the installation of isolation systems. Because of the costs of typical systems, the benefits of base isolation normally need to include significantly large reductions in damage potential and expected loss of revenue.

Chapter 11

Response Limits



Audience Interest Spectrum

11.1 General

To determine whether a building meets a specified performance objective, response quantities from a nonlinear static analysis (Chapter 8) are compared with limits for appropriate performance levels (Chapter 3). This chapter presents those structural response limits, which constitute acceptance criteria for the building structure. The response limits fall into two categories:

- Global building acceptability limits. These response limits include requirements for the vertical load capacity, lateral load resistance, and lateral drift. They are given in Section 11.3.
- Element and component acceptability limits. Each element (frame, wall, diaphragm, or foundation) must be checked to determine if its components respond within acceptable limits. Section 11.4 describes the minimum checks that must be made for each building element.

As described in Chapter 3, a performance objective represents a desired performance level for a specified earthquake ground motion. If calculated response for the specified ground motion exceeds any of the global building or element and component acceptability limits given in this chapter for the appropriate performance level, then the building should be deemed to not achieve the performance objective.

In addition to the numerical response limits of Sections 11.3 and 11.4, this chapter also presents descriptive limits of expected performance in Section 11.2. These descriptive limits are intended to guide selection of appropriate performance levels, and are not intended to be used as strict acceptance criteria. See Section 9.6 for notation.

Create a model with Chapters 9 and 10.

Analyze it with Chapter 8.

Compare results with limits in Chapter 11.

Where acceptance criteria are not met, it is necessary either to redefine the performance objectives or to retrofit the building. Chapter 6 describes a variety of retrofit strategies.

Commentary: *The response limits in this chapter relate only to the building structure. Nonstructural response limits are not included in the discussion, except to the extent that global building drift limits will influence nonstructural element acceptability.*

11.2 Descriptive Limits of Expected Performance

Performance levels given in Chapter 3 are broad, general categories used to set performance objectives. For the structural elements and components commonly found in the building types addressed by this methodology–cast-in-place concrete nonductile frame and wall frame structures–a brief description of anticipated damage at four structural performance levels is outlined in Table 11-1 (Table 11-1 also includes descriptions of nonstructural damage expected in typical buildings). The Damage Control performance level represents a range of states bounded by Life Safety and Immediate Occupancy; in Table 11-1, it is distinguished by a level of nonstructural damage higher than that permitted for Immediate Occupancy. Although the descriptions focus on existing building elements, similar descriptions could be appended for buildings with added retrofit elements.

> *All performance levels require vertical and lateral stability. Global drift limits may also apply.*

Damage descriptions in Table 11-1 contain two types of information: first, the absolute statements that the gravity load capacity of the building remains substantially intact, and second, the qualitative/quantitative statements regarding the magnitude of observed damage to structural and nonstructural components. Qualitative descriptions are based on damage observed in prior earthquakes; they describe the borderline between building collapse and maintenance of the gravity load system. Accurate quantitative descriptions, perhaps more useful to engineers and building officials, are difficult to construct; they typically reflect damage observed in prior earthquakes as well as reports on similar structural systems tested to varying degrees of destruction.

11.3 Global Building Acceptability Limits

11.3.1 Gravity Loads

The gravity load capacity of the building structure must remain intact for acceptable performance at any level. Where an element or component loses capacity to support gravity loads, the structure must be capable of redistributing its load to other elements or components of the existing or retrofit system.

Commentary: *Older reinforced concrete construction often contains proportions and details that make it prone to collapse in strong earthquake ground motions Loss of gravity load carrying capacity in columns, beam-column joints, or slab-column connections has been the primary cause of collapse in past earthquakes. The analysis must evaluate the consequences of loss of vertical load carrying capacity in any structure components.*

11.3.2 Lateral Loads

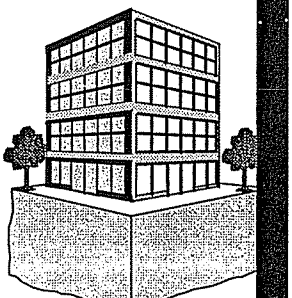
As discussed in Chapter 9, some component types are subject to degrading over multiple load cycles. If a significant number of components degrades, the overall lateral force resistance of the building may be affected. The lateral load resistance of the building system, including resistance to the effects of gravity loads acting through lateral displacements, should not degrade by more than 20 percent of the maximum resistance of the structure. Where greater degradation occurs, either the structure should be retrofit to reduce the degradation, or alternative methodologies should be employed to refine the estimates of expected response. Degrading of secondary elements (Section 11.4.2.1) need not be considered for this check.

Table 11-1. Representative Damage Descriptions for Elements and Components in Nonductile Concrete Frame and Frame-Wall Buildings

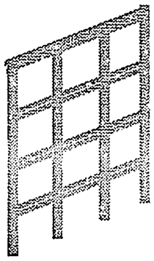
Element/ Component	Immediate Occupancy	Damage Control	Life Safety	Structrual Stability
Columns	Very limited flexural and shear cracking with no spalling. No permanent horizontal offset. Gravity capacity maintained.	Limited flexural and shear cracking with little or no spalling. No permanent horizontal offset. Gravity capacity maintained.	Hinges have formed in the lower portions of the building, causing spalling above and below beam-column joints. Permanent horizontal offset approaching 2.0% interstory drift with small areas marginally higher. Gravity capacity maintained.	Hinges have formed in the lower portions of the building causing significant spalling above and below beam-column joints and pulverizing of concrete within the core. Permanent horizontal offset approaching 3.5% interstory drift with small areas marginally higher. Gravity capacity maintained throughout nearly all of the structure.
Beams	Very limited spalling around beam column joint. Very limited flexural cracking in hinge region. No permanent deflection. Gravity capacity maintained.	Limited spalling around beam column joint. Limited flexural cracking in hinge region. No permanent deflection. Gravity capacity maintained.	Spalling around hinge region and beam column joint. Flexural and shear cracking in hinge region progressing into the beam column joint. Elongation of shear stirrups adjacent to joint. Permanent vertical deflection approaching *L*/175. Gravity capacity maintained.	Extensive spalling around hinge region and beam column joint. Extensive flexural and shear cracking in hinge region, progressing into beam column joint. Rupture of shear stirrups, permanent vertical deflection approaching *L*/75. Gravity capacity maintained.
Slabs	Very limited cracking adjacent to beam-column joint or other supports. Gravity capacity maintained.	Limited cracking adjacent to beam-column joint or other supports. Gravity capacity maintained.	Cracking adjacent to beam-column joint or other supports. Gravity capacity maintained.	Extensive cracking adjacent to beam-column joint or other supports. Chunks of concrete pulverized and missing from areas between slab steel. Vertical offsets in slabs adjacent to supports amounting to one-fourth the slab thickness but no collapse.
Walls and Pilasters (Piers)	Very minor shear cracking in plane of wall. Very little or no cracking at end of wall or of pilasters. No permanent horizontal offset. Gravity capacity maintained.	Minor shear cracking in plane of wall. Little or no cracking at end of wall or of pilasters. No permanent horizontal offset. Gravity capacity maintained.	Extensive spalling and shear and flexural cracking, particularly at the ends and heels of shear walls. Evidence of sliding shear failures. Permanent horizontal offset approaching 2.0% interstory drift with other areas marginally higher. Gravity capacity maintained.	Extensive spalling and shear and flexural cracking throughout wall, particularly in areas with greatest permanent offset Evidence of longitudinal rebar buckling. Evidence of sliding shear failures along construction joints and at base of wall. Permanent horizontal offset approaching 3.5% interstory drift with other areas marginally higher. Gravity capacity maintained throughout nearly all of structure.
Foundations	No evidence of differential settlement between two adjacent columns.	No evidence of differential settlement between two adjacent columns.	Differential settlement approaching *L*/150 between two adjacent columns.	Differential settlement approaching *L*/60 between two adjacent columns.
Nonstructural Elements	No exterior glass crushed. All exterior and interior doors operative. Very limited damage to suspended ceilings and light fixtures and no collapse. Very few furniture items overturned. Very limited cracking of interior partitions and stairwell finishes. Elevators and building utilities operative. Very limited damage to penthouses.	Very limited exterior glass crushed. All exterior and interior doors operative. Limited damage to suspended ceilings and light fixtures. Isolated furniture overturned. Limited cracking of interior partitions and stairwell finishes. Elevators and building utilities operative. Limited damage to penthouses.	Some glass crushed and limited portions missing on floors with greatest permanent offset. Most exterior doors operational. Some interior doors racked and inoperative. Some suspended ceilings collapsed. Light fixtures damaged. Interior partitions extensively cracked with limited toppling. Stairwell interiors extensively cracked. Building furniture overturned. Elevators and building utilities inoperative. Penthouses extensively damaged.	Most exterior glass crushed and missing at floors with greatest permanent displacement and smaller amounts of glass crushed on other floors. Exterior and interior doors racked and inoperative. Most suspended ceilings and light fixtures collapsed. Interior partitions extensively cracked and partially toppled. Stairwell interiors extensively cracked. Furniture overturned. Elevators and building utilities inoperative. Penthouses partially collapsed.

Checking for Acceptable Performance

For a given Performance Objective, calculated response quantities at the performance point must not exceed appropriate response limits associated with:



Global Building Response
- Gravity load support
- Lateral load stability
- Drift limits



Element and Component Response
Check all components for:
- Strength
- Deformation capacity

Commentary: Two effects can lead to loss of lateral load resistance with increasing displacement. The first is gravity loads acting through lateral displacements, known as the P-Δ effect. The P-Δ effect is most prominent for flexible structures with little redundancy and low lateral load strength relative to the structure weight. The second effect is degradation in resistance of individual components of the structure under the action of reversed deformation cycles (see Sections 9.5.1 and 9.5.4.1). When lateral load resistance of the building degrades with increasing displacement, there is a tendency for displacements to accumulate in one direction. This tendency is especially important for long-duration events.

The nonlinear static procedures presented in this methodology have not been adequately verified for structures whose resistance degrades substantially. For this reason, and to preclude the potential failure to identify inadequate performance of degrading structures, the methodology does not permit more than 20% strength degradation. Where resistance degrades by more than 20 percent of the maximum resistance, either nonlinear dynamic analysis methods should be used to assess earthquake demands (although software limitations may be prohibitive), or, preferably, the structure should be retrofitted. By definition, secondary elements are not essential to lateral load resistance, so their degradation is not a particular concern to global building performance.

11.3.3 Lateral Deformations

Lateral deformations at the performance point displacement (see Chapter 8) are to be checked against the deformation limits of this section.

Table 11-2 presents deformation limits for various performance levels. Maximum total drift is defined as the interstory drift at the performance point displacement. Maximum inelastic drift is defined as the portion of the maximum total drift beyond the effective yield point. For Structural Stability,the maximum total drift in story i at the performance point should not

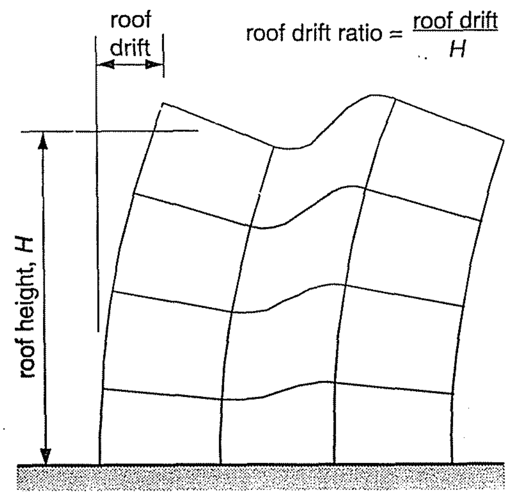
Table 11-2. Deformation Limits

	Performance Level			
Interstory Drift Limit	*Immediate Occupancy*	*Damage Control*	*Life Safety*	*Structural Stability*
Maximum total drift	0.01	0.01 - 0.02	0.02	$0.33\frac{V_i}{P_i}$
Maximum inelastic drift	0.005	0.005 - 0.015	no limit	no limit

exceed the quantity $0.33\frac{V_i}{P_i}$, where V_i is the total calculated lateral shear force in story i and P_i is the total gravity load (i.e. dead plus likely live load) at story i.

Commentary: *The maximum drifts in Table 11-2 are based on judgment of the project team. The following considerations were used to establish the recommended limits.*

For Immediate Occupancy, the maximum total drift limit of 0.01 is based on observed damage in laboratory tests on well-detailed frame structures. The maximum inelastic drift limit is based on the objective of avoiding significant residual deformations after the earthquake. Lower drifts may be required to avoid damage in other types of structures; component deformation limits in Section 11.4 are available for this purpose. Table 11-2 limits consider the building structure only. Additional limits may be required to protect nonstructural components and contents.



Roof Drift and Roof Drift Ratio

For Life Safety, the maximum total drift limit of 0.02 is recommended because significant experience with responses to larger deformation levels is lacking. Laboratory tests on relatively complete structural systems seldom extend beyond this deformation level. Furthermore, most tests have been conducted on structures satisfying or nearly satisfying current proportioning and detailing requirements for new buildings. Measured responses of buildings subjected to actual earthquakes also do not extend beyond this limit.

For Structural Stability, the maximum total drift limit is derived from similar limits in the NEHRP provisions for design of new buildings (BSSC, 1995). This limit is not very restrictive. For example, for a structure with a base shear coefficient of 15 percent of the building weight, the maximum total drift limit is 0.33 x 0.15 = 0.05. Many engineers would find this level of drift unacceptably high, especially for an older building with questionable details. Lower limits may be appropriate in many cases.

11.4 Element and Component Acceptability Limits

11.4.1 General

Each element must be checked to determine whether its individual components satisfy acceptability requirements under performance point forces and deformations.

Section 11.4.2 defines primary and secondary components and presents general information on strength and deformability checks. Sections 11.4.3 through 11.4.7 present specific recommendations for various elements and the components that typically compose those elements.

Commentary: *Together with the global requirements of Section 11.3, acceptability limits for individual components are the main criteria for assessing the calculated building response. Section 11.4 defines those components and actions that must be checked and recommends specific limits. As described in Chapter 9, elements (frames, walls, diaphragms, and foundations) are* *the main load resisting parts of the building and are composed of components (such as beams, columns, slabs, chords, and connections).*

All structural elements and components should be assessed for combined vertical and lateral forces and deformations. Components should be classified as primary or secondary (see Section 11.4.2.1) and checked against the requirements of Section 11.4.2 and Tables 11-3 through 11-10. Secondary components that do not carry gravity loads need not be checked against specific deformation limits. Refer to Section 11.4.2.3 commentary for additional discussion.

Elements that carry significant lateral load at the performance point are considered primary. Other secondary components are permitted greater inelastic deformations.

11.4.2 General Approach for Component Acceptability

11.4.2.1 Primary and Secondary Elements and Components

Each element and component is classified as primary or secondary depending on its significance to the lateral load resisting system at or near the performance point. Elements and components that provide a significant portion of the structure's strength or lateral stiffness at the performance point are considered primary. Other elements and components may be considered secondary.

Commentary: *Primary and secondary components are defined in terms of significance to building performance, not in terms of stiffness or strength and not in terms of the need to model them (see Chapter 9). Therefore, participation in the lateral load resisting system is measured at the performance point as a means of distinguishing those components that are able/needed to resist lateral load after several cycles of earthquake ground motion; these are considered primary. Some components may carry lateral load and still be considered secondary; examples include weak ductile components that yield early and carry little load relative to the performance point base shear and stiff components that degrade and shed their loads long before the performance point is reached. Flexible gravity load resisting elements in an otherwise stiff building are generally secondary. "Significant portion" both requires and allows for engineering judgment.*

11.4.2.2 Component Strength

Strength demands at the performance point are not permitted to exceed the strengths established in Chapter 9. For ductile, deformation-controlled actions (see Section 9.5.4.1), inelastic response is acceptable as long as deformation acceptance limits are not exceeded. Furthermore, analysis results should be checked to ensure that strain hardening of the analytical model does not result in unrealistic internal actions that exceed the component's expected strength. For brittle, force-controlled actions, components should be modeled with degrading resistance once the strength is reached.

Commentary: *Section 9.5.1 commentary discusses techniques for modeling degradation. Some applications are illustrated in Volume 2, Example Building Studies.*

11.4.2.3 Component Deformation Capacity

Calculated component deformations are not permitted to exceed deformation limits for appropriate performance levels.

Where test data or analyses are used directly to develop deformation acceptance criteria, their use should be consistent with the descriptions in

Table 11-1. A multilinear load-deformation relation should be established using procedures described in Section 9.5 (see Figures 9-10 and 9-11). The deformation acceptability criteria should be established as follows:

- For primary actions, components, and elements: The component deformation capacity at the Structural Stability performance level is defined as the deformation at which significant lateral load strength degradation begins. The component deformation capacity at the Life Safety performance level is defined as 75 percent of the Structural Stability deformation.
- For secondary actions, components, and elements: The component deformation capacity at the Structural Stability performance level is defined as the deformation at which vertical load carrying capacity is lost. The component deformation capacity at the Life Safety performance level is defined as 75 percent of the Structural Stability deformation.

Commentary: *Defining the component deformation capacities at the Life Safety performance level as 75 percent of the Structural Stability deformation is largely judgmental and arbitrary. It is believed to provide a reasonable margin of safety.*

As an alternative to calculating deformation capacities or deriving them from test data, the numerical limits in Tables 11-3 through 11-10 (at end of this chapter) may be used. To use these tables, the components must have been modeled according to the procedures in Chapter 9, with modeling parameters according to Tables 9-6 through 9-12. Sections 11.4.3 through 11.4.7 provide additional information on application of tabulated criteria.

Commentary: *Deformation acceptance criteria may be calculated or derived directly from test data. Figure 11-1 illustrates the deformation acceptance criteria when this approach is used. In this figure a relatively simple load-deformation relation is shown. Point C (to which primary actions, components, and elements are linked) corresponds to the deformation at which significant strength degradation begins. Point E (to which secondary actions, components, and elements are linked) corresponds to the deformation at which gravity load resistance is lost.*

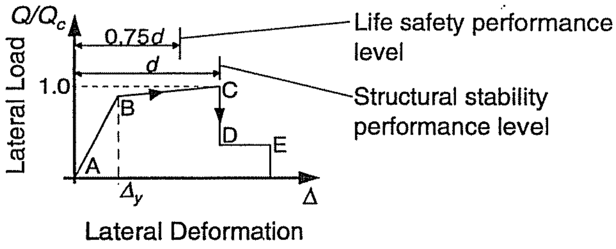
Secondary components that do not carry gravity loads need not be checked against specific deformation limits because they do not lead to collapse (assuming excessive deformation does not make them falling hazards themselves). However, engineering judgment must still be used to assess performance relative to general expectations and requirements for desired performance levels. For example, short, force-controlled coupling beams that reach their shear capacity long before the performance point are likely to be classified as secondary components carrying no gravity load. After degrading, they can probably undergo excessive deformations without affecting the behavior or capacity of adjacent components, so specific deformation limits may not apply. Still, if the performance objective involves an Immediate Occupancy performance level, the associated cracking and spalling may be unacceptable. A similar example involves hollow clay tile floor-to-floor partitions in a frame building. As long as they do not affect exits or corridors, these nonstructural components can be heavily damaged without affecting structural performance. Their assessment will be based on repair and inconvenience costs. This is a matter of engineering judgment; specific deformation limits probably do not apply.

11.4.3 Concrete Frame Acceptability

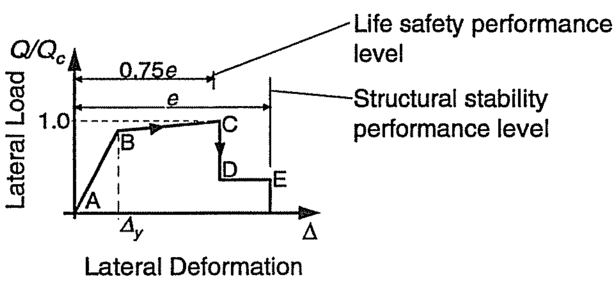
11.4.3.1 Beam-Column Frames

Acceptability should consider the strength and deformation capacity of beams, columns, beam-column joints, and other components, considering also connections with adjacent elements. The assessment should consider at least the following:

- Axial-flexural response of beams and columns, including evaluation of likely yield mechanisms



(a) Primary Actions, Components and Elements



(b) Secondary Actions, Components and Elements

Figure 11-1. Typical Load-Deformation Acceptance Criteria

and calculation of inelastic strength and rotation demands

- Shear response of beams and columns, at component ends and at locations along the span where reinforcement changes
- Actions on beam-column joints considering equilibrium of forces from beams and columns framing into the joint
- Adequacy of development lengths, splice lengths, and embedments of longitudinal and transverse reinforcement in beams, columns, and beam-column joints
- Potential for loss of gravity load capacity due to failure of beams, columns, or beam-column joints

Inelastic response is restricted to those components and actions listed in Tables 11-3 through 11-5, except where it is demonstrated by analysis or tests that other inelastic action can be tolerated considering the selected performance levels. In Table 11-3 for beams and Table 11-4 for columns, acceptance criteria are expressed in terms of plastic rotation angles within the yielding plastic hinge. In Table 11-5, the acceptance criteria are expressed in terms of total shear angle of the joint.

Commentary: Primary causes of collapse in beam-column frames are failures in columns and in joints. The evaluation should be especially attentive to flexural, shear, and splice actions in columns with nonconforming transverse reinforcement, and shear actions in joints with minimal transverse reinforcement.

The numerical acceptance criteria in Tables 11-3 through 11-5 were established from data identified in Chapter 9. The criteria are in terms of deformation quantities (plastic hinge rotations in beams and columns and total shear deformations in joints) recommended for these components in Chapter 9.

11.4.3.2 Slab-Column Frames

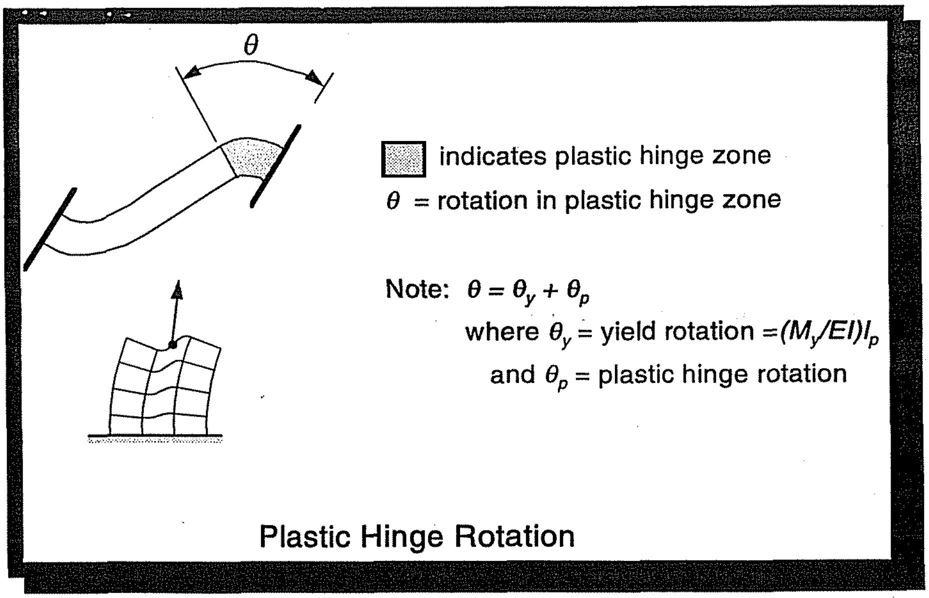
Acceptability should consider the strength and deformation capacity of slabs, columns, slab-column connections, and other components, considering also connections with adjacent elements. The assessment should consider at least the following:

- Flexural response of slabs and columns, including evaluation of likely yield mechanisms and calculation of inelastic strength and rotation demands
- Shear and moment transfer capacity of connections
- Adequacy of development lengths, splice lengths, and embedments of longitudinal and transverse reinforcement in slabs, columns, and slab-column joints
- Potential for loss of gravity load capacity due to failure of slabs, columns, or slab-column connections

Inelastic response should be restricted to those components and actions listed in Tables 11-4 and 11-6, except where it is demonstrated by analysis or tests that other inelastic action can be tolerated considering the selected performance levels. In Table 11-6 acceptance criteria are expressed in terms of plastic rotation angles within the yielding plastic hinge of the slab; alternatively, they refer to the plastic rotation of the connection element if one is used.

***Commentary:** Primary causes of collapse in slab-column frames are failures in columns and slab-column connections. The evaluation should be especially attentive to flexural, shear, and splice actions in columns with nonconforming transverse reinforcement, and shear and moment transfer actions in slab-column connections with heavy gravity load shears ($V_g/V_o > 0.4$). The concern for slab-column connections applies primarily where the connections do not have effectively continuous bottom reinforcement sufficient to catch the slab in the event that punching shear failure occurs. In other cases, punching shear failure is likely to be localized and unlikely to lead to progressive collapse.*

The numerical acceptance criteria in Table 11-6 were established from data identified in Chapter 9. The criteria are in terms of deformation quantities (plastic hinge rotations in slabs and plastic connection rotations in connections) recommended for these components in Chapter 9.



Plastic Hinge Rotation

11.4.4 Concrete Shear Wall Acceptability

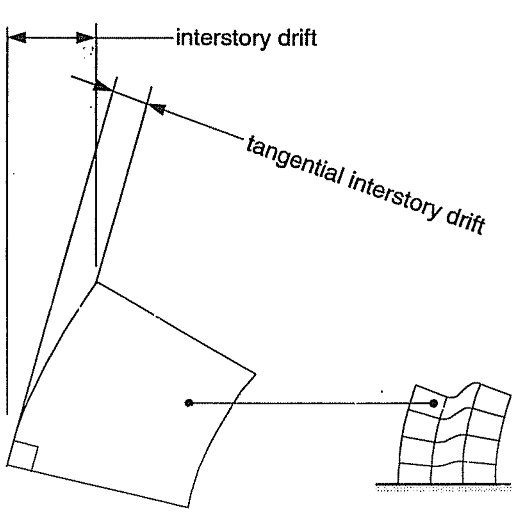
11.4.4.1 Solid Walls

Acceptability should consider the strength and deformation capacity of the wall and its connections with adjacent elements. The assessment should consider at least the following:

- Flexural deformation capacity
- Shear strength of the panel
- Sliding shear strength
- Foundation uplift

- Adequacy of development lengths and splice lengths of longitudinal and transverse reinforcement

Inelastic response should be restricted to those components and actions listed in Tables 11-7 through 11-9, except where it is demonstrated by analysis or tests that other inelastic action can be tolerated considering the selected performance levels. In Table 11-7, acceptance criteria are expressed in terms of plastic hinge rotation angles within the yielding plastic hinge. In Table 11-8, acceptance criteria are expressed in terms of tangential shear drift of the wall or wall segment. In Table 11-9, acceptance criteria are expressed in terms of total slip along the construction joint.



Interstory Drift Versus Tangential Interstory Drift

***Commentary:** The numerical acceptance criteria in Tables 11-7 through 11-9 were established from data identified in Chapter 9. The criteria are in terms of deformation quantities (plastic hinge rotations in walls, tangential drift ratio in walls or wall segments, and slip along the construction joint) recommended for these components in Chapter 9. Refer to Section 9.4.3.1 commentary for a discussion of construction joint slip.*

Tangential drift is the drift due to actual shear and flexural distortion of the wall, not including drift due to rigid body rotations (which may be caused by foundation yield or hinging at the wall base). Tangential drift ratio, in radians, is calculated as the tangential drift divided by the wall or story height.

11.4.4.2 Coupled, Perforated, or Punched Walls

Acceptability should consider the strength and deformation capacity of walls, wall segments, coupling beams, and pier-spandrel connections, considering also connections with adjacent elements. The assessment should consider at least the following:

- Flexural deformation capacity of walls, coupling beams, piers, and spandrels
- Shear strength of the wall panel, coupling beams, piers, spandrels, and connections
- Sliding shear strength at construction joints, intersections of walls and coupling beams, and interfaces between wall segments
- Foundation uplift
- Adequacy of development lengths and splice lengths of longitudinal and transverse reinforcement

Inelastic response should be restricted to those components and actions listed in Tables 11-7 through 11-10, except where it is demonstrated by analysis or tests that other inelastic action can be tolerated considering the selected performance levels. In Table 11-7, acceptance criteria are expressed in terms of plastic hinge rotation angles within the yielding plastic hinge. In Table 11-8, acceptance criteria are expressed in terms of tangential shear drift of the wall or wall segment. In Table 11-9, acceptance criteria are expressed in terms of total slip along the construction joint. In Table 11-10, acceptance criteria are expressed in terms of total chord rotation for the coupling beam.

***Commentary:** The numerical acceptance criteria in Tables 11-7 through 11-10 were established from data identified in Chapter 9. Note that no tests on perforated walls and only very few data on walls with irregular openings were available. Instead, tabulated limits are based on judgment and on results from tests of other wall element types.*

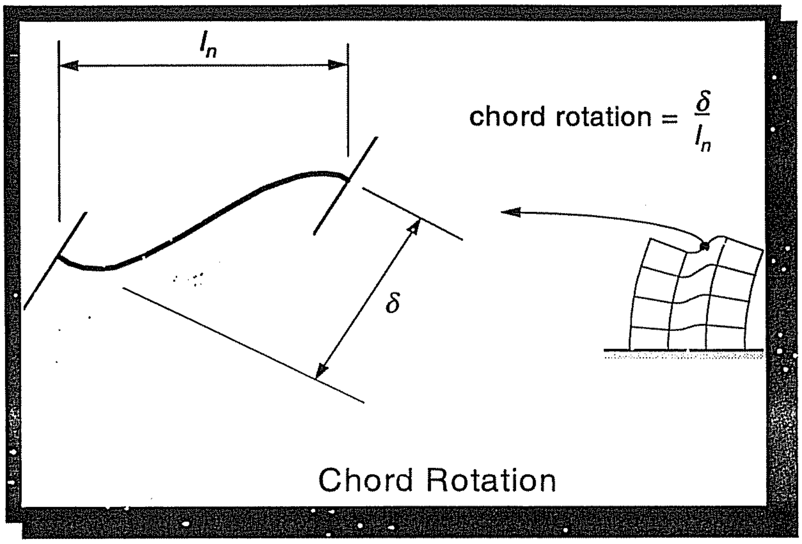
The criteria are in terms of deformation quantities (plastic hinge rotations in walls, tangential drift ratio in walls or wall segments, slip along the construction joint, or total chord rotation for coupling beams) recommended for these components in Chapter 9.

Refer to Section 9.4.3.1 commentary for a discussion of construction joint slip. Refer to Section 11.4.2.3 commentary for a discussion of deformation limits for coupling beams or wall segments that might be classified as secondary components with no gravity load.

11.4.4.3 Discontinuous Walls

Acceptability should consider the strength and deformation capacity of the element, with special attention given to components supporting the wall, mechanisms for transferring loads to those elements, and mechanisms for transferring loads to the floor diaphragm. Interconnecting elements should also be assessed. The assessment should consider at least the following:

- Flexure, shear, sliding shear, anchorage, development, and splice actions in the wall, as discussed in Sections 11.4.3.1 and 11.4.3.2
- Flexure, shear, sliding shear, anchorage, development, and splice actions in the components supporting the discontinuous wall, as discussed in Sections 11.4.2 through 11.4.3.2



- Force transfer mechanisms between the discontinuous wall and the supporting components
- Foundation uplift

Inelastic response should be restricted to those components and actions identified for the separate wall and frame elements, as described above.

***Commentary:** Discontinuous walls supported on columns have contributed to damage and collapse of numerous structures during strong earthquakes. The primary causes of collapse are compression failure of the column due to overturning action of the wall on the column, or soft-story failure in either column shear or flexure* *due to discontinuity in the shear transfer path over the building height. Often, the best approach is to forego the numerical acceptance criteria and focus on retrofitting the building so as to remove the discontinuity.*

11.4.5 Combined Frame-Wall Element Acceptability

Acceptability should consider the strength and deformation capacity of the wall and frame system, as well as connections with adjacent elements. The assessment should consider aspects of performance for individual wall and frame elements, plus at least the following:

- Strength and deformability of wall and frame elements, as described in Sections 11.4.3 and 11.4.4
- Strength and deformability of connections between the frame and wall components

Inelastic response should be restricted to those components and actions identified for the separate wall and frame elements, as described in Sections 11.4.3 and 11.4.4.

11.4.6 Concrete Floor Diaphragm Acceptability

Acceptability should consider the strength of the diaphragm and its connections with adjacent elements. The assessment should consider at least the following:

- Shear strength of the slab and its connections with walls or other elements and components
- Adequacy of chords along the boundaries of the diaphragm, at reentrant corners, and at other irregularities in plan or elevation
- Strength of drag struts and collectors near concentrated loads and openings
- Tension due to out-of-plane anchorage of walls, wall panels, and other elements and components

In general, inelastic response should not be permitted in diaphragms, except where it is demonstrated by analysis or tests that inelastic action can be tolerated considering the selected performance levels. Where inelastic action is permitted, the acceptability limits should be established on the basis of the general modeling procedures of Section 9.5 and the general acceptability guidelines of Section 11.4.2.

Commentary: *There is relatively little analytical or experimental experience associated with nonlinear response of concrete diaphragms. Some local inelastic response may need to be accepted, but its acceptability should be gauged on a case-by-case basis. Widespread inelastic response of diaphragms should not be accepted.*

11.4.7 Foundation Acceptability

The acceptability of foundations in most cases will be determined by the effect of soil deformations on the supported structure. In addition, acceptability of structural foundation components should consider the following:

- Embedment of vertical reinforcement in footing or pile cap
- Footing, mat, or pile cap shear capacity
- Tensile capacity of footing, mat, or pile cap reinforcement
- Flexural, tensile, and shear capacity of piles or piers, including consideration of pile or pier connection to the cap

Additional guidance on foundations is given in Chapter 10.

Table 11-3. Numerical Acceptance Criteria for Plastic Hinge Rotations in Reinforced Concrete Beams, in radians

Component Type			Performance Level[3]				
			Primary			Secondary	
			IO	LS	SS	LS	SS
1. Beams controlled by flexure[1]							
$\frac{\rho-\rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d\sqrt{f'_c}}$[4]					
≤ 0.0	C	≤ 3	0.005	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.0	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.0	0.005	0.005	0.005	0.01
2. Beams controlled by shear[1]							
Stirrup spacing ≤ *d*/2			0.0	0.0	0.0	0.01	0.02
Stirrup spacing > *d*/2			0.0	0.0	0.0	0.005	0.01
3. Beams controlled by inadequate development or splicing along the span[1]							
Stirrup spacing ≤ *d*/2			0.0	0.0	0.0	0.01	0.02
Stirrup spacing > *d*/2			0.0	0.0	0.0	0.005	0.01
4. Beams controlled by inadequate embedment into beam-column joint[1]							
			0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions 1, 2, 3, and 4 occur for a given component, use the minimum appropriate numerical value from the table. See Chapter 9 for symbol definitions.
2. Under the heading "transverse reinforcement," "C" and "NC" are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic region: 1) closed stirrups are spaced at ≤ *d*/3, and 2) for components of moderate and high ductility demand the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.
3. Linear interpolation between values listed in the table is permitted.
   IO = Immediate Occupancy
   LS = Life Safety
   SS = Structural Stability
4. V=Design Shear.
5. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).

Table 11-4. Numerical Acceptance Criteria for Plastic Hinge Rotations in Reinforced Concrete Columns, in radians

Component Type			Performance Level[4]				
			Primary			Secondary	
			IO	LS	SS	LS	SS
1. Columns controlled by flexure[1]							
$\frac{P^5}{A_g f_c'}$	Trans. Reinf.[2]	$\frac{V^6}{b_w d \sqrt{f_c'}}$					
≤ 0.1	C	≤ 3	0.005	0.01	0.02	0.015	0.03
≤ 0.1	C	≥ 6	0.005	0.01	0.015	0.01	0.025
≥ 0.4	C	≤ 3	0.0	0.005	0.015	0.010	0.025
≥ 0.4	C	≥ 6	0.0	0.005	0.01	0.01	0.015
≤ 0.1	NC	≤ 3	0.005	0.005	0.01	0.005	0.015
≤ 0.1	NC	≥ 6	0.005	0.005	0.005	0.005	0.005
≥ 0.4	NC	≤ 3	0.0	0.0	0.005	0.0	0.005
≥ 0.4	NC	≥ 6	0.0	0.0	0.0	0.0	0.0
2. Columns controlled by shear[1,3]							
Hoop spacing ≤ *d*/2, or $\frac{P}{A_g f_c'} \leq 0.1$			0.0	0.0	0.0	0.01	0.015
Other cases			0.0	0.0	0.0	0.0	0.0
3. Columns controlled by inadequate development or splicing along the clear height[1,3]							
Hoop spacing ≤ *d*/2			0.0	0.0	0.0	0.01	0.02
Hoop spacing > *d*/2			0.0	0.0	0.0	0.005	0.01
4. Columns with axial loads exceeding $0.70P_o$[1,3]							
Conforming reinforcement over the entire length			0.0	0.0	0.005	0.005	0.01
All other cases			0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions 1, 2, 3, and 4 occur for a given component, use the minimum appropriate numerical value from the table. See Chapter 9 for symbol definitions.
2. Under the heading "transverse reinforcement," "C" and "NC" are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic hinge region: 1) closed hoops are spaced at ≤ *d*/3, and 2) for components of moderate and high ductility demand the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.
3. To qualify, 1) hoops must not be lap spliced in the cover concrete, and 2) hoops must have hooks embedded in the core or must have other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
4. Linear interpolation between values listed in the table is permitted.
   IO = Immediate Occupancy
   LS = Life Safety
   SS = Structural Stability
5. P=Design axial load
6. V=Design shear force
7. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).

Table 11-5. Numerical Acceptance Criteria for Total Shear Angle in Reinforced Concrete Beam-Column Joints, in radians

Component Type			Performance Level[4]				
			Primary[6]			Secondary	
			IO	LS	SS	LS	SS
1. Interior joints							
$\frac{P}{A_g f'_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]					
≤ 0.1	C	≤ 1.2	0.0	0.0	0.0	0.02	0.03
≤ 0.1	C	≥ 1.5	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≤ 1.2	0.0	0.0	0.0	0.015	0.025
≥ 0.4	C	≥ 1.5	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≤ 1.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≥ 1.5	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≤ 1.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≥ 1.5	0.0	0.0	0.0	0.01	0.015
2. Other joints							
$\frac{P}{A_g f'_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]					
≤ 0.1	C	≤ 1.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	C	≥ 1.5	0.0	0.0	0.0	0.01	0.015
≥ 0.4	C	≤ 1.2	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≥ 1.5	0.0	0.0	0.0	0.01	0.015
≤ 0.1	NC	≤ 1.2	0.0	0.0	0.0	0.005	0.01
≤ 0.1	NC	≥ 1.5	0.0	0.0	0.0	0.005	0.01
≥ 0.4	NC	≤ 1.2	0.0	0.0	0.0	0.0	0.0
≥ 0.4	NC	≥ 1.5	0.0	0.0	0.0	0.0	0.0

1. Under the heading "transverse reinforcement," "C" and "NC" are abbreviations for conforming and non-conforming details, respectively. A joint is conforming if closed hoops are spaced at ≤ $h_c/3$ within the joint. Otherwise, the component is considered non-conforming. Also, to qualify as conforming details under ii., 1) hoops must not be lap spliced in the cover concrete, and 2) hoops must have hooks embedded in the core or must have other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
2. The ratio $\frac{P}{A_g f'_c}$ is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional area of the joint and the concrete compressive strength. The design axial force is to be calculated considering design gravity and lateral forces.
3. The ratio V/V_n is the ratio of the design shear force to the shear strength for the joint.
4. Linear interpolation between values listed in the table is permitted.
   IO = Immediate Occupancy
   LS = Life Safety
   SS = Structural Stability
5. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).
6. No inelastic deformation is permitted since joint yielding is not allowed in a conforming building.

Table 11-6. Numerical Acceptance Criteria for Plastic Hinge Rotations in Reinforced Concrete Two-Way Slabs and Slab-Column Connections, in radians

		Performance Level[4]				
		Primary			Secondary	
Component Type		*IO*	*LS*	*SS*	*LS*	*SS*
1. Slabs controlled by flexure, and slab-column connections[1]						
$\frac{V_g}{V_o}$ [2]	Continuity Reinforcement[3]					
≤ 0.2	Yes	0.01	0.015	0.02	0.03	0.05
≥ 0.4	Yes	0.0	0.0	0.0	0.03	0.04
≤ 0.2	No	0.01	0.015	0.02	0.015	0.02
≥ 0.4	No	0.0	0.0	0.0	0.0	0.0
2. Slabs controlled by inadequate development or splicing along the span[1]						
		0.0	0.0	0.0	0.01	0.02
3. Slabs controlled by inadequate embedment into slab-column joint[1]						
		0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions 1, 2, and 3 occur for a given component, use the minimum appropriate numerical value from the table.
2. V_g = the gravity shear acting on the slab critical section as defined by ACI 318, V_o = the direct punching shear strength as defined by ACI 318.
3. Under the heading "Continuity Reinforcement," assume "Yes" where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, assume "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, assume "No."
4. Linear interpolation between values listed in the table is permitted.
   IO = Immediate Occupancy
   LS = Life Safety
   SS = Structural Stability
5. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).

Table 11-7. Numerical Acceptance Criteria for Plastic Hinge Rotations in Reinforced Concrete Walls and Wall Segments Controlled by Flexure, in radians

Component Type			*Performance Level*[4]				
			Primary			*Secondary*	
			IO	*LS*	*SS*	*LS*	*SS*
1. Walls and wall segments controlled by flexure							
$\frac{(A_s - A_s')f_y + P}{t_w l_w f_c'}$ [1]	$\frac{V}{t_w l_w \sqrt{f_c'}}$ [2]	Boundary Element[3]					
≤ 0.1	≤ 3	C	0.005	0.010	0.015	0.015	0.020
≤ 0.1	≥ 6	C	0.004	0.008	0.010	0.010	0.015
≥ 0.25	≤ 3	C	0.003	0.006	0.009	0.009	0.012
≥ 0.25	≥ 6	C	0.001	0.003	0.005	0.005	0.010
≤ 0.1	≤ 3	NC	0.002	0.004	0.008	0.008	0.015
≤ 0.1	≥ 6	NC	0.002	0.004	0.006	0.006	0.010
≥ 0.25	≤ 3	NC	0.001	0.002	0.003	0.003	0.005
≥ 0.25	≥ 6	NC	0.001	0.001	0.002	0.002	0.004

1. A_s = the cross-sectional area of longitudinal reinforcement in tension, A_s' = the cross-sectional area of longitudinal reinforcement in compression, f_y = yield stress of longitudinal reinforcement, P = axial force acting on the wall considering design load combinations, t_w = wall web thickness, l_w = wall length, and f_c' = concrete compressive strength.
2. V = the design shear force acting on the wall, and other variables are as defined above.
3. The term "C" indicates the boundary reinforcement effectively satisfies requirements of ACI 318. The term "NC" indicates the boundary requirements do not satisfy requirements of ACI 318.
4. Linear interpolation between values listed in the table is permitted.
   IO = Immediate Occupancy
   LS = Life Safety
   SS = Structural Stability
5. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).

Table 11-8. Numerical Acceptance Criteria for Tangential Drift Ratios for Reinforced Concrete Walls and Wall Segments Controlled by Shear, in radians

	Performance Level				
	Primary			Secondary	
Component Type	IO	LS	SS	LS	SS
1. Walls and wall segments controlled by shear					
All cases	0.004	0.006	0.0075	0.0075	0.015

1. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).
IO = Immediate Occupancy
LS = Life Safety
SS = Structural Stability

Table 11-9. Numerical Acceptance Criteria for Shear Sliding Displacements for Reinforced Concrete Walls and Wall Segments Controlled by Shear, in inches

	Performance Level				
	Primary			Secondary	
Component Type	IO	LS	SS	LS	SS
1. Walls and wall segments controlled by shear					
All cases	0.2	0.3	0.4	0.4	0.8

1. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).
IO = Immediate Occupancy
LS = Life Safety
SS = Structural Stability

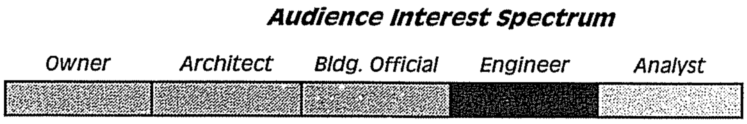
Table 11-10. Numerical Acceptance Criteria for Chord Rotations for Reinforced Concrete Coupling Beams, in radians

		Performance Level[3]				
		Primary			*Secondary*	
Component Type		*IO*	*LS*	*SS*	*LS*	*SS*
1. Coupling beams controlled by flexure						
Longitudinal reinforcement and transverse reinforcement[1]	$\frac{V}{b_w d\sqrt{f_c'}}$ [2]					
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.006	0.015	0.025	0.025	0.040
Conventional longitudinal reinforcement with conforming transverse reinforcement	≥ 6	0.005	0.010	0.015	0.015	0.030
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≤ 3	0.006	0.012	0.020	0.020	0.035
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≥ 6	0.005	0.008	0.010	0.010	0.025
Diagonal reinforcement	N/A	0.006	0.018	0.030	0.030	0.050
2. Coupling beams controlled by shear						
Longitudinal reinforcement and transverse reinforcement[1]	$\frac{V}{b_w d\sqrt{f_c'}}$ [2]					
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.006	0.012	0.015	0.015	0.024
Conventional longitudinal reinforcement with conforming transverse reinforcement	≥ 6	0.004	0.008	0.010	0.010	0.016
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≤ 3	0.006	0.008	0.010	0.010	0.020
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≥ 6	0.004	0.006	0.007	0.007	0.012

1. Conventional longitudinal steel consists of top and bottom steel parallel to the longitudinal axis of the beam. The requirements for conforming transverse reinforcement are: 1) closed stirrups are to be provided over the entire length of the beam at spacing not exceeding $d/3$; and 2) the strength provided by the stirrups (V_s) should be at least three-fourths of the design shear.
2. V = the design shear force on the coupling beam in pounds, b_w = the web width of the beam, d = the effective depth of the beam, and f_c' = concrete compressive strength in psi.
3. Linear interpolation between values listed in the table is permitted.
   IO = Immediate Occupancy
   LS = Life Safety
   SS = Structural Stability
4. For lightweight concrete, use 75 percent of tabulated values (see Section 9.5.2.2).

Chapter 12

Nonstructural Components



12.1 Introduction

Five levels of performance of nonstructural components and systems are described in Chapter 3: Operational, Immediate Occupancy, Life Safety, Hazards Reduced, and Not Considered. These nonstructural performance levels are combined with certain limits on structural damage, i.e., structural performance levels, to form a complete target Building Performance Level as shown in Table 3-1.

Depending on the performance goals for a particular building, the scope of the field investigation for nonstructural elements will vary. General Guidance for field investigation is provided in Chapter 5. Some nonstructural items are relatively sensitive to accelerations and others are more affected by displacement or drift. This needs to be considered in the acceptability analysis on nonstructural components. The technical procedures currently available are limited and only qualitative in many instances. The recent federal guidelines (ATC 1996a) provide a comprehensive summary and commentary.

12.2 Acceptability Criteria

This chapter gives minimum criteria that are expected to provide the Immediate Occupancy, Life Safety and Hazards Reduced levels of nonstructural performance. Considerations required to develop criteria for the Operational level of performance are noted but complete criteria for this level are not included in this document. The Not Considered level of performance requires no criteria.

Commentary: *In this document, acceptance criteria are limited to a listing of the nonstructural components and systems that should be considered for each performance level.*

12.2.1 Operational, NP-A

The Operational level of performance for the building's nonstructural systems and components requires consideration of the following:

- Internal damage and disruption
- The ability of critical equipment to function after being subjected to the motion of the building
- Limitations that may be imposed by the external availability of utilities, supplies, or communications

These considerations are in addition to the criteria for the Immediate Occupancy level of performance. Development of criteria to maintain function of critical equipment and to provide adequate backup or alternate sources for external services is building specific and is not included in this document.

12.2.2 Immediate Occupancy, NP-B

All equipment, systems, and finishes permanently attached to the building structure, as

*Table 12-1. Items to be Investigated for the Life Safety Nonstructural Performance Level**

Equipment and Distribution Systems	*Other*
◆ Gas or fuel boilers and furnaces**	◆ Exterior cladding and decoration
◆ Independently mounted or hung overhead HVAC equipment weighing over 20 lb.	◆ Masonry partitions
◆ High pressure steam piping over 2" ID	◆ Partitions that are unusually tall, heavily decorated, or that support heavy casework or shelving
◆ Fire suppression piping*	◆ Suspended lath and plaster or heavily decorated ceilings
◆ Hazardous material piping*	◆ Traction elevators
◆ Light fixtures, diffusers, or returns supported only by light gage lay-in ceiling runners	◆ Hazardous material storage

* The items listed in this table include all items listed in Table 12-2.

** To be investigated regardless of location.

listed in Table 16A-O of the 1995 *California Building Code* (CBSC 1995), should be investigated for their ability to remain in place with only minor damage under the demands of the earthquake ground motions incorporated in the designated performance objective. For consideration under the Design Earthquake, the acceptance criteria listed therein for essential facilities may be used.

Contents, not permanently attached to the building, are not controlled by design codes, but can be the source of considerable damage and disruption that could affect continued occupancy of some spaces. The anchorage and storage of contents should be reviewed by owners and tenants of buildings that require immediate occupancy.

12.2.3 Life Safety, NP-C

The equipment, systems, and finishes permanently attached to the building structure listed in Table 12-1 and located in or over occupied areas, should be included for consideration for the Life Safety nonstructural performance level. Occupied areas are defined as spaces intended for more than incidental human use or regularly occupied by persons for approximately two hours per day or more.

The items listed in Table 12-1 shall be shown by analysis or test not to present a falling or secondary risk under the demands of the earthquake ground motion incorporated in the designated performance objective, or shall be replaced or retrofitted.

As noted in the nonstructural performance level descriptions in Chapter 3 and the representative damage descriptions in Table 11-1, drift-sensitive components may be expected to experience considerable damage for this and lower levels of performance.

12.2.4 Hazards Reduced, NP-D

The nonstructural components or systems listed in Table 12-2 should be included for consideration under the Hazards Reduced nonstructural performance level. In general, elements should be investigated if their failure would endanger a large area or would likely injure more than one person.

The items listed in Table 12-2 shall be shown by analysis or test not to present a falling or secondary risk under the demands of the earthquake ground motion incorporated in the designated performance objective, or shall be replaced or retrofitted.

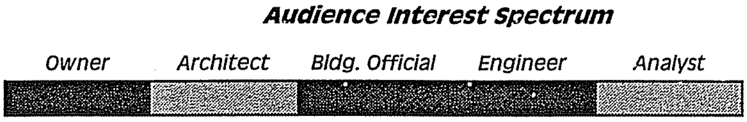
Commentary: *The cost and disruption of bringing nonstructural systems in older buildings into conformance with current codes is high. Although these systems have suffered considerable damage in past earthquakes, the damage has generally not caused extensive hazardous conditions. Nonstructural systems, therefore, have not been reviewed in most retrofits to date. However, large, highly vulnerable elements have often been investigated for their potential to fall and cause injury, particularly over points of egress. The criteria used to determine the need to investigate is unclear, but vulnerability-to-damage and the extent of occupant-exposure are initial considerations. The extent of retrofit is often a cost consideration. The Nonstructural Performance Level of Hazards Reduced is intended to include only major hazards and encourage cost effective risk reduction.*

Table 12-2. Items to be Investigated for the Hazards Reduced Nonstructural Performance Level

- Parapets, exterior walls, cladding, and other major external falling hazards, particularly directly over exits and over roofs of lower adajacent buildings
- Interior masonry walls or partitions
- Major suspended plaster ceilings or other overhead elements that could endanger a large area or are likely to directly injure more than one person
- Elements and equipment that store or transport hazardous materials
- Elements and equipment vulnerable to damage that are likely to start fires

Chapter 13

Conclusions and Future Directions



13.1 Introduction

Initially the focus of the development of this document was directed toward an analytical methodology. Specifically the project team intended to adapt nonlinear static structural analysis procedures to the evaluation and retrofit of concrete buildings using capacity spectrum techniques. While the promise of these analytical procedures is formidable, their use has wide ranging implications. The ability to estimate realistically the response of a building to actual earthquakes naturally leads to the question of whether this response is adequate. The systematic evaluation of the adequacy of building response for earthquakes of various probabilities of occurrence is the essence of performance-based design. The subsequent decisions have implications that extend beyond the realm of the engineer. In fact, the impact on conventional practice for building owners, architects, engineers, building officials and others is fundamental. With this realization, the perspective of the methodology expanded beyond the analytical to the general.

The development of this document benefited greatly from input from numerous sources. The project team consisted of a large group of experienced engineers. An oversight panel of qualified individuals monitored the progress and reviewed each of the drafts. Engineering firms tested the basic concepts on example buildings. A benefit/cost analysis explored economic implications. There were a series of workshops for practitioners and other potential users to discuss the document and provide suggestions for improvement. This chapter summarizes collective opinions as to the strengths and weaknesses of the current methodology and presents consolidated recommendations for future improvements. These general conclusions fall into two categories in accordance with the dual perspective of the methodology itself. At one end of the spectrum are the technical issues of analysis, testing, component behavior and others of primary concern to engineers and analysts. The broader interests of owners, architects, and building officials in performance-based design, quality control, and costs of design and construction complete the picture of meeting the challenge of concrete buildings effectively and efficiently.

13.2 Additional Data

The development of this document relied heavily upon the collective past experience of the project team members. Supplemental specific studies augment and enhance this empirical base. Practicing engineers tested preliminary procedures on example buildings and provided guidance for further development. The results of the example building studies were ultimately the subject of an independent benefit/cost study. In order to formulate the foundation procedures of Chapter 10, a group of structural and geotechnical engineers reviewed and summarized relevant technical information on the seismic performance of building foundations. While none of these supplemental efforts are directly applicable to specific evaluation and retrofit projects, the information is instructive and useful to others.

13.2.1 Example Building Studies

Appendices A, B, C, and D document technical analyses of four concrete buildings. Each structural engineering firm responsible for one of the analyses had a representative on the product development team; however, other individuals from each firm performed the work on the example buildings. Consequently, their efforts provided a degree of independent testing of the procedures as they were developed.

The interchange of information between the developers of the methodology and the example building analysts occurred continuously over time. Initially, meetings among the entire group concentrated on preliminary analyses of each building. These consisted of traditional procedures (FEMA 178, code analysis, inelastic demand ratio procedures) and nonlinear procedures in some cases. Based on these meetings, the scope and emphasis of the technical procedures were adjusted to address the practical issues evident from the example building studies. The preliminary procedures were subsequently applied to the example buildings. The project team then continued the cycle of testing and improving the overall methodology. An example of this process is the procedure for determining displacement demand using the capacity spectrum method. Initially, the procedure was graphical in nature. Those working on the example buildings suggested a more numerical approach compatible with computer-oriented procedures. An iterative technique was formulated and applied to the example buildings. Finally, the document includes several techniques to offer alternatives depending on the inclination of individual users.

The example buildings served several other purposes. In order to assess the effects of variable soil strength and stiffness in building foundations, comparative analyses were made on individual buildings. On the one concrete frame building, foundation effects did not appear to be significant. On the other three shear wall buildings, in contrast, the foundation assumptions significantly affected the results of the performance analysis. This is discussed further below as one of the challenges for improving analysis procedures in the future.

The example building analysts subjected models of each structure to nonlinear time history analysis. The time history records were adjusted to be generally representative of the spectra used for the static nonlinear procedures. The results of these analyses were compared to those from the static nonlinear work. The agreement between the two approaches was variable, raising questions of relative accuracy of both. Discrepancies may, in part, be due to higher mode effects and deserve further investigation as discussed below.

In addition to evaluating existing performance characteristics of the example buildings, the analysts developed conceptual retrofit measures. They then re-analyzed the structures to measure the effectiveness of the retrofit. The results from this exercise were used for the benefit/cost study as were the opinions of the analysts on the strength and weaknesses of the general analysis methodology. These are discussed further below in this chapter.

It should be noted that the example building reports in the appendices are presented as professional reports by individual engineering firms. These are not necessarily tutorials representing complete and uniform application of the technical procedures in the document. Nonetheless, they illustrate the general application of nonlinear analysis procedures. Additionally, they reveal the application of substantial engineering judgment essential to the methodology by four different qualified firms. Finally, the results of the analysis are presented as research data to contribute to further development of analytical techniques on future projects.

13.2.2 Cost Effectiveness Study

Appendix E presents the results of a benefit/cost analysis of the methodology based primarily on the example building studies. The objectives of the study were:

1. Estimate the approximate construction costs for retrofit measures for the example buildings.
2. Compare these costs to those that might have resulted from the application of more traditional methodologies.
3. Examine the relationship between costs and retrofit performance goals for the example buildings.
4. Perform benefit/cost analyses for the performance levels evaluated in each example building study.
5. Investigate the ease of use of the analysis procedures.
6. Investigate the consistency of application of the procedures among the four engineering firms.

The cost analyses included the direct costs associated with retrofit measures to address seismic performance. Other costs such as disabled access, fire and life safety improvements, and hazardous material abatement, are often required for rehabilitation projects. The direct costs for seismic retrofit for the four example buildings to the Life Safety Performance Objective ranged from 5 to 18 percent of the estimated replacement cost for each building.

Comparison with traditional retrofit costs tends to indicate that the new methodology is very economical. This conclusion seems logical; yet the degree of economy is probably highly dependent on the individual characteristics of a given building. The relationship between performance level and cost also seems to be highly building specific although a clear relationship of higher costs for higher (better) performance was observed for all four buildings.

The new methodology was observed to be more difficult and time consuming than traditional techniques for the engineers. This is understandable since the procedures are new and not fully developed. The ease of application and consistency among users will improve over time. The compensating benefit to increased engineering effort is a more insightful understanding of building seismic performance. This understanding leads to more economical and effective retrofit measures.

13.2.3 Supplemental Information on Foundation Effects

The proposed methodology includes technical procedures for the explicit modeling of building foundations in the structural analysis. In the past, some engineers have modeled foundations. These procedures, however, provide a framework for the foundation modeling process consistent with the broader structural analysis methodology. They were developed by a team of geotechnical and structural engineers. As a part of the development process, this geo-structural working group assembled data on the post-earthquake performance of building foundations, past and current design practice, and theoretical and experimental work on foundations. As a further resource for practicing engineers, the group summarized this material in Appendix F. The conclusions of the group were:

1. Traditional seismic design procedures do not reflect realistic consideration of geotechnical and foundation effects.
2. Geotechnical and foundation response can significantly influence the performance of buildings during earthquakes.
3. Costs of retrofit of existing buildings warrant realistic assessment of seismic performance of components including foundations and underlying soils.
4. Proposed methodologies are based on estimates of force-displacement relationships for elements and components subject to seismic loads.
5. Existing empirical data on geotechnical materials are inadequate to provide sufficient design information for all cases.

6. Research to investigate force-displacement behavior of geotechnical materials and foundation assemblies is needed.
7. Damage reconnaissance for earthquakes should include documentation of geotechnical and foundation effects on buildings.

13.3 Potential Benefits

The benefits of the methodology are founded upon a fundamentally improved understanding of the seismic performance of buildings. This occurs as a result of the technical changes in the procedures used by the engineer in the evaluation of buildings and the design of retrofit measures. The implications, however, extend to a much broader scale. The technical improvements enable and facilitate the use of performance-based design. This transition greatly enhances the options for building owners in the management of seismic risk in an effective and efficient way.

13.3.1 Improved Understanding of Seismic Performance

The key benefit of the analytical portions of the methodology is a realistic picture of actual structural behavior during earthquakes. Traditional methods tend to obscure this understanding.

- **Explicit Inelastic Behavior:** The nonlinear static procedures consider the individual inelastic behavior of each component of a structure resisting seismic forces. Traditional procedures generally consolidate inelastic action into a global force reduction factor.
- **Sequence of Inelastic Actions:** The analysis technique is progressive, allowing the engineer to witness which components reach behavior thresholds before others. This is true throughout the response. IDR methods can identify components that yield first but cannot predict the effects of load redistribution.
- **Estimate of Degradation:** An engineer can modify the capacity of the structure to generate an envelope of behavior to reflect loss of strength or stiffness due to the duration of an earthquake. Although this degradation can only be approximated, this capability enhances the estimate of actual performance.
- **Foundation Effects:** Traditionally, foundations for buildings are assumed to be rigid. In reality, this is not the case and foundation flexibility greatly modifies response to earthquakes. The recommended procedures include explicit modeling of foundations analogous to other structural elements.
- **Specific Deficiencies:** The new techniques identify specific deficiencies of individual components of a structure as opposed to general lack of overall strength.
- **Directed Retrofit Strategy:** In the past, retrofit strategies for buildings gravitated toward the addition of lateral force resistance (strength) and the linking of resisting elements together to form a complete load path. While still satisfactory for many simple cases, this global approach can overlook critical specific deficiencies and overcompensate for less important ones. The new techniques facilitate the focus on important deficiencies for both greater effectiveness and economy.

13.3.2 Performance Based Evaluation and Retrofit

The improved understanding of structural behavior allows the use of performance-based evaluation. Specific goals for seismic behavior can be implemented.

- **Flexibility:** Owners need not simply accept the vague objectives of conventional prescriptive techniques. They can adjust performance goals to meet their specific needs given a realistic understanding of the expected, and acceptable, damage states.
- **Prioritization:** Priorities within individual buildings and among groups of buildings can be set to address relatively important deficiencies and critical needs.
- **Coordination:** Performance-based design provides the opportunity to integrate seismic

risk management into the day-to-day operations of the building owner.

- **Uncertainty and Reliability**: Earthquakes and concrete buildings pose many sources of uncertainty. Performance-based evaluation and nonlinear static analysis procedures do not eliminate risks associated with this uncertainty. They provide a more refined context for dealing with uncertainty. Traditional procedures tend to deal with uncertainty and risk on a global level. The new techniques facilitate the consideration of uncertainty for individual parameters. For example, ground motions can be specified in probabilistic terms independently of structural characteristics. Component properties for analysis and performance acceptability limits replace the traditional "R-factors". The new techniques used for a single building can both eliminate unnecessary conservatism for some parameters and increase it for others, based on the specific judgment of the engineer. The overall results may be more conservative for some buildings and less for others when compared to current procedures. If implemented properly by a qualified engineer, the results reflect, more reliably than traditional procedures, the effects of unavoidable uncertainties on the specific building in question.

13.4 Major Challenges

There are presently a number of drawbacks to implementation of the methodology. Not surprisingly, weaknesses of the methodology parallel its strengths. The new technical procedures require greater information on material properties and more sophisticated analysis tools. From the broader perspective, performance-based design demands basic changes on the part of everyone involved.

13.4.1 Technical Issues

The recommended procedures are new. Technical tools and information, not previously needed for traditional methods, are required to improve the application.

- **Foundation Material Properties:** Soils design information has been based on consideration of long-term settlements under static dead loads. Realistic seismic performance analysis requires estimates of behavior under short-term dynamic loading.
- **Inelastic Behavior of Structural Components:** Modeling and acceptability criteria for structural components are necessarily conservative now. This is due to the lack of coordinated experimental and theoretical data.
- **Nonstructural Component Characteristics:** The behavior of nonstructural components is treated only qualitatively in most cases. Damage needs to be correlated with displacements and accelerations for a broad range of components to enhance performance-based design procedures.
- **Nonlinear Analysis Software:** Currently available software to implement nonlinear static procedures are not adequate. Component modeling alternatives are restrictive. Torsional response cannot be modeled easily.
- **Preferred Nonlinear Static Procedure:** While there is general agreement that nonlinear static procedures provide great insight, there is a lack of consensus on the preferred format for the analyses. Some find the capacity spectrum method conversion to ADRS format needlessly complex and offer alternative formulations of the same basic parameters (Powell, 1996).
- **Higher Mode Effects:** Nonlinear static procedures do not address directly the potential for participation of higher modes of vibration in structural response. The need for, and efficacy of, approximate measures require extensive future study.
- **Degradation, Damping, and Duration:** Although the new analysis techniques include more explicit consideration of important seismic response parameters, they currently approximate the effects of degradation of stiffness and strength, damping, and duration of

shaking primarily at a global structural level. It is theoretically possible to include these at a more refined component level.

13.4.2 Fundamental Change to Traditional Practice

The greatest challenge posed by the new methodology is the basic change required in the design and construction process.

- **Greater Analytical Expertise and Effort:** Engineers must exercise a greater degree of analytical expertise with the new technical procedures compared to the traditional. The greater effort will require larger engineering design fees. This may appear to be unnecessary and self-serving by those not cognizant of the benefits. Further guidance is required on when simplified analysis procedures might be used in place of the more sophisticated new techniques.
- **Peer Review:** The methodology demands a large amount of judgment on the part of the engineer. Peer review is essential to avoid misuse by inexperienced individuals. Some may consider peer review as an unnecessary complication and expense.
- **Building Officials:** Building officials will not be able to check designs using conventional practices for prescriptive criteria. Significant resistance is likely, particularly considering the current atmosphere of government budget constraints.
- **Owners:** Building owners cannot simply transfer risk to others with "code" compliance. Performance-based design is a technique to manage risk. The desired performance is a goal whose achievement, it must be understood, is not guaranteed. Owners play a central role in decision-making and goal-setting. The amount of guidance and explanation currently available to owners is very minimal. Owners often select engineers for projects on the basis of lowest cost. Contractual arrangements for geotechnical input are at odds with the need for collaboration between the geotechnical and structural engineer.
- **Uncertainty and Reliability:** The sophistication of the new technical procedures and the explicit consideration of performance goals can imply a deceptive impression of accuracy and reliability. The complexity of some procedures can lead to mistakes. Computer modeling can take on an unwarranted air of infallibility. Engineers without the requisite experience and judgment can misuse the methodology. Owners mistakenly can interpret goals as implied warrantees or guarantees. The uncertainties inherent to earthquakes and concrete buildings still prevail.

13.5 Recommended Action Plan

To meet the challenges presented in the previous section a dual action plan is recommended. The technical portion of these recommendations concentrates on filling knowledge gaps with projects and programs for traditional focused research. Equally important are programs to educate all of the individuals with an interest in evaluation and retrofit of buildings about the changes resulting from performance-based techniques and the new technical procedures.

13.5.1 Basic Research

Technical needs should be addressed with a coordinated basic research program. The objectives must be focused on the specific needs of the engineer in practice.

- **Geotechnical components and foundation systems:** This area of research is virtually untouched. The emerging analysis procedures put pressure on the research community to produce practical data for this important aspect of seismic behavior. In-situ and laboratory tests, as well as theoretical modeling techniques, are required. The effect of short-term dynamic loading on basic soil properties is of particular interest. The effects of partial basements and sloping sites require further research. Structural evaluation and retrofit strategies for liquefaction and lateral spreading beneath buildings should be developed.

- **Structural Components Modeling and Acceptability Criteria:** An organized testing protocol for structural elements is necessary to focus work on practical issues. The nonlinear static procedures all rely on the same generalized component properties. Research products should be formatted to facilitate the transfer of information to practical application.
- **Nonstructural Component Damage:** A basic framework for categorizing nonstructural components according to their sensitivity to displacement- and/or acceleration-related damage is required. This framework could be used by vendors, manufacturers, and researchers to provide engineers with quantitative input to performance analyses.
- **Nonlinear Analysis Procedures and Software:** Products targeted for practical application by design offices are needed. Capabilities should be broad to avoid "force-to-fit solutions". A graphical user-interface is essential to reduce data management effort and facilitate visualization of behavior. Three dimensional capability is essential to address torsional behavior.
- **Probabilistic Characterization of Capacity and Performance:** Eventually the uncertainty associated with structural capacity should be quantified to be compatible with the specification of seismic demand in probabilistic terms. This will enable an enhanced measure of performance and risk.
- **Revision of Building Codes:** As the procedures for evaluating and retrofitting concrete buildings emerge, the goal should be simplified, concise measures suitable for use in building code provisions.

13.5.2 Increase Awareness

Much training and communication are necessary to facilitate the change that all involved must make. At present, the new methodology appears costly and complicated. The benefits have yet to be adequately communicated.

- **Broadly Based Examples:** The current building examples are technically focused. Many more broadly based examples demonstrating the strengths and weaknesses and illustrating the differences between the many new methods and traditional approaches are required. These should be oriented toward building owners. Cost analyses including engineering fees and life-cycle considerations could greatly enhance the understanding of alternatives for evaluation and retrofit.
- **Assistance to Owners:** Seminars and workshops should be held to help owners adapt to the new performance-based approach. Engineers should be included to broaden their perspective. The descriptions of performance levels that are now available could be improved to relate better to the experience and language of owners. State agencies are a natural starting point for workshops and seminars aimed at the management level. These could be expanded to the private sector through organizations such as the Building Owners and Managers Association. This initiative to engage building owners has not yet been implemented in any effective program.
- **Technical Training for Engineers:** Continued development of training seminars and materials will assist engineers in honing their technical skills and gaining familiarity with the new technology. This training is actually already occurring through programs offered by organizations such as the Structural Engineers Association of California. It is likely that the demand for this information will grow and be filled by similar programs by other existing organizations.
- **Specific Guidance on the Use of Simple Procedures:** The current methodology recognizes that, for some buildings, simple linear elastic analysis procedures could be used instead of the more complex nonlinear static procedures. Although some qualitative

guidance is offered, the choice of technique is largely a matter of engineering judgment. In the future, as more applications of the methodology become available, some trends may emerge that may facilitate development of a more prescriptive format for selection of evaluation and design techniques. For example, a matrix of choices might relate procedures, peer review requirements, and foundation modeling to building size, height, configuration, and system.

- **Seminars for Building Officials:** A dialogue with building officials will go a long way in facilitating the implementation. They need to know who can perform peer reviews and how these reviews relate to the plan check process. Specific guidance should be developed on the need for peer review depending on the characteristics of buildings and the complexity of the analysis procedure. Curricula for programs which might be administered through the California Building Officials (CALBO) or the International Conference of Building Officials (ICBO) should be developed.
- **Collaborative Workshops for Structural and Geotechnical Engineers:** There currently is a lack of adequate communication between geotechnical and structural engineers on the issues and the information needed for realistic assessment foundation effects. This is an interdisciplinary problem that has long been neglected. A long-term, concerted, collaborative effort is needed. A mechanism for continual exchange of information is essential.

References

ACI, 1985, "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures," ACI-ASCE Committee 352, *Journal of the American Concrete Institute*, vol. 82, no. 3, pp. 266-283.

ACI, 1986, *Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures,* ACI Committee 209, American Concrete Institute, Detroit, Michigan.

ACI, 1988, "Recommendations for Design of Slab-Column Connections in Monolithic Reinforced Concrete Structures," ACI-ASCE Committee 352. *ACI Structural Journal*, vol. 85, no. 6, pp. 675-696.

ACI, 1995, *Building Code Requirements for Structural Concrete* (ACI 318-95) *and Commentary* (ACI 318R-95), ACI Committee 318, American Concrete Institute, Farmington Hills, Michigan.

Alami, M. S., and J. K. Wight, 1992, *Numerical Modeling and Parametric Investigation of R/C Coupled Wall-Frame Systems Under Seismic Loading*, UMCEE 92-20, University of Michigan, Department of Civil and Environmental Engineering, Ann Arbor, Michigan.

Ali, A., and J. K. Wight. 1991, "RC Structural Walls With Staggered Door Openings," *Journal of Structural Engineering*, ASCE, vol. 117, no. 5, pp. 1514-1531.

Army, Navy, and Air Force, Department of the U. S. (ARMY), 1986, *Seismic Design Guidelines for Essential Buildings,* Army TM 5-809-10-1, NAVFAC P355.1, AFM 88-3, Chap. 13, Sec. A, U.S. Department of the Army, Washington, D.C.

Army, Navy, and Air Force, Department of the U. S. (ARMY), 1996, *Seismic Dynamic Analysis for Buildings,* Draft Revision of Army TM 5-809-10-1 (unpublished).

ASCE, 1995, *Standard for Independent Peer Review,* American Society of Civil Engineers, Draft.

Aschheim, M., and J. P. Moehle, 1992, *Shear Strength and Deformability of RC Bridge Columns Subjected to Inelastic Cyclic Displacements,* UCB/EERC-92/04, University of California, Earthquake Engineering Research Center, Berkeley, California.

ASTM Standards (various numbers), American Society for Testing Materials, Philadelphia, Pennsylvania.

ATC, 1983, *Seismic Resistance of Reinforced Concrete Shear Walls and Frame Joints: Implications of Recent Research for Design Engineers*, ATC-11 Report, Applied Technology Council, Redwood City, California.

ATC, 1987, *Evaluating the Seismic Resistance of Existing Buildings,* ATC-14 Report, Applied Technology Council, Redwood City, California.

ATC, 1996a, *Guidelines for the Seismic Rehabilitation of Buildings, Volume I, Guidelines, and Volume II, Commentary,* FEMA 273/274–Ballot Version. Prepared by the Applied Technology Council for the Building Seismic Safety Council; published by the Federal Emergency Management Agency.

ATC, 1996b, *Improved Seismic Design Criteria for California Bridges: Provisional Recommendations,* ATC-32 Report, Applied Technology Council, Redwood City, California.

Aycardi, L. E., J. B. Mander, and A. M. Reinhorn, 1992, *Seismic Resistance of Reinforced Concrete Frame Structures Designed Only for Gravity Load, Part II: Experimental Performance of Subassemblages,* NCEER-92-0028, National Center for Earthquake Engineering Research, Buffalo, New York.

Bartlett, P. E., 1976, "Foundation Rocking on a Clay Soil," Report no. 154, M. E. thesis, University of Auckland, School of Engineering, Aukland, New Zealand.

Bass, R. A., R. L. Carrasquillo, and J. O. Jirsa, 1989, "Shear Transfer Across New and Existing Concrete Interfaces," *ACI Structural Journal,* vol. 86, no. 4, pp. 383-393.

Beres, A., R. N. White, and P. Gergely, 1992, *Seismic Behavior of Reinforced Concrete Frame Structures with Nonductile Details, Part I: Summary of Experimental Findings of Full Scale Beam-Column Joint Tests,* NCEER-92-0024, National Center for Earthquake Engineering Research, Buffalo, New York.

Bowles, J. E., 1982, *Foundations Analysis and Design.*

BSSC, 1992, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings,* FEMA 178, Prepared by the Building Seismic Safety Council; published by the Federal Emergency Management Agency, Washington, D.C.

BSSC, 1995, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings: 1994 Edition,* FEMA 222A (Provisions), and FEMA 223A (Commentary), prepared by the Building Seismic Savety Council, published by the Federal Emergency Management Agency, Washington, D.C.

BSSC, 1996, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings: 1997 Proposed Edition,* FEMA 222A (Provisions), and FEMA 223A (Commentary), prepared by the Building Seismic Safety Council (Unpublished).

CBSC, 1995, *California Building Code, Title 24, Part 2, CCR,* prepared by the California Building Standards Commission; published by the International Conference of Building Officials, Whittier, California.

CBSC, 1996, *Code Change Item 41-Earthquake Evaluation and Design for Retrofit of Existing State-Owned Buildings*, Monograph of Code Change Submittals for 1996 (June 1996), pp. 106-140, California Building Standards Commission, Sacramento, California.

CDGS, 1994, *State Building Seismic Program, Report and Recommendations,* California Department of General Services, Sacramento, California.

CDMG, 1985, *Fault Rupture Zones in California,* Special Publication 42, California Department of Conservation, Division of Mines and Geology, Sacramento, California.

CDMG, 1994, *Geologic Data Map No. 6,* Prepared by C. Jennings, California Department of Conservation, Division of Mines and Geology, Sacramento, California.

Charney, F. A., 1991, "Correlation of the Analytical and Experimental Inelastic Response of a 1/5-Scale Seven-Story Reinforced Concrete Frame-Wall Structure," *Earthquake Resistant Concrete Structures: Inelastic Response and Design,* ACI Special Publication 127, American Concrete Institute, Detroit, Michigan.

Chopra, A. K., 1995, *Dynamics of Structures-Theory and Applications to Earthquake Engineering,* Prentice-Hall, Inc., Englewood Cliffs, New Jersey.

COLA, 1995, *Earthquake Hazard Reduction in Existing Reinforced Concrete Buildings and Concrete Frame Buildings with Masonry Infills,* City of Los Angeles, January 31, 1995 (unpublished).

CSI, 1992, *SAP90: A Series of Computer Programs for the Finite Analysis of Structures,* Computers and Structures, Inc., Berkeley, California.

CSI, 1995, *ETABS: Three Dimensional Analysis of Building Systems,* Computers and Structures, Inc., Berkeley, California.

CSSC, 1991a, *Breaking the Pattern: A Research and Development Plan to Improve Seismic Retrofit Practices for Government Buildings,* SSC 91-05, Prepared by Charles C. Thiel Jr., Frederick J. Willsea, and Jogeshwar P. Singh, California Seismic Safety Commission, Sacramento, California.

CSSC, 1991b, *Policy on Acceptable Levels of Earthquake Risk in State Buildings,* SSC 91-01, California Seismic Safety Commission, Sacramento, California.

CSSC, 1992, *Architectural Practice and Earthquake Hazards,* SSC 92, California Seismic Safety Commission, Sacramento, California.

CSSC, 1994a, *Provisional Commentary for Seismic Retrofit,* Product 1.1 of the Proposition 122 Seismic Retrofit Practices Improvement Program, SSC 94-02, Prepared by Rutherford & Chekene, Sacramento: California Seismic Safety Commission, Sacramento, California.

CSSC, 1994b, *Review of Seismic Research Results on Existing Buildings* Product 3.1 of the Proposition 122 Seismic Retrofit Practices Improvement Program, SSC 94-03, Prepared by the Applied Technology Council, California Universities for Research in Earthquake Engineering, and the Structural Engineers Association of California; California Seismic Safety Commission, Sacramento, California.

CSSC, 1995, *Turning Loss to Gain: Northridge Earthquake Report to the Governor,* SSC 95-01, California Seismic Safety Commission, Sacramento, California.

Culver, C. G., 1976, *Survey Results for Fire and Loads in Office Buildings,* NBS Building Science Series Report 85, Center for Building Technology, National Bureau of Standards, Washington, D.C.

DSA, 1995, *Guidelines for the Seismic Retrofit of State Buildings,* Division of the State Architect, Sacramento, California.

Durrani, A. J., Y. Du, and Y. H. Luo, 1995, "Seismic Resistance of Nonductile Slab-Column Connections in Existing Flat-Slab Buildings," *ACI Structural Journal,* vol. 92, no. 4, pp. 479-487.

EERI, 1994, *Expected Seismic Performance of Buildings,* Publication SP-10. Ad Hoc Committee on Seismic Performance, Earthquake Engineering Research Institute, Oakland, California.

Ferguson, P. M., J. E. Breen, J. O. Jirsa, 1988, *Reinforced Concrete Fundamentals,* Wiley and Sons, Inc., New York, New York.

Frankel, A., C. Mueller, T. Barnhard, D. Perkins, E. V. Leyendecker, N. Dickman, S. Hanson, and M. Hopper, 1996, *Interim National Seismic Hazard Maps: Documentation,* United States Geological Survey (http://gldage.cr.usgs.gov), Draft.

Freeman, S. A., J. P. Nicoletti, and J. V. Tyrell, 1975, "Evaluations of Existing Buildings for Seismic Risk: A Case Study of Puget Sound Naval Shipyard, Bremerton, Washington." Proceedings of the U.S. National Conference of Earthquake Engineers, pp. 113-122, Earthquake Engineering Research Institute, Berkeley, California.

French, C., and J. P. Moehle, 1991, "Effect of Floor Slab on Behavior of Beam-Slab-Column Connections," *Design of Beam-Column Joints for Seismic Resistance,* ACI Special Publication 123, American Concrete Institute, Detroit, Michigan.

Gazetas, G., 1991, "Foundation Vibrations," *Foundation Engineering Handbook*, Ed. Fang.

Gentry, T. R., and J. K. Wight, 1994, "Wide Beam-Column Connections Under Earthquake-Type Loading," *Earthquake Spectra*, EERI, vol. 10, no. 4, pp. 675-703.

Goel, R. K., and A. K. Chopra, 1991, "Inelastic Seismic Response of One-Storey, Asymmetric-Plan Systems: Effects of System Parameters and Yielding," *Earthquake Engineering & Structural Dynamics*, vol. 20, no. 3, pp. 201-222.

Graf, W. P., and M. Mehrain, 1992, "Analysis and Testing of a Flat Slab Concrete Building," Tenth World Conference on Earthquake Engineering, Vol. 6, Madrid, Spain.

Hawkins, N. M., 1980, "Lateral Load Design Considerations for Flat Plate Structures," Proceedings, CSCE-ASCE-ACI-CEB International Symposium on Nonlinear Design of Concrete Structures, University of Waterloo Press, Waterloo, Ontario, pp. 581-613.

Hwang, S.-J., and J. P. Moehle, 1993, *An Experimental Study of Flat-Plate Structures Under Vertical and Lateral Loads,* UCB/EERC-93/03, University of California, Earthquake Engineering Research Center, Berkeley, California.

ICBO, 1994, *Uniform Building Code,* International Conference of Building Officials, Whittier, California.

ICBO, 1996, *Code Change Item 178 - Committee Action and Challenge 1,* Building Standards, Part III, pp. 148-172, International Conference of Building Officials, Whittier, California.

Joh, O., Y. Goto, and T. Shibata, 1991, "Behavior of Reinforced Concrete Beam-Column Joints With Eccentricity." *Design of Beam-Column Joints for Seismic Resistance*, James O. Jirsa, ed, ACI Special Publication 123, American Concrete Institute, Detroit, Michigan.

Kitayama, K., S. Otani, and H. Aoyama, 1991, "Development of Design Criteria for Reinforced Concrete Interior Beam-Column Joints," *Design of Beam-Column Joints for Seismic Resistance*, James O. Jirsa, ed, ACI Special Publication 123, American Concrete Institute, Detroit, Michigan.

Liao, S. S., D. Veneziano, and R. V. Whitman, 1988, "Regression Models for Evaluating Liquefaction Probability," *Journal of the Geotechnical Engineering Division*, ASCE, vol. 114, no. 4, pp. 389-411.

Luke, P. C., C. Chon, and J. O. Jirsa, 1985, "Use of Epoxies for Grouting Reinforcing Bar Dowels in Concrete," *ACI Structural Journal.*

Luo, Y., A. J. Durrani, and J. P. Conte, 1994, "Equivalent Frame Analysis of Flat Plate Buildings for Seismic Loading," *Journal of Structural Engineering*, vol. 120, no. 7, pp. 2137-2155.

Lynn, A. C., J. P. Moehle, and S. A. Mahin, 1995, "Evaluation of Existing Reinforced Concrete Building Columns" *EERC Newsletter.*

Mahaney, J. A., T. F. Paret, B. E. Kehoe, and S. A. Freeman, 1993, "The Capacity Spectrum Method for Evaluating Structural Response During the Loma Prieta Earthquake," National Earthquake Conference, Memphis.

Martin, G. R., and I. P. Lam, 1995, "Seismic Design of Pile Foundations: Structural and Geotechnical Issues," Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri.

Martin, G. R., and L. Yan, 1995, "Modeling Passive Earth Pressure for Bridge Abutments," ASCE Conference: Earthquake Induced Movements and Seismic Remediation of Existing Foundations and Abutments, Geotechnical Special Publication 55, San Diego, California.

Martinez Cruzado, J. A., A. N. Qaisrani, and J. P. Moehle, 1994, "Post-Tensioned Flat Plate Slab-Column Connections Subjected to Earthquake Loading," Fifth U.S. National Conference on Earthquake Engineering, Chicago, Illinois.

Mirza, S. A., and J. G. MacGregor, 1979, "Variability of Mechanical Properties of Reinforcing Bars," Proceedings, ASCE, vol. 105, ST5, pp. 921-937.

Moehle, J. P., 1992, "Displacement-Based Design of RC Structures Subjected to Earthquakes," *Earthquake Spectra*, EERI, vol. 8, no. 3, pp. 403-428.

NAVFAC, 1986, *Foundations and Earth Structures,* NAVFAC DM-7.02, Naval Facilities Engineering Command, Department of the United States Navy, Alexandria, Virginia.

NIBS, 1996, *Development of a Standardized Earthquake Loss Estimation Methodology,* 100% draft technical manual, Prepared by RMS, Inc., for the National Institute of Building Sciences/Federal Emergency Management Agency, Washington, D.C..

Newmark, N. M., and W. J. Hall, 1982, *Earthquake Spectra and Design,* EERI Monograph Series, Earthquake Engineering Research Institute, Oakland, California.

Otani, S., 1980, "Nonlinear Dynamic Analysis of Reinforced Concrete Building Structures," *Canadian Journal of Civil Engineering*, vol. 7, no. 2, pp. 333-344.

Otani, S., 1991, "The Architectural Institute of Japan Proposal of Ultimate Strength Design Requirements for RC Buildings With Emphasis on Beam-Column Joints," *Design of Beam-Column Joints for Seismic Resistance*, ACI Special Publication 123, American Concrete Institute, Detroit, Michigan.

Otani, S., and K.-N. Li, 1984, "Kurayoshi City East Building Damaged by the 1983 Tattori Earthquake," Proceedings, Second Workshop on Seismic Performance of Existing Buildings, Cornell University, July 1989.

Otani, S., T. Kabeyasawa, H. Shiohara, and H. Aoyama, 1985, "Analysis of the Full Scale Seven Story Reinforced Concrete Test Structure," *Earthquake Effects on Reinforced Concrete Structures: US-Japan Research,* ACI Special Publication 84, pp. 203-240, American Concrete Institute, Farmington Hills, Michigan.

Otani, S., S. Nagai, and H. Aoyama, 1994, *Load-Deformation Relations of High-Strength Reinforced Concrete Beams,* University of Tokyo, Tokyo, Japan.

Pan, A. D., and J. P. Moehle, 1992, "An Experimental Study of Slab-Column Connections," *ACI Structural Journal*, vol. 89, no. 6, pp. 626-638.

Pan, A., J. P. Moehle, 1989, "Lateral Displacement Ductility of Reinforced Concrete Flat Plates," *ACI Structural Journal*, vol. 86, no. 3, pp. 250-258.

Park, R., and T. Paulay, 1974 *Reinforced Concrete Structures,* John Wiley and Sons, New York, New York.

Paulay, T., 1971a, "Coupling Beams of Reinforced Concrete Shear Walls," *Journal of the Structural Division*, ASCE, vol. 97, no. 3, pp. 843-862.

Paulay, T., 1971b, "Simulated Seismic Loading of Spandrel Beams," *Journal of the Structural Division*, ASCE, vol. 97, no. 9, pp. 2407-2419.

Paulay, T., 1986, "The Design of Ductile Reinforced Concrete Structural Walls for Earthquake Resistance," *Earthquake Spectra*, EERI, vol. 2, no. 4, pp. 783-823.

Paulay, T., and M. J. N. Priestley, 1992, *Seismic Design of Reinforced Concrete and Masonry Buildings,* John Wiley and Sons, Inc., New York, New York.

Pessiki, S. P., C. H. Conley, P. Gergely, R. N. White, 1990, *Seismic Behavior of Lightly Reinforced Concrete Column and Beam-Column Joint Details,* NCEER-90-0014, National Center for Earthquake Engineering Research, Buffalo, New York.

Powell, G. H., 1996a, Additional Comments on the Report "Recommended Methodology for Seismic Evaluation and Retrofit of Existing Concrete Buildings," Personal Correspondence (Unpublished), 325 Goshen Court, San Ramon, California.

Powell, G. H., 1996b, Comments on the Report "Recommended Methodology for Seismic Evaluation and Retrofit of Existing Concrete Buildings," Personal Correspondence (Unpublished), 325 Goshen Court, San Ramon, California.

Powell, G. H. et al., 1992, *DRAIN-2DX Version 1.00,* Department of Civil Engineering, University of California, Berkeley, California.

Priestley, M. J. N., R. Verma, and Y. Xiao, 1994, "Seismic Shear Strength of Reinforced Concrete Columns," *Journal of Structural Engineering*, vol. 120, no. 8, pp. 2310-2329.

Qi, X., and J. P. Moehle, 1991, *Displacement Design Approach for RC Structures Subjected to Earthquakes,* UCB/EERC-91/02, University of California, Earthquake Engineering Research Center, Berkeley, California.

Raffaelle, G. S., and J. K. Wight, 1995, "Reinforced Concrete Eccentric Beam-Column Connections Subjected to Earthquake-Type Loading," *ACI Structural Journal*, vol. 92, no. 1, pp. 45-55.

Reese, L. C., S. T. Wang, and P. H. F. Lam, 1994, *GROUP: A Computer Program for Analysis of a Group of Piles Subjected to Axial and Lateral Loading,* Ensoft Inc., Austin, Texas.

RISA Technologies (RISA), 1993, *RISA-2D: Rapid Interactive Structural Analysis-2 Dimensional,* RISA Technologies, Lake Forest, California.

Saatcioglu, M., 1995, "Hysteretic Response of Low-Rise Walls," *Concrete Shear in Earthquake*, Elsevier Applied Science, New York, New York, pp. 105-114.

San Francisco, City and County of., 1991, *City and County of San Francisco Municipal Code,* Seattle Book Publishing Company, Seattle, Washington.

Scott, R. F., 1981, *Foundation Analysis,* Prentice-Hall, Englewood Cliffs, New Jersey.

SEAOC, 1990, *Recommended Lateral Force Requirements and Commentary* (the Blue Book), Structural Engineers Association of California, Sacramento, California.

SEAOC, 1995a, *Recommended Guidelines for the Practice of Structural Engineering in California,* Structural Engineers Association of California, Sacramento, California.

SEAOC, 1995b, *Vision 2000 Progress Report*, Proceedings of 1995 Convention, Structural Engineers Association of California, San Francisco, California.

Sedarat, H., and V. V. Bertero, 1990, "Effects of Torsion on the Nonlinear Inelastic Seismic Response of Multi-Story Structures" *Proceedings, Fourth U.S. National Conference on Earthquake Engineering*, Earthquake Engineering Research Institute, El Cerrito, California, vol. 2, pp. 421-430.

Seneviratna, G. D., and H. Krawinkler, 1994, "Strength and Displacement Demands for Seismic Design of Structural Walls" *Proceedings of the Fifth U.S. National Conference on Earthquake Engineering*, Chicago, Illinois.

SHBCB, 1993, *California Historical Building Code, Title 24, Part 8, CCR,* State Historical Building Code Board, International Conference of Building Officials, Whittier, California.

Sheikh, S. A., 1982, "A Comparative Study of Confinement Models," *Journal of the American Concrete Institute*, vol. 79, no. 4, Title 79-30, pp. 296-306.

Shibata, A., and M. A. Sozen, 1976, "Substitute-Structure Method for Seismic Design in R/C," *Journal of the Structural Division*, ASCE, vol. 102, no. 1, pp. 1-18.

Somerville, P., 1996, "Forward Rupture Directivity in the Kobe and Northridge Earthquakes, and Implications for Structural Engineers," *Proceedings, Seventh U.S.-Japan Workshop on Improvement of Structural Design and Construction Practices: Lessons Learned from Kobe and Northridge*, Kobe, Japan; Applied Technology Council.

Sozen, M. A., and J. P. Moehle, 1993, *Stiffness of Reinforced Concrete Walls Resisting In-Plane Shear*, EPRI TR-102731, Electric Power Research Institute, Palo Alto, California.

Spacone, E., V. Ciampi, and F. C. Filippou, 1992, *A Beam Element for Seismic Damage Analysis,* UCB/EERC-92/07, University of California, Earthquake Engineering Research Center, Berkeley, California.

Thomsen, J. H. IV, and J. W. Wallace, 1995, *Displacement-Based Design of RC Structural Walls: An Experimental Investigation of Walls With Rectangular and T-Shaped Cross-Sections,* CU/CEE-95/06, Clarkson University, Department of Civil and Environmental Engineering, Structural Engineering Research Lab, Potsdam, New York.

Vanderbilt, M. D., and W. G. Corley, 1983, "Frame Analysis of Concrete Buildings," Concrete International, American Concrete Institute, pp. 33-43.

Vulcano, A., V. V. Bertero, and V. Colotti, 1989, "Analytical Modeling of R/C Structural Walls," *Proceedings, Ninth World Conference on Earthquake Engineering*, Japan Association for Earthquake Disaster Prevention, Tokyo, Japan, vol. VI, paper 9-1-7, pp. 41-46.

Wallace, J. W., and J. P. Moehle, 1992, "Ductility and Detailing Requirements for Bearing Wall Buildings," *Journal of Structural Engineering*, ASCE, vol. 118, no. 6, pp. 1625-1644.

Wilson, R. C., and D. K. Keefer, 1985, "Predicted Areal Limits of Earthquake-Induced Landsliding," in *Evaluating Earthquake Hazards in the Los Angeles Region*, United States Geological Survey Professional Paper 1360, J. I. Ziony, ed. pp. 317-493.

Wood, S. L., 1990, "Shear Strength of Low-Rise Reinforced Concrete Walls," *ACI Structural Journal*, vol. 87, no. 1, pp. 99-107.

Wood, S. L., 1991, "Evaluation of Long-Term Concrete Properties," *ACI Materials Journal,* vol. 88, no. 6, pp. 630-643.

Yanez, F. V., R. Park, and T. Paulay, 1992, "Seismic Behavior of Walls With Irregular Openings," *Proceedings of the Tenth World Conference on Earthquake Engineering*: A. A., Balkema, Rotterdam, vol. 6, pp. 3303-3308.

Youd, T. L., and D. M. Perkins, 1978, "Mapping of Liquefaction-Induced Ground Failure Potential," *Journal of the Geotechnical Engineering Division*, ASCE, vol. 104, no. 4, pp. 433-446.