C2. General Requirements (Simplified and Systematic Rehabilitation)

C2.1 Scope

No commentary is provided for this section.

C2.2 Basic Approach

The basic steps that the rehabilitation design process comprises are indicated in this section. Prior to embarking on a rehabilitation design, it is necessary to understand whether the building, in its existing condition, is capable of meeting the intended Performance Levels. This requires that a preliminary evaluation of the building be performed. BSSC (1992a) is indicated as one potential guideline for performing such evaluations; however, it is noted that BSSC (1992a) does not directly address many of the Rehabilitation Objectives that are included within the scope of this document. One possible approach to performing a preliminary evaluation, in order to determine if rehabilitation is necessary to meet other Rehabilitation Objectives, would be to analyze the building, without corrective measures, using the methods contained in this document.

An important step in the design of rehabilitation measures is the development of a preliminary design. While the *Guidelines* provide information on alternative rehabilitation strategies that could be employed, they do not provide a direct methodology for arriving at a preliminary design. The general approach recommended is one of examining the deficiencies in the existing structure—relative to the acceptance criteria provided in the *Guidelines* for the desired Performance Level—in order to determine the principal requirements for additional strength, stiffness, or deformation capacity. A strategy should be selected that addresses these requirements in an efficient manner. Preliminary design must be made largely by trial and error, relying heavily on the judgment of the design engineer.

C2.3 Design Basis

The *Guidelines* provide uniform criteria by which existing buildings may be rehabilitated to attain a wide range of different Performance Levels, when subjected to earthquakes of varying severities and probability of occurrence. This is a unique approach, distinctly different from that presently adopted by building codes for new construction. In the building codes for new construction, building performance is implicitly set in a manner that is not transparent to the user. Therefore, the user frequently does not understand the level of performance to be expected of buildings designed to the code, should they experience a design event. Further, the user is not given a clear understanding of what design changes should be made in order to obtain performance different from that implicit in the codes. The *Guidelines* start by requiring that the user select specific performance goals, termed Rehabilitation Objectives, as a basis for design. In this way, users can directly determine the effect of different performance goals on the design requirements.

It is important to note that when an earthquake does occur, there can be considerable variation in the levels of performance experienced by similar buildings located on the same site, and therefore apparently subjected to the same earthquake demands. This variability can result from a number of factors, including random differences in the levels of workmanship, material strength, and condition of each structure, the amount and distribution of live load present at the time of the earthquake, the influence of nonstructural components present within each structure, the response of the soils beneath the buildings, and relatively minor differences in the character of the ground motion transmitted to the structures. Many of these factors cannot be completely identified or quantified at our current level of understanding and capability.

It is the intent of the *Guidelines* that most, although not necessarily all, structures designed to attain a given performance at a specific earthquake demand would exhibit behavior superior to that predicted. However, there is no guarantee of this. There is a finite possibility that—as a result of the variances described above, and other factors—some rehabilitated buildings would experience poorer behavior than that intended by the Rehabilitation Objective.

The concept of redundancy is extremely important to the design of structures for seismic resistance, in that it is expected that significant damage to the structural elements can occur as a result of building response to severe ground motion. In a redundant structure, multiple elements (or components) will be available to resist forces induced by such response. Should one or more of these elements fail, or become so badly damaged that they are no longer effective in providing structural resistance, additional elements are available to prevent loss of stability. In a nonredundant structure, failure of one or two elements can result in complete loss of lateral resistance, and collapse.

In many structures, nearly all elements and components of the building participate in the structure's lateral-load-resisting system, to some extent. As the structure is subjected to increasing lateral demands, some of these elements may begin to fail and lose strength much sooner than others. If a structure has sufficient redundancy, it may be permissible to allow failure of some of these elements, as long as this does not result in loss of gravity load-carrying capacity or overall lateral stability. The *Guidelines* introduce the concept of "primary" and "secondary" elements in order to allow designers to take advantage of the inherent redundancy in some structures, and to permit a few selected elements of the structure to experience excessive damage rather than requiring massive rehabilitation programs to prevent such damage.

Any element in a structure may be designated as a secondary element, so long as expected damage to the element does not compromise the ability of the structure to meet the intended performance levels. Secondary elements are assumed to have minimal effective contribution to the lateral-force-resisting system. When linear analysis procedures are used, secondary elements are not typically modeled as part of the system, or if they are, they are modeled at greatly reduced stiffnesses, simulating their anticipated stiffness degradation under large lateral response. Primary elements must remain effective in resisting lateral forces, in order to provide the basic stability of the structure.

For some structures, it may be possible to determine at the beginning of the design process which elements should be classified as primary or secondary. For other, more complex structures, it may be necessary to perform initial evaluations assuming all elements are primary. If some of the elements cannot meet the applicable acceptance criteria, or have demands that exceed their acceptance criteria by substantially greater margins then other elements, these could be designated as secondary, and the analysis repeated with the model altered to remove the stiffness contribution of these elements. If too many elements are designated as secondary, the structure's ability to resist the required demands will be impaired, indicating that additional rehabilitation measures are required.

C2.4 Rehabilitation Objectives

The Rehabilitation Objective(s) selected for a project are an expression of the desired building behavior when it experiences earthquake effects of projected severity. In the *Guidelines*, selection of a Rehabilitation Objective controls nearly all facets of the design process, including the characterization of earthquake demands, the analytical techniques that may be used to predict building response to these demands, and the acceptance criteria (strength and deformability parameters) used to judge the design's adequacy.

In the *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings: 1994 Edition* (BSSC, 1995), three different design performance objectives are implicitly set, based on the building's intended occupancy. Most buildings are contained within Seismic Hazard Exposure Group I, for which a basic design objective of minimizing the hazard to life safety is adopted. For high-occupancy buildings, contained in Seismic Hazard Exposure Group II, the same performance objective is set, but with a higher degree of reliability. Buildings that contain occupancies essential to post-disaster response are grouped within Seismic Hazard Exposure Group III, for which a design objective of post-earthquake functionality is set. The Seismic Hazard Exposure Group together with the site seismicity determine the building's Seismic Performance Category and, therefore, the permissible structural systems, the analytical procedures that may be employed, the types of structural detailing that must be incorporated, and the design requirements for nonstructural components.

In the formation of the *Guidelines*, it was felt that a rigid requirement to upgrade all buildings to the performance objective corresponding with their Seismic Hazard Exposure Group in the *NEHRP Recommended Provisions* would be prohibitively expensive; could result in extensive demolition of structures that are valuable cultural, societal and historic resources; or alternatively, would achieve no improvement in the public safety, through a lack of implementation. It was also recognized that there are a number of owners who desire better seismic performance for individual structures than is provided for in the corresponding

Seismic Hazard Exposure Group of the BSSC (1995) provisions. Therefore, the *Guidelines* adopt a flexible approach with regard to selection of Rehabilitation Objectives. For each building, a decision must be made as to the acceptable behavior for different levels of seismic hazard, balanced with the cost of rehabilitating the structure to obtain that behavior. For many buildings, multiple rehabilitation objectives will be adopted—ranging from negligible damage and occupancy interruption for earthquake events with a high probability of occurrence, to substantial damage but protection of life safety for events with a low probability of occurrence. Figure C2-1 summarizes the various Rehabilitation Objectives available to users of the *Guidelines*. BSE-1 is the Basic Safety Earthquake 1; BSE-2, the more severe ground motion defined with regard to the Basic Safety Objective (BSO), is Basic Safety Earthquake 2.

In general, Rehabilitation Objectives that expect relatively low levels of damage for relatively infrequent earthquake events will result in more extensive rehabilitation work and greater expense than objectives with more modest goals of controlling damage. Figure C2-2 schematically presents the relationship between different Rehabilitation Objectives and probable program cost. VSP (1992), *A Benefit-Cost Model for the Seismic Rehabilitation of Buildings* provides a methodology for evaluation of the costs and benefits of seismic rehabilitation.

The formation of project Rehabilitation Objectives requires the selection of both the target Building Performance Levels and the corresponding earthquake hazard levels for which they are to be achieved. Hazard levels may be selected on either a probabilistic or deterministic basis and may be selected at any level of severity. This is also a significant departure from the practice adopted in building codes for new construction.

C2.4.1 Basic Safety Objective

Rehabilitation design for the Basic Safety Objective (BSO) under the *Guidelines* is expected to produce earthquake performance similar—but not identical—to that desired for new buildings in Seismic Hazard Exposure Group I of BSSC (1995). Buildings that are rehabilitated for the BSO will in general present a low level of risk to life safety at any earthquake demand level likely to affect them. However, some potential for life safety endangerment at the extreme levels of demand that can occur at the site will remain. In addition, buildings rehabilitated to these Performance Levels may also have significant potential for extreme damage and total economic loss when subjected to relatively infrequent but severe earthquake events. To the extent that it is economically feasible, all buildings should be rehabilitated to meet this objective, as a minimum.

The *Guidelines* specify a two-level design check (Life Safety Performance Level for BSE-1 demands and Collapse Prevention Performance Level for BSE-2 demands) in order to design for the BSO. This is in contrast to the BSSC (1995) provisions, which employ only a single level design check. The BSSC (1995) provisions can adopt the single level design approach because for new structures it is possible to control the ductility and configuration of the design to an extent that will permit those structures designed to achieve the Life Safety Performance Level for a 10%/50 year event to also avoid collapse for much larger events. Existing buildings have not generally been constructed with the same controls on configuration and detailing, and therefore may not have comparable capacity to survive stronger earthquake demands, even when rehabilitated. Therefore, it was considered prudent to explicitly require evaluation of the rehabilitated structure for its capacity to resist collapse when subjected to extreme earthquake demands.

The *Guidelines* permit individual building officials to declare, or deem, that buildings in compliance with the 1994 or later editions of the *Uniform Building Code* (ICBO, 1994) or *Standard Building Code* (SBCCI, 1994), or with the 1993 edition of the *National Building Code* (BOCA, 1993) meet the requirements of the BSO. This was done recognizing that the *Guidelines* represent new technology which would in some cases provide different results than would the provisions of current model codes, and to avoid the problem of creating a class of hazardous buildings comprising newly constructed, code-compliant structures. Buildings that have been adequately designed and constructed in conformance with the provisions of the 1994 *Uniform Building Code* for seismic zones 3 and 4, or with the provisions of the 1993 *National Building Code* or 1994 *Standard Building Code* for Seismic Performance Categories D or E, should, in actuality, meet or exceed the BSO. However, buildings designed for lower seismic zones or performance categories, or that have not been adequately designed and constructed in conformance with the code provisions, may not be able to meet the technical requirements or performance expectations of the BSO. It is anticipated that buildings

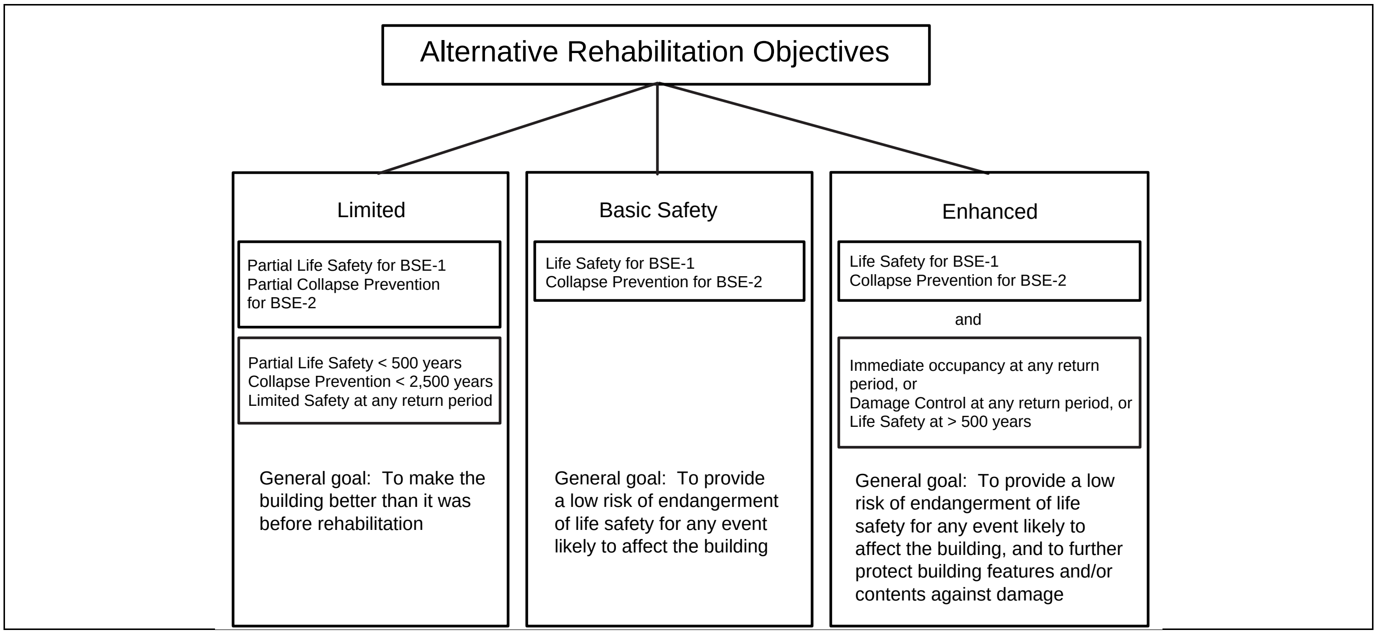

Figure C2-1 Rehabilitation Objectives

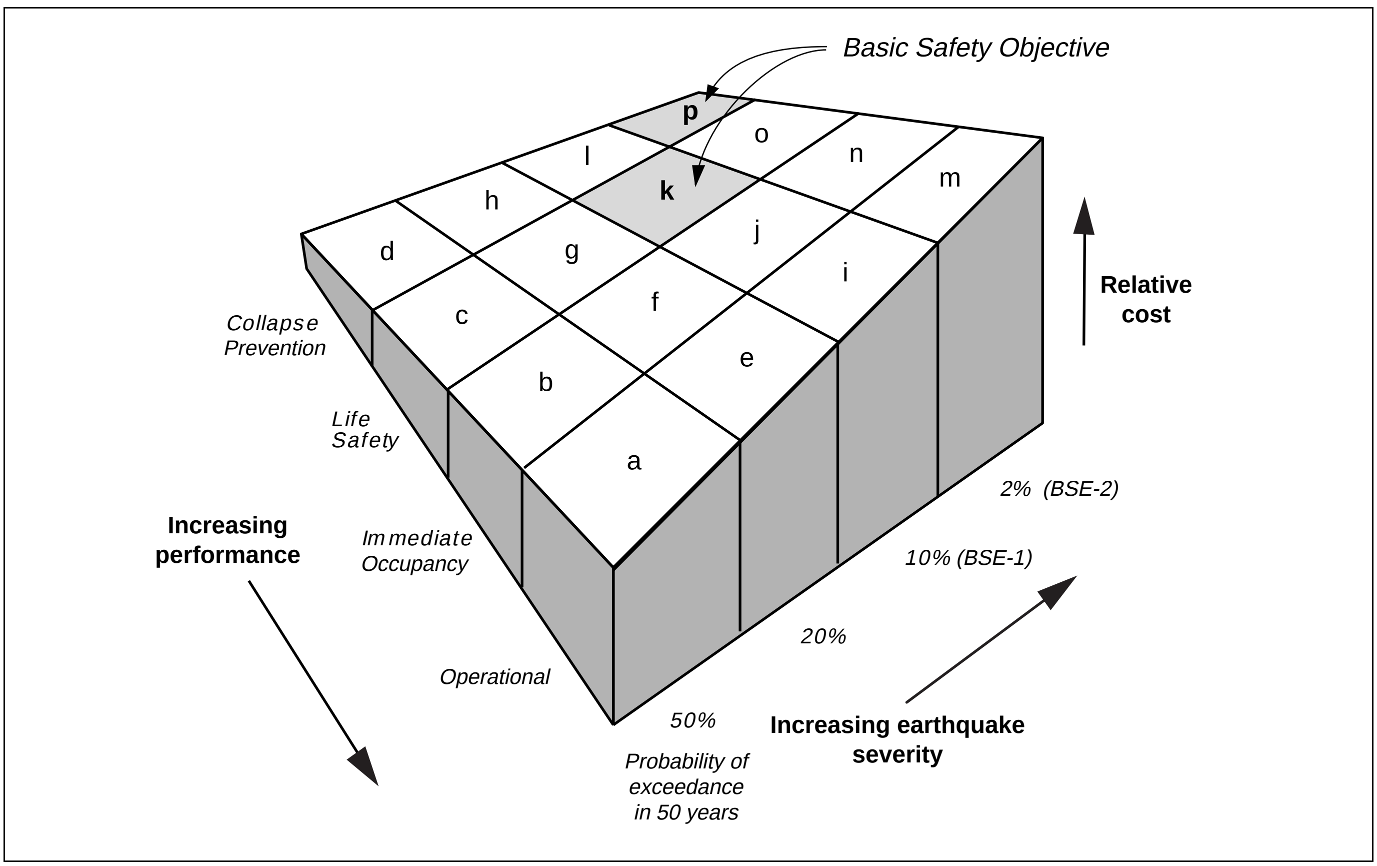

Figure C2-2 Surface Showing Relative Costs of Various Rehabilitation Objectives

meeting code provisions based on seismic design criteria contained in the *NEHRP Recommended Provisions* (BSSC, 1997) would be able to meet or exceed the BSO regardless of the seismic zone or performance category ("Seismic Design Category" in the 1997 *NEHRP Provisions*) for which they have been designed.

C2.4.2 Enhanced Rehabilitation Objectives

Individual agencies and owners may elect to design to Rehabilitation Objectives that provide for lower levels of damage than anticipated for buildings rehabilitated to the BSO. Benefits of such rehabilitation are potential reductions of damage repair costs and loss of facility use, as well as greater confidence in the protection of life safety.

There are many buildings for which the levels of damage that may be sustained under the BSO will be deemed inappropriate. These may include buildings in NEHRP Seismic Hazard Exposure Group III as defined in the 1994 *NEHRP Provisions* (BSSC, 1995)—such as hospitals, fire stations, and similar facilities critical to post-earthquake disaster response and recovery—as well as buildings housing functions critical to the economic welfare of business concerns, such as data processing centers and critical manufacturing facilities. It may be desirable that such buildings be available to perform their basic functions shortly after an earthquake occurs. Designing to the Immediate Occupancy Performance Level, or to a custom level within the Damage Control Performance Range, at an appropriate earthquake hazard level, provides an opportunity to achieve such performance.

The importance of maintaining operations or controlling damage within an individual building should be considered in selecting an appropriate Rehabilitation Objective to use in the rehabilitation design. For buildings in NEHRP Seismic Hazard Exposure Group III, Performance Levels consisting of Immediate Occupancy for BSE-1 and Life Safety for BSE-2 demands could be considered as a basis for design. Buildings designed to such objectives will in general present a low level of risk that the buildings could not be occupied at any earthquake demand level likely to affect them, and a very low risk of life safety endangerment. However, it is not intended that structures designed to these Rehabilitation Objectives would behave so well that no interruption in their service occurs. Some cleanup and repair may be required in order to restore such structures to service; however, it is intended that such activities can be quickly accomplished.

For buildings contained in NEHRP Seismic Hazard Exposure Group II, and for buildings in critical business occupancies, Rehabilitation Objectives consisting of Damage Control Performance Range for 10%/50 year earthquake demands and Life Safety Performance Level for MCE demands should be considered. Buildings rehabilitated to such objectives would have a low level of risk of long-term occupancy interruption resulting from earthquake damage, as well as a very low level of risk of life safety endangerment.

It is important to note that mere provision of structural integrity does not ensure that buildings housing critical functions will be operable immediately following an earthquake. In addition to damage control, functionality following an earthquake typically requires electric power, as well as other utilities. Facilities that must remain in service in the immediate post-earthquake period should be provided with reliable standby utilities to service their essential systems. In addition, critical equipment within the facilities should be safeguarded to ensure functionality. Discussions of these requirements are contained in Chapter 11 on nonstructural components.

The determination as to whether a project should be designed to Enhanced Rehabilitation Objectives, and if so, which Performance Levels should be coupled with which earthquake demand levels, largely depends on the acceptable level of risk for the facility. Cost-benefit analysis may be a useful tool for establishing an appropriate Enhanced Rehabilitation Objective for many facilities.

C2.4.3 Limited Rehabilitation Objectives

Limited Rehabilitation provides for seismic rehabilitation to reliability levels that are lower than the BSO. It is included in the *Guidelines* to provide a method for owners and agencies with limited economic resources to obtain a reduction in their existing seismic risk, rather than doing nothing. Rehabilitation to objectives that do not meet the BSO may be selected by individual agencies or owners when it is deemed economically impractical to design for the BSO. The usual intent of such rehabilitation is to achieve highly cost-effective improvement in the probable earthquake performance of the building. Two types of Limited Rehabilitation Objectives are included.

C2.4.3.1 Partial Rehabilitation

Partial Rehabilitation is rehabilitation that addresses only a portion of the building. The typical goal of Partial Rehabilitation is to reduce the specific risks related to one or more common or particularly severe vulnerabilities, without addressing the building's complete lateral-force-resisting system or all nonstructural components. It is recommended that Partial Rehabilitation Objectives be identical to those for the BSO. In this way, partial rehabilitation may be implemented as one of a series of incremental rehabilitation measures that, when taken together, achieve full rehabilitation of the building to the BSO. Alternatively, other Rehabilitation Objectives could be selected as the basis for partial rehabilitation.

C2.4.3.2 Reduced Rehabilitation

Reduced Rehabilitation Objectives address the entire structure; however, they permit greater levels of damage, at more probable levels of ground motion, than is permitted under the BSO. Reduced Rehabilitation Objectives permit owners with limited resources to reduce the levels of damage in the more moderate events likely to occur with relative frequency over the building's life. These objectives may be most appropriate for buildings with limited remaining years of life or with relatively low or infrequent occupancies.

C2.5 Performance Levels

Building performance in these *Guidelines* is expressed in terms of Building Performance Levels. These Building Performance Levels are discrete damage states selected from among the infinite spectrum of possible damage states that buildings could experience as a result of earthquake response. The particular damage states identified as Building Performance Levels in these *Guidelines* have been selected because these Performance Levels have readily identifiable consequences associated with the post-earthquake disposition of the building that are meaningful to the building user community. These include the ability to resume normal functions within the building, the advisability of post-earthquake occupancy, and the risk to life safety.

Although a building's performance is a function of the performance of both structural systems and nonstructural components and contents, these are treated independently in the *Guidelines*, with separate Structural and Nonstructural Performance Levels defined. Each Building Performance Level comprises the individual Structural and Nonstructural Performance Levels selected by the design team. This subcategorization of building performance into separate structural and nonstructural components was adopted in the *Guidelines* because building owners have frequently approached building rehabilitation projects in this manner. Historically, many building owners have performed seismic rehabilitation projects that concentrated effort in the improvement of the structural performance capability of the building without addressing nonstructural vulnerabilities. Such owners typically believed that if the building performance could be controlled to provide limited levels of structural damage, damage to nonstructural components could be dealt with in an acceptable manner. Many other owners have taken a directly contrary approach, believing that it was most important to prevent damage to nonstructural building components, since such components have often been damaged in even relatively moderate earthquakes, resulting in costly business interruption. The approach taken by the *Guidelines* provides sufficient flexibility to accommodate either approach to building rehabilitation, as well as approaches that address structural and nonstructural vulnerabilities in a more balanced manner.

C2.5.1 Structural Performance Levels and Ranges

When a building is subjected to earthquake ground motion, a pattern of lateral deformations that varies with time is induced into the structure. At any given point in time, a particular state of lateral deformation will exist in the structure, and at some time within the period in which the structure is responding to the ground motion, a maximum pattern of deformation will occur. At relatively low levels of ground motion, the deformations induced within the building will be limited, and the resulting stresses that develop within the structural components will be within the elastic range of behavior. Within this elastic range, the structure will experience no damage. All structural components will retain their original strength, stiffness, and appearance, and when the ground motion stops, the structure will return to its pre-earthquake condition.

At more severe levels of ground motion, the lateral deformations induced into the structure will be larger. As these deformations increase, so will demands on the individual structural components. At different levels of deformation, corresponding to different levels of ground motion severity, individual components of the structure will be strained beyond their elastic range. As this occurs, the structure starts to experience damage in the form of cracking, spalling, buckling, and yielding of the various components. As components become damaged, they degrade in stiffness, and some elements will begin to lose their strength. In general, when a structure has responded to ground motion within this range of behavior, it will not return to its pre-earthquake condition when the ground motion stops. Some permanent deformation may remain within the structure and damage will be evident throughout. Depending on how far the structure has been deformed, and in what pattern, the structure may have lost a significant amount of its original stiffness and, possibly, strength.

Brittle elements are not able to sustain inelastic deformations and will fail suddenly; the consequences may range from local and repairable damage to collapse of the structural system. At higher levels of ground motion, the lateral deformations induced into the structure will strain a number of elements to a point at which the elements behave in a brittle manner or, as a result of the decreased overall stiffness, the structure loses stability. Eventually, partial or total collapse of the structure can occur. The Structural Performance Levels and Ranges used in the *Guidelines* relate the extent of a building's response to earthquake hazards to these various possible damage states.

Figure C2-3 illustrates the behavior of a ductile structure as it responds with increasing lateral deformation. The figure is a schematic plot of the lateral force induced in the structure as a function of lateral deformation. Three discrete points are indicated, representing the discrete Performance Levels: Immediate Occupancy, Life Safety, and Collapse Prevention.

At the Immediate Occupancy Level, damage is relatively limited. The structure retains a significant portion of its original stiffness and most if not all of its strength. At the Collapse Prevention Level, the building has experienced extreme damage. If laterally deformed beyond this point, the structure can experience instability and collapse. At the Life Safety Level, substantial damage has occurred to the structure, and it may have lost a significant amount of its original stiffness. However, a substantial margin remains for additional lateral deformation before collapse would occur.

Specifically, it is intended that structures meeting the Life Safety Level would be able to experience at least 33% greater lateral deformation (minimum margin of 1.33) before failure of primary elements of the lateral-force-resisting system and significant potential for instability or collapse would be expected. As indicated in the *Commentary* to the *NEHRP Recommended Provisions* (BSSC, 1997), significantly better performance is expected of new structures when subjected to their design earthquake ground motions. Such structures are anticipated to provide a margin of at least 1.5 against collapse at the design earthquake level. Lower margins were specifically selected for the Life Safety Performance Level under the *Guidelines* to be consistent with historic practice that has accepted higher levels of risk for existing structures, based largely on economic considerations.

It should be noted that for given buildings the relative horizontal and vertical scales shown on this plot may vary significantly, and the margin of deformation between individual performance levels may not be as large as indicated in this figure. Figure C2-4 is a similar curve, representative of the behavior of a nonductile, or brittle, structure. Note that for such a structure, there may be relatively little margin in the response that respectively defines the three performance levels.

For a given structure and design earthquake, it is possible to estimate the overall deformation and force demand on the structure and, therefore, the point on the corresponding curves shown in Figures C2-3 or C2-4 to which the earthquake will push the building. This either will or will not correspond to the desired level of performance for the structure. When structural/seismic rehabilitation is performed, modifications to the structure are made to alter its strength, stiffness, or ability to dampen or resist induced deformations. These actions will alter the characteristics of both the shape of the curves in these figures and the deformation demand produced by the design earthquake on the building, such that the expected performance at the estimated deformation level for the rehabilitated structure is acceptable.

In addition to the three performance levels, two performance ranges are defined in the *Guidelines* to allow users greater flexibility in selecting design Rehabilitation Objectives. Specific design parameters for use in designing within these ranges are not provided. The Damage Control Performance Range represents all those behavior states that occur at lower

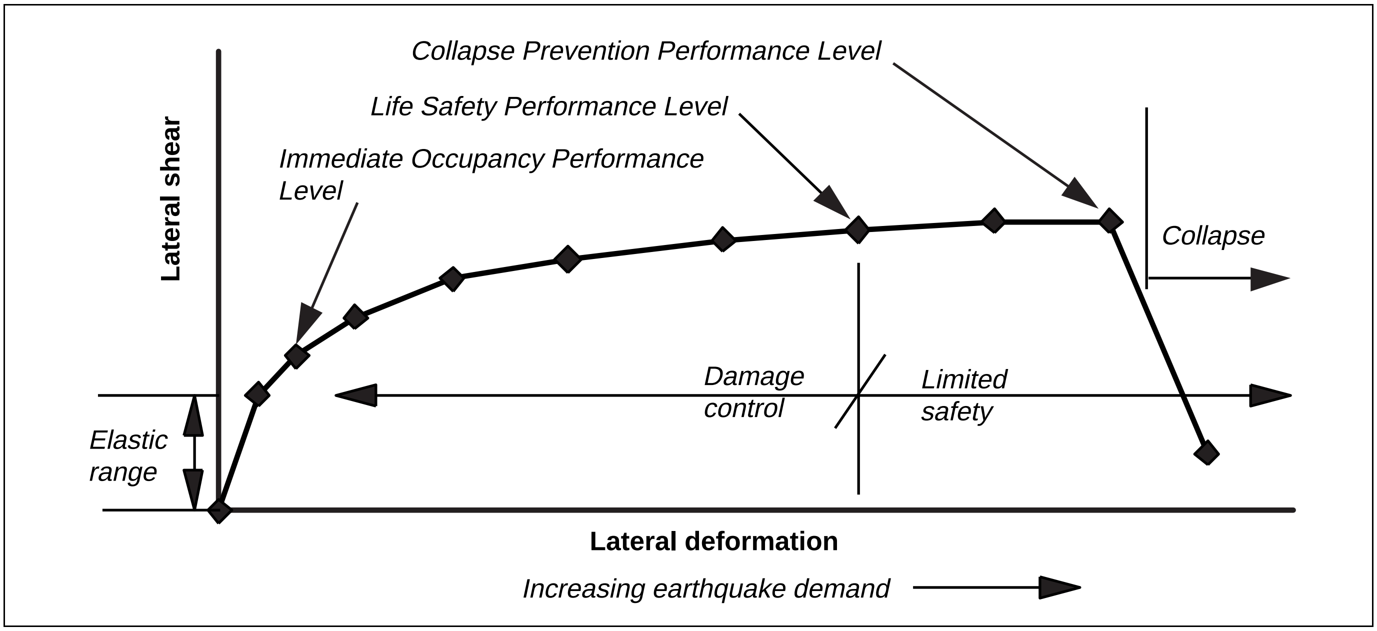

Figure C2-3 ***Performance and Structural Deformation Demand for Ductile Structures***

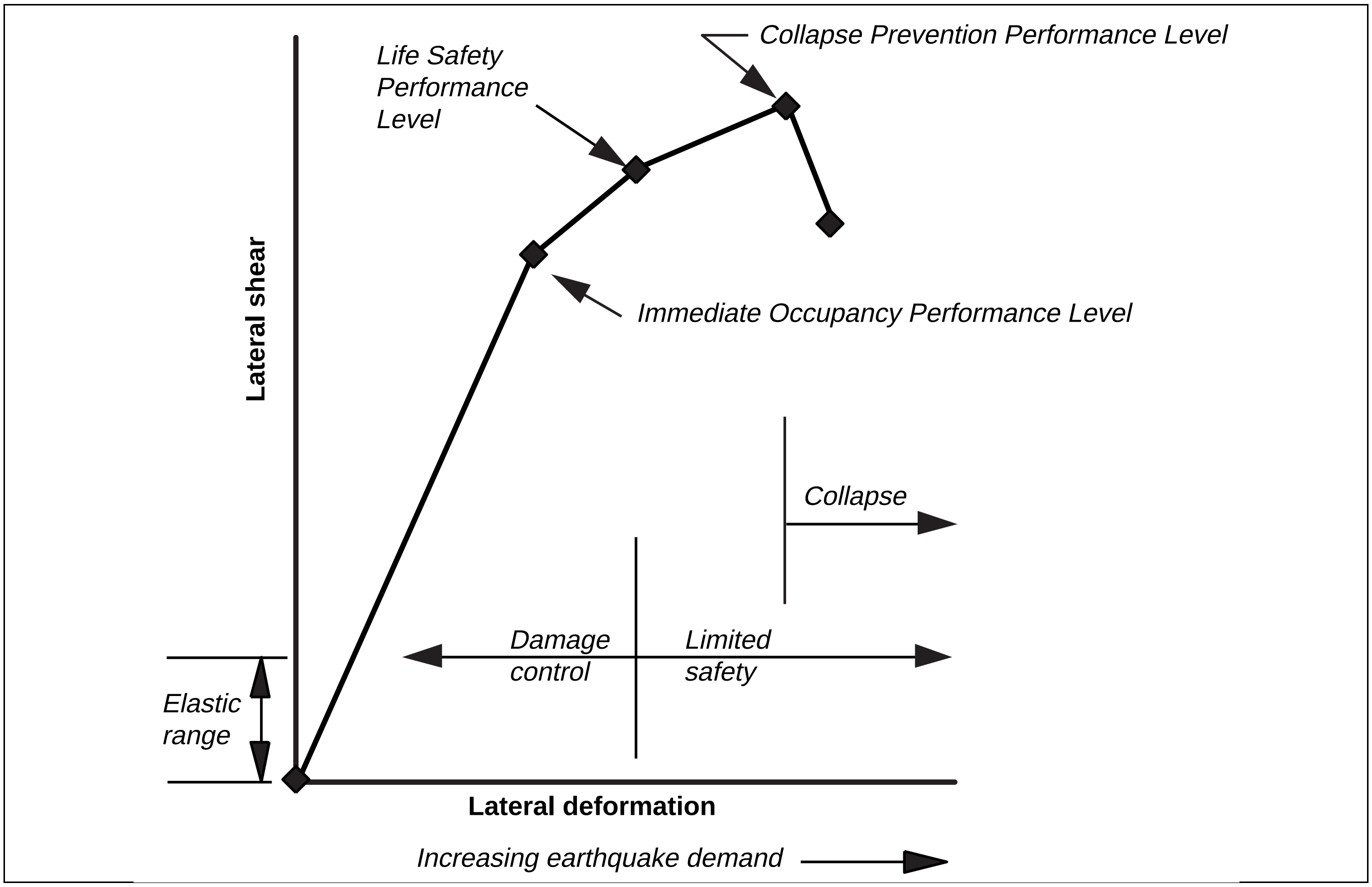

Figure C2-4 ***Performance and Structural Deformation Demand for Nonductile Structures*** levels of lateral deformation than that defined for Life Safety. At the lower levels of deformation contained within this range, the structure would behave in a predominantly elastic manner. At upper levels of deformation within this range, the structure may experience significant inelastic behavior. In general, the more inelastic behavior the structure experiences, the greater the extent of structural damage expected.

The Limited Safety Performance Range of behavior includes all those behavior states that occur at lateral deformation levels in excess of the Life Safety Performance Level, including, possibly, collapse states. Designing for performance within the Limited Safety Range may imply a significant risk of life and economic loss.

C2.5.2 Nonstructural Performance Levels

Nonstructural Performance Levels define the extent of damage to the various nonstructural components included in a building, such as electrical, mechanical, plumbing, and fire protection systems; cladding, ceilings, and partitions; elevators, lighting, and egress; and various items of tenant contents such as furnishings, computer systems, and manufacturing equipment. Although structural engineers typically have relatively little input to the design of these items, the way in which they perform in an earthquake can significantly affect the operability and even fitness for occupancy of a building following an earthquake. Even if a building's structure is relatively undamaged, extensive damage to lights, elevators, and plumbing and fire protection equipment could render a building unfit for occupancy.

There are three basic issues related to the performance of nonstructural components. These are:

- Security of component attachment to the structure and adequacy to prevent sliding, overturning, or dislodging from the normal installed position
- Ability of the component to withstand earthquake-induced building deformations without experiencing structural damage or mechanical or electrical fault
- Ability of the component to withstand earthquake-induced shaking without experiencing structural damage or mechanical or electrical fault

Until recently, the building codes for new construction were generally silent on the issue of how to design nonstructural components for seismic performance. Even in contemporary codes, the consideration of nonstructural performance is generally limited to the security of attachment of components to the structure, specifically with regard to the protection of occupant life safety. Consequently, widespread vulnerabilities of nonstructural components exist within the building inventory.

Mitigation of nonstructural seismic vulnerabilities is a complex issue. Many nonstructural components, if adequately secured to the structure, are seismically rugged. Further, retroactive provision of appropriate anchorage or bracing for some nonstructural components can be implemented very economically and without significant disruption of building function. However, mitigation of some vulnerabilities, such as provision of bracing for mechanical and electrical components within suspended ceiling systems, or the improvement of the ceiling systems themselves, can result in extensive disruption of occupancy and can also be quite costly.

C2.5.2.1 Operational Nonstructural Performance Level (N-A)

In designing for the Operational Nonstructural Performance Level, it will typically be necessary to secure all significant nonstructural components. Further, it will also be necessary to ensure that the components required for normal operation of the facility can function after being subjected to the displacements and forces transmitted by the structure. In order to obtain such assurance, it may be necessary to conduct tests of the behavior of prototype components on shaking tables, using motion that simulates that which would be transmitted to the component by the building structure. This is a tedious and extremely costly process that is beyond the economic capabilities of most owners. However, the nuclear industry has typically incorporated such procedures in the design of critical safety systems for their facilities.

C2.5.2.2 Immediate Occupancy Nonstructural Performance Level (N-B)

It will generally be more practical for most owners to design for the Immediate Occupancy Nonstructural Performance Level. At this level, all major nonstructural components are secured and prevented from sliding, toppling, or dislodging from their mountings. Since many nonstructural components are structurally rugged, it would be expected that most would be in an operable condition, assuming that the necessary power and other utilities are available. However, even attaining this level of nonstructural performance can be quite costly, as it may require modification of the installation of systems such as piping, ductwork, and ceilings throughout the building.

C2.5.2.3 Life Safety Nonstructural Performance Level (N-C)

The Life Safety Nonstructural Performance Level is obtained by structurally securing those nonstructural components that could pose a significant threat to life safety if they were to be dislodged by earthquake shaking. The primary difference between this level and that for Immediate Occupancy is that many small, lightweight components that are addressed under the Immediate Occupancy Performance Level are deemed not to be a significant life hazard and are not addressed under the Life Safety Performance Level. In addition, the Immediate Occupancy Performance Level requires somewhat more control of building lateral deflections than does the Life Safety Performance Level, in order to control to a somewhat greater degree the extent of damage resulting from in-plane deformation of elements such as cladding and partitions.

C2.5.2.4 Hazards Reduced Nonstructural Performance Level (N-D)

The Hazards Reduced Nonstructural Performance Level is similar to the Life Safety Performance Level except that the components that must be secured are limited to those that, if dislodged, would pose a major threat to life safety, capable of severely injuring a number of people. This would include elements such as parapets and exterior cladding panels. However, components such as individual light fixtures or HVAC ducts would not be addressed, nor would building deflections be limited as a method of controlling damage to items such as partitions and doors. This Performance Level provides for cost-effective mitigation of the most serious nonstructural hazards to life safety.

C2.5.2.5 Nonstructural Performance Not Considered (N-E)

No commentary is provided for this section.

C2.5.3 Building Performance Levels

No commentary is provided for this section.

C2.6 Seismic Hazard

Until the publication of ATC-3-06 (1978), the consideration of seismic hazards by the building codes was performed in a highly qualitative manner. The codes contained seismic hazard maps that divided the nation into a series of zones of equivalent seismicity. Until the mid-1970s, these maps contained four zones: (0) negligible seismicity, (1) low seismicity, (2) moderate seismicity, and (3) high seismicity. In the mid-1970s, zone 3 was further divided to produce another zone, zone 4, encompassing regions within 20 miles of major active faults. The classification of sites within the various zones was based on the historic seismicity of the region. If there were no historic reports of damaging earthquakes in a region it was classified as zone 0. If there were many large damaging earthquakes in an area, it was classified as zone 3, or later zone 4. Design force levels for structures were directly tied to the seismic zone in which a building was sited; however, these force levels were not correlated in any direct manner with specific ground motion spectra.

The ATC (1978) publication introduced the concept of acceleration response spectra into the design process and suggested that the design force levels then being used for design in the zones of highest seismicity corresponded to design response spectra that had an effective peak ground acceleration of 0.4g. This publication further suggested that this level of ground motion roughly corresponded with that which would be exceeded roughly one time every 500 years, having approximately a 10% probability of exceedance in 50 years. In place of seismic zones, hazard maps published with the ATC document represented seismic hazard in terms of two ground motion parameters, A_a and A_v, plotted by county on the maps. The A_a parameter represented an effective peak ground acceleration—that is, the acceleration that a perfectly rigid structure, having a period of 0 seconds, would effectively experience if subjected to the ground motion. The A_v parameter represented the response acceleration corresponding to the effective peak response velocity that a structure would experience when subjected to this ground motion. While neither the ATC document itself nor the maps published with the document were immediately adopted into the building codes, it became accepted doctrine that the design forces specified in the building codes, still based on the old seismic zonation maps, represented hazards with a 10%/50 year exceedance probability, and that the design procedures contained in the building codes provided a performance level for this ground shaking that would ensure protection of the life safety of building occupants as well as control damage in most structures to levels that would be repairable under these levels of ground shaking. Further, it was considered by many of the participants in the ATC project that structures designed for values of A_a and A_v equal to 0.4g, together with the detailing requirements recommended in the document for that level of design, would be able to survive any earthquake of the type likely to be experienced in California. Together, these combined performance levels were considered to provide a socially acceptable level of risk.

During the 1980s and 1990s, seismologists' ability to estimate ground shaking hazard levels improved significantly. This was largely due to the occurrence of a number of moderate- to large-magnitude earthquakes in regions of California in which there were many strong motion instruments. This provided a wealth of data on the variation of ground motion correlated with distance from the causative fault, magnitude, site characteristics, and other parameters. At the same time, the use of paleoseismic techniques permitted re-evaluation of the recurrence rates of rare, large-magnitude earthquakes in areas such as the New Madrid region in the Mississippi embayment, the region around Charleston, South Carolina, and the Pacific Northwest. Based on this re-evaluation, several inconsistencies in the previous definition of acceptable risk, as described above, became apparent. First, it appeared clear that the 0.4g effective peak ground acceleration, previously assumed to be representative of ground motion with a 10%/50 year exceedance level in zones of high seismicity, significantly underestimated the motion that would be experienced in the near field of major active faults. Also, it became apparent that in areas that experienced truly infrequent, but very large-magnitude earthquakes, such as the Mississippi embayment, structures designed to the 10%/50 year hazard level might not have adequate seismic resistance to resist even historic earthquakes without collapse.

In response, the 1988 *NEHRP Recommended Provisions for New Buildings* published a second series of seismic risk maps, providing A_a and A_v contours for 2% probability of exceedance in 50 years (termed a 2%/50 year exceedance level in the *Guidelines*) in addition to the standard 10%/50 year maps published with previous editions. However, there was no consensus that it was appropriate to actually design buildings for these levels of ground motion. The design community was divided on this issue, some believing that the 10%/50 year maps did not provide adequate protection of the public safety, and others believing that design for the 2%/50 year hazards would be economically impractical.

In the early 1990s, the United States Geological Survey (USGS) developed a new series of ground motion hazard maps, utilizing the latest seismological knowledge. The BSSC attempted to incorporate these maps for use in the 1994 *NEHRP Recommended Provisions*; however, the necessary consensus was not achieved. Some engineers in the western United States believed that the hazards represented by the proposed 10%/50 year maps provided values that were unacceptably high for design purposes in the regions surrounding major active faults, and unacceptably low for design purposes in regions remotely located from such faults. Further, it was felt by some that these maps still did not adequately address the possibility of infrequent, large-magnitude earthquakes in the eastern United States.

The *NEHRP Recommended Provisions* (BSSC, 1997) update process included the formation of a special Seismic Design Procedures Group (SDPG), consisting of earth scientists from the USGS and engineers engaged in the update process. The SPDG was charged with the responsibility of working with the USGS to produce ground motion maps incorporating the latest earth science procedures, and with appropriate design procedures to allow use of these maps in the *Recommended Provisions*. The SDPG determined that rather than designing for a nationwide uniform hazard—such as a 10%/50 year or 2%/50 year hazard—it made more sense to design for a uniform margin of failure against a somewhat arbitrarily selected maximum earthquake level.

This maximum earthquake level was termed a Maximum Considered Earthquake (MCE) in recognition of the fact that this was not the most severe earthquake hazard level that could ever affect a site, but it was the most severe level that it was practical to consider for design purposes. The SDPG decided to adopt a 2%/50 year exceedance level definition for the MCE in most regions of the nation, as it was felt that this would capture recurrence of all of the large-magnitude earthquakes that had occurred in historic times.

There was concern, however, that the levels of ground shaking derived for this exceedance level were not appropriate in zones near major active faults. There were several reasons for this. First, the predicted ground motions in these regions were much larger than those that had commonly been recorded by near field instrumentation in recent magnitude 6 or 7 California events. Second, it was noted, based on the observed performance of buildings in these earthquakes, that structures designed to the code had substantial margin against collapse for ground shaking that is much larger than that for which the building had nominally been designed; in the judgment of the SDPG members, this margin represented a factor of at least 1.5. Consequently, it was decided to adopt a definition of the MCE in zones near major active faults that consisted of the smaller of the probabilistically estimated 2%/50 year motion or 150% of the mean ground motion calculated for a deterministic characteristic earthquake on these major active faults, and to design all buildings, regardless of location, to provide for protection of occupant life safety at earthquake ground shaking levels that are 1/1.5 times (2/3) of the MCE ground motion.

Except in zones near faults with very low recurrence rates, deterministic estimates of ground motion typically result in smaller accelerations than do the probabilistic 2%/50 year estimates of ground motion. The SDPG considered it inappropriate to permit design of structures for lower levels of ground motion than that required by the 1994 *NEHRP Recommended Provisions*, in zones of high seismicity. Consequently, the definition of the MCE incorporated a transition zone between those regions where the MCE has a probabilistic definition and those where there is a deterministic definition; that is, in which the ground motion is taken at 150% of the levels required by the 1994 *NEHRP Provisions*.

The implied performance of buildings designed to the 1997 *NEHRP Recommended Provisions*, assuming the SDPG recommendations are ratified, is related to, but somewhat different from, that which historically has been defined as being an acceptable risk. Specifically, it is implied that buildings conforming to the 1997 *Recommended Provisions* would be able to withstand MCE ground shaking without collapse, and withstand design level ground shaking (2/3 of MCE) at reduced levels of damage associated with both protection of occupant safety and provision of reasonable assurance that the building could be repaired and restored to service.

The calculations of probabilistic ground motions conducted by the USGS as a basis for the response acceleration maps incorporated a number of parameters with significant uncertainties. Potential variation and uncertainty in the values of the most significant parameters, such as the probability of events of varying magnitudes and rupture mechanisms occurring along a given source and the variability of attenuation of ground motion over distance, were considered directly in the probabilistic calculations. Uncertainties in many other parameters were not directly accounted for. Initial studies conducted by the USGS of the potential effects of these other uncertainties indicate that the mapped values represent estimates for which there is a high degree of confidence (about the mean plus one standard deviation level) of non-exceedance at a given probabilistic level.

The *Guidelines* have adopted the same definition of the MCE proposed for adoption in the 1997 *NEHRP Recommended Provisions*, as described above, and have designated it Basic Safety Earthquake 2 (BSE-2). However, the *Guidelines* have not directly adopted the concept of a design earthquake, at 2/3 of the MCE level, as proposed for the *Recommended Provisions*. This was not adopted because this design earthquake would have a different probability of exceedance throughout the nation, depending on the seismicity of the particular region. It was felt such an event would be inconsistent with the intent of the *Guidelines* to permit design for specific levels of performance for hazards that have specific probabilities of exceedance selected by the design team. Consequently, instead of adopting the design earthquake concept, it was decided to adopt the Basic Safety Earthquake 1 (BSE-1).

The BSE-1 is typically taken as that ground motion with a 10%/50 year exceedance probability, except that it need never be taken as larger than 2/3 of the BSE-2 ground motion. The 10%/50 year exceedance probability is consistent with that level of hazard that has traditionally been assumed to be an acceptable basis for design in the building codes for new construction. The limitation of 2/3 of the MCE ground motion was adopted so that design requirements for the BSO, defined in Section 2.4.1, would not be more severe than the design requirements for new construction under the 1997 *NEHRP Provisions*.

Ground shaking hazards may be determined by either of two procedures. Section 2.6.1 of the *Guidelines* provides a general procedure in which spectral response acceleration parameters are obtained by reference to the maps in the package distributed with the *Guidelines*. These parameters are then adjusted, if required, to the desired exceedance probability, and modified for site class effects. The resulting parameters are sufficient to allow development of a complete acceleration response spectrum that is directly referenced by the analysis procedures of Chapters 3 and 9. Section 2.6.2 provides general guidance for the application of site-specific procedures in which regional seismicity and geology and individual site characteristics are considered in the development of response spectra.

On a regional basis, the maps referenced in the general procedure may provide reasonable estimates of the response accelerations for the indicated hazard levels. However, these estimates may be insufficiently conservative for some sites, including those with particularly soft soil profiles or soils subject to seismic-induced instability, and sites located in the near field of a fault. Since many of the structural provisions of the *Guidelines* incorporate lower margins of safety than do the FEMA 222A (BSSC, 1995) provisions, it is important that ground motion characterizations used as the basis of design not be underestimated. Use of the Site-Specific Procedures for these sites will generally result in improved estimates of the likely ground shaking levels, and increase design reliability. Use of the Site-Specific Procedures is also recommended for buildings with Enhanced Rehabilitation Objectives, because such objectives are typically adopted for important buildings in which the greater design reliability provided by a site-specific hazard estimate is appropriate. Site-specific procedures should also be used when a Time-History Analysis is to be performed as part of the rehabilitation procedure, since the development of site-specific ground motions is commensurate with the greater effort required for the structural analysis, and the greater expectations for reliability common to buildings analyzed by that technique.

C2.6.1 General Ground Shaking Hazard Procedure

In the general procedures, reference is made to a series of hazard maps to obtain key spectral response acceleration parameters. These acceleration parameters, when adjusted for probability of exceedance and for site class effects, are sufficient to define an acceleration response spectrum suitable for use for analysis and design. Two sets of two maps are in the map package distributed with the *Guidelines*. One set of maps provides contours of the key response acceleration parameters for the MCE hazard level, as defined in Section 2.4. These maps were developed by the USGS for inclusion in the 1997 *NEHRP Recommended Provisions* in a joint project with the BSSC, known as Project ’97, and incorporate the latest scientific thought on ground motion estimation as of early 1996. The second set of maps was also developed by the USGS as part of the same project, using a 10%/50 year exceedance probability. Other ground shaking demand maps can be used, provided that 5%-damped response spectra are developed that represent the ground shaking for the desired earthquake return period, and that the site soil classification is considered.

For each hazard level, the maps provide contours of the parameters S_S and S_1. The S_S parameter is the 5%-damped, elastic spectral response acceleration for rock sites (class B) at a period of 0.2 seconds. The S_1 parameter represents the 5%-damped, elastic spectral response acceleration for rock (class B) sites at a period of 1.0 second. In the period range of importance to the response of most structures, acceleration response spectra can be represented by a bilinear curve, consisting of a constant response acceleration at short periods and a constant response velocity at longer periods. Since spectral response acceleration is related to pseudo-spectral response velocity by the equation:

$$S_a = \omega S_v = \frac{2\pi}{T} S_v \tag{C2-1}$$

where S_a is the spectral acceleration, ω is the radial frequency of periodic motion, T is the period of motion, and S_v is the pseudo-spectral velocity, then, in the constant velocity range of response, spectral acceleration at any period can be related to that at a one-second period by the factor $1/T$. Thus, the two spectral response acceleration parameters, S_S and S_1, when adjusted for exceedance probability and site class, completely define a response spectrum curve useful for design purposes.

C2.6.1.1 Mapped MCE Response Acceleration Parameters

The MCE maps in the package distributed with the *Guidelines* are the same as those developed by the

SDPG for use in the 1997 *NEHRP Recommended Provisions*. As proposed for use there, the spectral values obtained from these maps would be reduced by a factor of 2/3 to arrive at design spectral values. The *Recommended Provisions* would then provide criteria for design to a performance level within the Damage Control Performance Range, having somewhat more margin against failure (estimated at 150%) than the Life Safety Performance Level, defined in the *Guidelines* with a margin of 133% against failure. In the *Guidelines*, the BSE-2 response accelerations are used to evaluate the ability of structures to meet the Collapse Prevention Performance Level, when designing to achieve the BSO.

In developing acceptance criteria for component actions, the following criteria are set. The permitted inelastic deformation demand for a primary element is set at 75% of the deformation level at which significant strength loss occurs. Although most structures have sufficient redundancy so that collapse would not occur at the loss of the first primary element, this would imply a minimum margin against failure for the Life Safety Performance Level of 1/0.75 or 1.33, apart from inaccuracies inherent in the analysis method. In a similar, but far less rigorous manner, the SDPG, the committee responsible for development of the new NEHRP maps and the corresponding design procedure, judged that the minimum margin against failure contained in the *NEHRP Provisions* is 150%. This was not based on any evaluation of actual acceptance criteria contained in the *NEHRP Provisions*, but rather the judgment that appropriately constructed buildings designed to NEHRP Seismic Performance Category D (or Zone 4 of the 1994 UBC) criteria should not encounter serious problems until ground motion levels of at least 0.6g. The ratio of 0.6g (the judgmentally selected minimum limiting ground motion) to the contemporary design value of 0.4g (for A_a and A_v or Z in the 1994 UBC) resulted in the projected margin of 150%. This 150% is directly related to the 2/3 reduction between the MCE and Design Based Earthquake (DBE) maps in the *NEHRP Provisions* (2/3 = 1/1.5).

It is important to note that the BSE-2 hazards defined by these maps cannot be associated with a particular exceedance probability. Although the hazards indicated for most regions covered by the map have been probabilistically calculated as having a 2%/50 year exceedance probability, the regions surrounding major active fault systems, such as those in coastal California, have been adjusted to include deterministic estimates of ground shaking for specific maximum earthquake events on each of the several faults known to be present in the region. Consequently, the values of the spectral response accelerations obtained from these maps should not be used when attempting to develop hazards with a particular exceedance probability, in accordance with Section 2.6.1.3.

C2.6.1.2 Mapped 10%/50 Year and BSE-1 Response Acceleration Parameters

The probabilistic maps in the package distributed with the *Guidelines* provide contours for the spectral response acceleration parameters at a uniform 10%/50 year exceedance probability. These acceleration parameters, once adjusted for site class effects and to limit maximum accelerations to 2/3 of those of BSE-2, can be used directly to evaluate the ability of structures to meet the Life Safety Performance Level when designing to achieve the BSO. In addition, these acceleration parameters, having a uniform exceedance probability, can be used to derive response acceleration parameters for any exceedance probability, using the procedure of Section 2.6.1.3.

C2.6.1.3 Adjustment of Mapped Response Acceleration Parameters for Probability of Exceedance

An examination was performed of typical hazard curves used by the USGS to construct the ground motion maps distributed with the *Guidelines*. A log-log plot of these curves in a domain of annual frequency of exceedance (or return period) versus response spectral acceleration is nearly linear between probability of exceedance levels of 2% and 10% in 50 years. Therefore, for regions in which the BSE-2 maps directly provide spectral response acceleration parameters with a 2%/50 year exceedance rate, a linear interpolation on a log-log plot of spectral response acceleration versus return period can be made to find the response spectral accelerations for any desired probability levels within these ranges. This approach is applicable anywhere that the short period response acceleration parameter, S_S, is less than 1.5g. Equation 2-1 provides a closed form solution for this logarithmic interpolation. Equation 2-2 allows return period, P_R, to be determined for any defined probability of exceedance in 50 years.

In regions where the short period spectral response accelerations provided on the BSE-2 map are equal to or greater than 1.5g, the response acceleration contours on the maps are based on deterministic rather than probabilistic concepts. In these regions the BSE-2 map values cannot be used to interpolate for intermediate exceedance rates. Instead, Equation 2-3 is used to estimate the spectral response acceleration parameters at arbitrary return periods by extrapolating from the 10%/50 year value, obtained from the maps with an approximate hazard curve slope, represented by the coefficient *n*. These approximate hazard curve slopes have been estimated on a regional basis. They were derived by examining the typical hazard curves developed by the USGS for representative sites in each of the major seismicity zones including California, the Pacific Northwest, the Intermountain region, Central, and Eastern United States and taking an approximate mean value for these sites. A similar approach is used to estimate spectral response accelerations parameters for hazards with exceedance rates greater than 10%/50 years in all regions of the nation, as the logarithmic extrapolation that may be used between exceedance rates of 2%/50 years and 10%/50 years is not valid outside this range.

C2.6.1.4 Adjustment for Site Class

The definitions of the site classes, A through F, and site coefficients, F_a and F_v, were originated at a workshop on site response held at the University of Southern California in November 1992. In that workshop, convened by the National Center for Earthquake Engineering Research (NCEER), Structural Engineers Association of California (SEAOC), and BSSC (Martin and Dobry, 1994; Rinne, 1994), consensus values for the ratios of response spectra on defined soil profile types relative to rock for the short-period range and long-period range were developed on the basis of examination of empirical data on site amplification effects (especially data from the 1989 Loma Prieta earthquake) and analytical studies (site response analyses). The response spectral ratios relative to rock (site class B) were designated F_a for the short-period range (nominally at a period of 0.3 second) and F_v for the long-period range (nominally at a period of 1.0 second). The recommendations of this workshop for both the soil profile types and the site factors F_a and F_v were adopted by the BSSC for the 1994 edition of *NEHRP Recommended Provisions* (BSSC, 1995).

The 1994 *NEHRP Recommended Provisions* defined values of F_a and F_v in Tables 2-13 and 2-14 for ground motions with effective peak ground accelerations on rock sites equal to or less than 0.40g—the highest value used in the *Provisions*. For effective peak ground accelerations on rock equal to 0.50g, the values of F_a and F_v were similarly obtained by using the values recommended by the workshop. The workshop did not present recommendations for values of F_a and F_v for effective peak ground accelerations on rock greater than 0.50g. In fact, because of a lack of recorded data on site amplification effects at higher acceleration levels, there is increasing uncertainty as to appropriate values of F_a and F_v for higher accelerations. It is not clear that the site factors would continue the trend of reduction with increasing acceleration. Therefore, values of F_a and F_v for effective peak ground accelerations on rock exceeding 0.50g have been obtained using the values of F_a and F_v defined by the workshop for an acceleration coefficient of 0.50. Consistent with the workshop recommendations, site-specific studies incorporating dynamic site response analyses are recommended for soft soils (profile E) for effective peak ground accelerations on rock equal to or greater than 0.50g. Therefore, values of F_a and F_v are not presented in Tables 2-13 and 2-14 for Type E soils for effective peak ground accelerations on rock equal to or greater than 0.50g.

It should be noted that, in contrast to the site factors in previous editions of the *NEHRP Recommended Provisions for New Buildings* and in the *Uniform Building Code* (ICBO, 1994), the new site factors incorporate two significant features. First, there are factors for short periods as well as long periods, whereas the previous site factors were only for long periods. This reflects the empirical observation (especially from the 1989 Loma Prieta earthquake) that short-period as well as long-period ground motions are amplified on soil relative to rock, especially for lower acceleration levels. Second, the factors are a function of acceleration level, whereas the previous factors were independent of the acceleration. This reflects the nonlinearity of soil response; soil amplifications decrease with increasing acceleration due to increased damping in the soil. In common with the previous site factors, the new site factors increase as the soils become softer, but the new factors are higher than the previous factors at the lower acceleration levels.

C2.6.1.5 General Response Spectrum

Section 2.6.1.5 provides guidelines for the development of a general acceleration response spectrum based on the values of the design response acceleration parameters, S_{XS} and S_{X1}, that include necessary adjustments for probability of exceedance and site class effects. The shape of this general response spectrum incorporates two basic regimes of behavior—a constant response acceleration range at short periods and a constant response velocity range at long periods, in which, as previously described, response acceleration varies inversely with structural period. The transition between the two regimes occurs simply at that period where acceleration values calculated assuming constant response velocity would exceed those of the constant acceleration regime.

This general spectrum is a somewhat simplified version of the spectrum presented by Newmark and Hall (1982). The Newmark and Hall spectrum, derived from a statistical evaluation of a number of historic earthquake ground motion recordings, actually included four distinct domains. In addition to the constant response acceleration and constant response velocity domains included in the spectra contained in the *Guidelines*, the Newmark and Hall spectrum included a constant response displacement domain at very long periods, in which response acceleration varies with the inverse of the square of structural period ($1/T^2$), and a transition zone in the very short period range, in which the response acceleration increased rapidly from the effective peak ground acceleration for infinitely rigid structures (natural period of 0 seconds) to the constant response acceleration value.

The simplified version of the general spectrum presented in the *Guidelines* is sufficiently accurate for use for most structures on most sites, and adequately represents the response of structures to the random vibratory ground motions that dominate structural response on sites located 10 or more kilometers from the fault rupture surface. However, it does potentially overstate the response acceleration demand for very rigid (short-period) structures and for very flexible (long-period) structures. In addition, it potentially understates the effects of the impulsive-type motions that have been experienced on sites located within a few kilometers of the fault rupture surface. These impulsive motions can cause very large response in structures with periods ranging from perhaps one second to as long as four seconds. For buildings within this period range, and located on sites where such impulsive motions are likely to be experienced, the site-specific procedures should be considered.

The approach adopted by the *Guidelines* for construction of a general response spectrum is similar to the approach that has been adopted by the *NEHRP Recommended Provisions* for designs based on the equivalent lateral force technique. In the development of the *Guidelines*, it was decided, for several reasons, to neglect the very short period range of the spectrum, in which response accelerations are somewhat lower than those in the constant acceleration domain. First, it was the feeling of the development team that very few building structures actually have effective periods within this very short period range, especially when the likely effects of soil structure interaction and degradation due to inelastic behavior are considered. Second, designing for acceleration response within this very short period range could lead to unconservative designs. This is because as a structure responds inelastically to earthquake ground motion, its stiffness will tend to degrade somewhat, resulting in a longer effective period. Therefore, if a structure has a very short period and is designed for the resulting reduced accelerations, under the effects of stiffness degradation it could shift to a somewhat longer period and experience more acceleration response than that for which it had been designed.

The decision to neglect the constant displacement domain of the spectrum was made for several reasons. First, at the time of the *Guidelines* development, there were no readily available rules for determining the period at which the constant displacement domain initiates. This transition period would appear to be a function of the site class, as well as the location and position of the individual site with respect to the fault rupture plane and direction of rupture propagation. Such effects are very difficult to incorporate in a series of general purpose rules. The *NEHRP Recommended Provisions* have adopted a period of four seconds as a general guideline for this transition period, when performing dynamic analyses. However, this period is somewhat arbitrary and may produce unconservative designs on some sites. Second, relatively few structures that will be rehabilitated using the *Guidelines* are likely to have periods long enough to fall within this domain. Those structures that do have such long periods are likely to be quite tall and, therefore, of the class for which site-specific ground motion determination is recommended. Nothing in these *Guidelines* would prevent the adoption of spectra with a constant displacement domain if it is developed on the basis of site-specific study by a knowledgeable earth scientist or geotechnical engineer.

It should be noted that spectra generated using site-specific procedures may not have well-defined constant acceleration, constant velocity, and constant displacement domains, although they will typically resemble spectra that have these characteristics. For such spectra, it is recommended that, at least for consideration of first mode response, the effective value of the response acceleration for very short periods be taken as not less than that obtained at a period of 0.3 seconds, or that which would be derived by the general procedure. Consideration could be given to using the value of accelerations for very short period response when evaluating the effect of higher modes of response.

The general response spectrum has been developed for the case of 5%-damped response. A procedure is also provided in the *Guidelines* for modifying this 5%-damped spectrum for other effective damping ratios. These modification factors are based on the recommendations contained in Newmark and Hall (1982) for median estimates of response, except that for damping ratios *ß* of 30% and greater, more conservative estimates have intentionally been used, consistent with the approach adopted for seismic-isolated structures in the 1994 *NEHRP Provisions*. Again, it is important to note that structures may not respond with the same effective damping when they are subjected to impulsive-type motions, as they do when subjected to the more typical random vibratory motions represented by the general response spectrum.

C2.6.2 Site-Specific Ground Shaking Hazard

In developing site-specific ground motions, both response-spectra, and acceleration time histories, it should be kept in mind that the characteristics of the ground motion may be significantly influenced by not only the soil conditions but also the tectonic environment of the site. Of particular importance for long-period structures is the tendency for near-source ground motions to exhibit a long-period pulse (e.g., Sommerville and Graves, 1993; Sadigh et al., 1993; Boatwright, 1994; Heaton and Hartzell, 1994; Heaton et al., 1995). The existence of very hard rock in the eastern U.S. (relative to typical rock in the western U.S.) results in an increase in the high-frequency content of ground motion in the east as compared to that in the west (e.g., Boore and Joyner, 1994). Duration of strong ground shaking is closely related to earthquake magnitude and also dependent on distance and site conditions (e.g., Dobry et al., 1978).

A greater number of acceleration time histories is required for nonlinear procedures than for linear procedures because nonlinear structural response is much more sensitive than linear response to characteristics of the ground motions, in addition to the characteristics of response spectral content. Thus, nonlinear response may be importantly influenced by duration as well as by the phasing and pulse sequencing characteristics of the ground motions.

C2.6.3 Seismicity Zones

No commentary is provided for this section.

C2.6.4 Other Seismic Hazards

No commentary is provided for this section.

C2.7 As-Built Information

Prior to evaluating an existing building and developing a rehabilitation scheme, as much existing data as are available should be gathered. This includes performing a site visit, contacting the applicable building department that may have original and modified plans and other documents, and conducting meetings with the building owners, managers, and maintenance engineers who may have direct knowledge of the condition and construction of the building and its past history, as well as files and documents with similar valuable information. Also, if the original design professionals (e.g., architects and engineers) and construction contractors and subcontractors can be identified, additional information—such as design bases, calculations, change orders, shop drawings, and test reports—may be attainable. After available documents are reviewed, field surveys should be made to verify the accuracy and applicability of the available documents. When documents are not available, field measurements are required. A program for destructive and nondestructive tests should be developed and implemented.

The importance of attempting to obtain all available documentation of a building’s construction prior to proceeding with an evaluation and rehabilitation program cannot be overemphasized. Without a clear understanding of the construction of a building, it is difficult to predict its response to future seismic demands and, therefore, to determine an appropriate program for rehabilitation. If documentation of the building’s construction is not available, it is often necessary to conduct extensive surveys of the building to allow development of this documentation. In most buildings, critical details of the structural system are obscured from view by architectural finish, fireproofing, and the structural elements themselves. Therefore, destructive examination may often be required to obtain an appropriate level of information.

For those buildings for which good documentation, in the form of original design drawings and specifications, is available, it should not be assumed that these documents represent the actual as-built or current configuration of the structure. As a minimum, a general survey of the structure should be conducted to confirm that the construction generally conforms to the intent of the documents and that major modifications have not been made. It may also be advisable to confirm that certain critical details of construction were actually constructed as indicated.

Though some useful information, such as probable material strengths, can be obtained by reference to the building codes and standard specifications commonly in use at the time of construction, such data should be used with caution. Since many municipalities are slow in their adoption of current standards, buildings constructed in one era may actually have been designed in accordance with earlier standards. Also, there is no guarantee that a building has actually been designed and constructed in conformance with the applicable code requirements.

C2.7.1 Building Configuration

Most buildings have a substantial lateral-load-resisting system, although this may not be adequate to achieve the Rehabilitation Objectives. Often, a significant portion of a building's resistance to lateral demands will be provided by elements that were not specifically intended by the original designer to serve this purpose. In particular, the walls of many buildings, although not intended to participate in lateral force resistance, will in actuality do so, and may not only provide substantial resistance but also alter the manner in which the primary system behaves. These elements can also introduce critical irregularities into a building's lateral-load-resisting system. Architectural walls and partitions can affect the stiffness of structural elements and also introduce soft story and torsional conditions into otherwise regular buildings. It is important to consider these aspects when developing a concept of the building's configuration.

C2.7.2 Component Properties

In order to define the strength and deformation characteristics of the building and its elements, one must know the relevant properties of the components, including the cross sections present, material strengths, and connectivity details. Since the strength of materials actually present in a structure can vary significantly from that indicated on original construction drawings, testing is the preferred method of ascertaining material strength. In some cases, original construction quality control data—including mill test certificates, concrete cylinder test reports, and similar documentation—may provide a direct indication of the material strengths. Such data should be adequate if the structure has remained in good condition.

It is important to obtain the force-displacement characteristics of the existing elements—whether or not they are to be included in the lateral-force-resisting system—because of the need to determine the deformation compatibility relationships of existing materials with the new materials used in the rehabilitation concepts. When a building responds to ground motion, the demands on nearly all components of the building are altered. There is potential for components that do not provide significant lateral resistance in a structure to experience demands that can result in severe damage. Reinforced concrete buildings with flat slab floors and perimeter shear walls provide a good example. The equivalent frames comprising the flat slabs and columns may provide relatively little lateral-force resistance compared to that of the perimeter shear walls. However, such slabs can be extremely vulnerable to lateral deformations that induce relatively large shear stresses in the column-to-floor-slab connections. Although most engineers would not consider the slabs to be part of the lateral-force-resisting system for such buildings, it is important to quantify the lateral deformation capacity of these components to ensure that earthquake demands are maintained below a level that would result in collapse potential. Therefore, investigation of the properties of such secondary elements may be required.

When determining the deformation capacity of a component, or its ability to deliver load to adjacent components, its strength should be calculated using the expected values of strengths for the materials in the building. The expected strengths are the best estimates of the actual strength of the materials in the building as represented by the average value of strengths that one would obtain from tests on a series of samples. The expected strength is different from the nominal or specified strength that is commonly used when materials are specified for new construction. Typically, the actual strengths of materials in new construction are considerably higher than the specified strengths, which provides an additional margin of safety in new construction. Expected strengths are used in the *Guidelines* for two reasons. First, the use of artificially low values, based on nominal or specified values, would result in poor predictions of building performance. Second, the use of such low values, particularly in nonlinear procedures, could result in underestimation of the actual strength demands on some elements of the system.

In addition to expected strengths, the *Guidelines* require estimates of lower-bound strengths for the evaluation of the adequacy of component force actions during force-controlled behavior.

For many existing buildings, information on the strengths obtained in the original construction is not readily available; hence, it is necessary to determine expected strengths from field or laboratory tests. The individual material sections of the *Guidelines* recommend appropriate types, methods, and numbers of tests to define adequately the material strength of an existing building (see Chapters 5 through 8). Actual strengths of materials within a building may vary from component to component; for example, beams and columns in concrete structures may be constructed of materials having different strengths. Strengths may also be affected by deterioration, corrosion, or both.

The κ factor is used to express the confidence with which the properties of the building components are known, when calculating component capacities. The value of the factor is established from the knowledge that the engineer is able to obtain, based on either access to the original construction documents or surveys and destructive or nondestructive testing of representative components.

Two values for the κ factor have been established, indicating whether the engineer's knowledge of the structure is "minimal" or "comprehensive." Recommendations are given in the material chapters as to the level of investigation required for each class. The numerical values of the κ factor are selected to reward a more detailed investigation of the existing building by requiring the use of a discounted value of the expected capacity to be used for analysis and design purposes when only limited information on the structure is available. When nonlinear procedures are used for a building, a comprehensive level of knowledge should be obtained with regard to component properties; if this were not done, the apparent accuracy of the procedure could be misleading.

Examples of the type of knowledge needed for a reinforced concrete shear wall component, in order to qualify under the two classes of knowledge (κ factors), are as follows:

- "Comprehensive" Class

  a. Original construction documents are available and the construction was subject to adequate inspection. Limited visual access to the building and material testing confirm the provisions of the original documents.

  b. Original construction documents are not available, but full access to critical load path components is available, and an adequate testing and inspection program provides information sufficient to define component properties and to conduct structural analyses. Critical details such as the location and length of reinforcing splices are confirmed.

- "Minimal" Class

  a. Only limited or no construction documentation is available.

  b. Access is provided to some but not all load path elements.

  c. Nondestructive Examination (NDE) provides location of reinforcing bars in the wall and limited exposure provides information on bar size and splice lengths. Limited testing for concrete and steel strengths has been performed, and the strength levels and variation in strength levels are consistent with building construction for the age of the building.

C2.7.3 Site Characterization and Geotechnical Information

Regional geologic maps produced by the USGS, as well as those produced by a number of state and local agencies, can be a good source of basic geotechnical data for a site. Information from the geologic maps could include data relative to the surficial geologic unit mapped in the vicinity of the building site. These maps typically include a brief assessment of engineering parameters and performance characteristics that may be attributed to specific geologic units. Information obtained from topographic maps would be used to evaluate potential effects from landslides occurring either on-site or off-site. Finally, various cities have developed hazard maps that may indicate zones that may be susceptible to landslides, liquefaction, or significant amplification of ground shaking. Information obtained from these sources could be used in assessing the large-scale performance of the site, and the need to obtain site-specific data.

Relevant site information that could be obtained from geotechnical reports would include logs of borings and/or cone penetrometer tests, laboratory tests to determine the strength of the subsurface materials, and engineering assessments that may have been conducted addressing geologic hazards at the site, such as faulting, liquefaction, and landsliding. Information should be obtained from geotechnical reports or other regional studies regarding potential depths of groundwater at the site.

Existing building drawings should be reviewed for relevant foundation data. Information to be derived from these drawings could include:

- Shallow foundations
  - footing elevation
  - permissible bearing capacity
  - size

- Deep foundations
  - type (piles or piers)
  - material
  - tip elevation
  - cap elevation
  - design load

Visual site reconnaissance should be conducted to gather information for several purposes, including confirmation that the actual site conditions agree with information obtained from the building drawings, documentation of off-site development that may have a potential impact on the building, and documentation of the performance of the existing building and adjacent areas to denote signs of poor foundation performance.

C2.7.4 Adjacent Buildings

Although buildings are classically evaluated and designed with the assumption that they are isolated from the influence of adjacent structures, there are many instances in which this is not the case. In older urban centers, many buildings were constructed immediately adjacent to each other, with little if any clearance between the structures. Many such buildings have party walls and share elements of their vertical- and lateral-force-resisting systems. Building adjacency issues may also be important for large complexes of buildings constructed in different phases, over a number of years, and for large buildings provided with expansion joints between portions of the building. It is critical to the rehabilitation process to recognize the potential effects of adjacent structures on building behavior.

In order to evaluate potential building interaction effects, it is necessary to understand the construction and behavior of both buildings. In its simplest form, evaluation requires knowledge as to whether or not adjacent structures actually share elements, such as party walls, and an estimate of how much lateral motion each building is likely to experience so that the likelihood of pounding can be evaluated. This requires that at least a minimum level of information be obtained for the adjacent structure, or structures, as well as the building being rehabilitated. Obtaining as-built information for adjacent structures that have different ownership than the building may be difficult. Most owners will be willing to share available information, although they will be less motivated to do so than the owner for whom rehabilitation work is planned. It will seldom be possible or necessary to obtain material test data for adjacent structures. In many cases, it will be necessary to make informed assumptions as to the adjacent structure’s characteristics.

C2.7.4.1 Building Pounding

Building pounding is a phenomenon that occurs when adjacent structures are separated at distances less than the differential lateral displacements that occur in each structure as a result of their earthquake response. As a result, the buildings impact each other, or “pound.” Pounding can cause local crushing of the structures, and failure of structural and nonstructural elements located in the zone of impact. In addition, pounding can cause a transfer of kinetic energy and momentum from one structure to another, resulting in significantly different earthquake demands in each structure than would be experienced if pounding did not occur. Key to evaluating the potential effects of impact is identifying whether or not such impacts will occur. Conservatively, if the adjacent structures respond to the earthquake ground motion completely out of phase, impact can occur only if the separation of the adjacent structures is less than the sum of the maximum displacement response of the structures at the level of potential impact. Following this approach, the *Guidelines* suggest that adjacency evaluation should be conducted wherever the adjacent structure is closer to the building than 4% of its height above grade at the location of potential impact. This correlates with the assumption that most structures will not exceed a drift in excess of 2% when responding to earthquake ground motions.

C2.7.4.2 Shared Element Condition

In many older urban areas, two buildings under different ownership often share in common the wall separating the two structures. These “party” walls often form part of the lateral and gravity load systems for both structures. If the buildings attempt to move independently during response to earthquakes, the shared wall can be pulled away from one or the other of the structures, resulting in partial collapse. Similar conditions often occur in buildings constructed with expansion joints. In such buildings, a single line of columns may provide gravity support for portions of both structures. Again, differential lateral movement of the two structures can result in collapse.

C2.7.4.3 Hazards from Adjacent Structures

There are a number of instances on record in which buildings have experienced life-threatening damage, and in some cases collapse, not as a result of their own inadequacies, but because debris or other hazards from an adjacent structure affected them. In many cases, there may be little that can be done to mitigate this problem. However, it is important to recognize the problem’s existence and the consequences with regard to probable building earthquake performance. It makes little sense to rehabilitate a building to Enhanced Rehabilitation Objectives if it is likely to have an adjacent structure collapse on it. In such cases, the best seismic risk mitigation measure may be to relocate critical functions to another building.

C2.8 Rehabilitation Methods

Two basic methods for developing a rehabilitation design are defined in the *Guidelines*. These are Simplified Rehabilitation—a method available for some structures in which deficiencies common to certain model building types, and known to have caused poor earthquake performance in the past, are directly mitigated—and Systematic Rehabilitation, a method available for any building, in which a complete analysis of the structure is performed, and all elements and components critical to obtaining the desired Rehabilitation Objective are checked for adequacy to resist strength and deformation demands against specific acceptance criteria.

C2.8.1 Simplified Method

The Simplified Rehabilitation Method uses direct guidelines for mitigating specific types of deficiencies common to certain model buildings. They are based on the fact that for certain relatively simple types of structures, poor performance in earthquakes has repeatedly been observed to be the result of several critical failure modes, uniquely tied to the common construction detailing inherent in these model building types. Examples include light wood frame structures, which commonly experience partial collapse due to the presence of unbraced cripple walls; and reinforced and unreinforced masonry buildings and concrete tiltup buildings, which commonly experience partial collapse due to a lack of adequate out-of-plane attachment between the heavy walls and flexible diaphragms. The Simplified Rehabilitation Method provides specifications for direct remediation of these characteristic deficiencies, without necessarily requiring a complete numerical analysis of the building’s lateral-force-resisting system. However, as a minimum, a complete evaluation in accordance with FEMA 178 (BSSC, 1992a) is recommended prior to specifying the Simplified Rehabilitation Method.

Most building structures, regardless of whether or not they have explicitly been designed for lateral-force resistance, do have both formal and informal lateral-force-resisting systems and, therefore, significant capability to resist limited levels of ground shaking without experiencing severe damage or instability. As an example, the architectural partitions in light wood frame construction together with the ceilings, floors, and roofs will typically form a complete lateral-force-resisting system with capacity to resist a significant portion of the building’s weight, applied as a lateral force, even though few such structures have been designed for this behavior. Therefore, if the Simplified Rehabilitation guidelines for such structures are implemented, a structure with significant but unquantified seismic resistance will be obtained. If a FEMA 178 (BSSC, 1992a) evaluation is performed and all deficiencies identified in the evaluation are mitigated using the Simplified Rehabilitation Method, then the building is judged capable of achieving the Life Safety Performance Level for 10%/50 year ground shaking demands. However, because these procedures do not include a complete check of the adequacy of all important elements in the structures, and because the stability of the structure under larger levels of ground motion—or when subject to other hazards such as liquefaction or differential settlement—is not certain, Simplified Rehabilitation is not considered to achieve the BSO.

C2.8.2 Systematic Method

In Systematic Rehabilitation, a complete analysis of the adequacy of all important elements of the building to resist forces and deformations induced in the structure by its response to the ground motion and other earthquake hazards is conducted. Compared with procedures used in the design of new structures, greater attention is given to the effects of earthquake response on elements of the structure not specifically intended to be part of the lateral-force-resisting system. Any element that is critical to attainment of the desired performance level must be analyzed in Systematic Rehabilitation. This includes elements required to resist gravity loads, as well as nonstructural components that are important to the attainment of the performance.

C2.9 Analysis Procedures

Two basic analysis approaches for confirming the adequacy of a rehabilitation strategy are defined in the *Guidelines*. These are linear (elastic) analysis and nonlinear (inelastic) analysis. Both approaches may be performed using either static or dynamic procedures. The applicability of each of these procedures to a given structure is based on their ability to reasonably predict the likely distribution of seismic demands on the various structural elements and components that the building comprises. These issues are discussed below.

C2.9.1 Linear Procedures

In Linear Dynamic Procedures (LDP) and Linear Static Procedures (LSP), lateral forces are distributed to the various elements and components of the structure in accordance with their relative elastic stiffness characteristics. As in the *NEHRP Recommended Provisions,* FEMA 222A (BSSC, 1995), the lateral forces applied to the structure may be determined based upon a dynamic Time-History Analysis, a response spectrum method analysis, or a simplified equivalent static procedure based on the typical dynamic response of well-behaved, regular structures. While the linear procedures contained in the *Guidelines* are parallel to those contained in BSSC (1995) for new building design, the manner in which the forces and deformations predicted by these procedures are evaluated is significantly different.

The *NEHRP Recommended Provisions* for design of new structures attempt to control earthquake performance by requiring that buildings possess a minimum lateral-force-resisting strength and sufficient elastic stiffness to resist lateral forces within defined drift limits. The lateral forces used for design are based on an elastic analysis of the response of the structure to the design ground motion, but are scaled down substantially—by a response modification factor R—from the level that would be experienced by a structure with adequate strength to resist earthquake-induced forces within the elastic range. These response modification factors have been set based on the judgment and experience of those who wrote the building codes, and are based, to some extent, on the observed performance of buildings in past earthquakes. Use of these scaled-down forces in designing structures implies that when subjected to a design event, the structures will experience significant inelastic demands, and displacements will be substantially larger (by a factor C_d) than calculated under the specified design forces. Limitations on structural configuration, and special requirements for structural detailing and quality of materials, are included in the provisions in parallel with the strength requirements, so that the building may behave acceptably under these conditions.

The approach taken for new construction is not always directly applicable to existing buildings, which often have an unfavorable structural configuration, nonconforming detailing, and materials of substandard quality. Such a structure, even though provided with the minimum strength specified by the building codes for new construction, may not have adequate inelastic deformation capacity to resist the design earthquake within the desired performance limits. Therefore, the linear methods contained in the *Guidelines* have been specifically formulated to allow evaluation of the adequacy of the various building components to resist the inelastic deformation and strength demands which will be imposed on them by a design earthquake.

As with the *NEHRP Recommended Provisions*, an analysis is performed to determine the response (strength and deformation demands) that would be imposed on the structure by the design earthquake, if the building remained completely elastic. However, instead of reducing the earthquake forces by R and then combining them with other loads, the earthquake forces are directly combined with those imposed by dead and live loads and compared against the yield capacity of the components. If all critical actions of the components are found to have acceptable levels of capacity for the implied demands, as judged by the permissible values of a component ductility measure, m, specified in the materials chapters for the various Performance Levels, and the inter-story drifts predicted by the analyses are also within acceptable levels, then the rehabilitation design is deemed adequate. However, if some critical component actions are determined to have ductility demands that exceed acceptable levels, or if inter-story drifts are found to be excessive for the desired Performance Level, then the design is deemed inadequate.

When a linear procedure indicates that a rehabilitation design is inadequate for the desired performance levels, a number of alternatives are available. These include the following:

- If the inadequacy of the design is limited to a few primary elements (or components), it is possible to designate these deficient elements (or components) as secondary. The structure can then be reanalyzed and evaluated to determine if acceptable performance is predicted.

- If the analysis indicates only limited inadequacy, the use of a nonlinear procedure may demonstrate acceptable performance. This is because the nonlinear procedures provide more accurate estimates of demands than do linear procedures. This permits the use of somewhat more liberal acceptance criteria, resulting in some structures indicated as being marginal under linear procedures to be found to be acceptable by nonlinear procedures.

- The design can be revised to include additional rehabilitation measures that provide increased stiffening, strengthening, energy dissipation capacity, or response modification, or an alternative rehabilitation strategy can be selected.

Some structural components do not have significant inelastic deformation capacity. These brittle elements will fail if the load on them exceeds their capacity. An example is a column, which will buckle if loaded with excessive axial force. Such components could conservatively be evaluated in the linear procedures using a maximum permissible m value of 1.0. However, such an approach would often be too conservative. Because most elements in a structure have some ductility, and will respond in an inelastic manner in an earthquake, the unreduced force demands predicted on brittle components by a linear procedure may be substantially larger than those that the structure is actually capable of imposing on the component. To predict accurately the demands on such an element, a nonlinear procedure should be performed. In lieu of such a procedure, the linear procedures permit maximum strength demands on brittle elements to be estimated using an approximate force-delivery-reduction factor, designated J.

Linear procedures, while easy to apply to most structures, are most applicable to buildings that actually have sufficient strength to remain nearly elastic when subjected to the design earthquake demands, and buildings with regular geometries and distributions of stiffness and mass. To the extent that buildings analyzed by this method do not have such strength or regularity, the indications of inelastic ductility demands predicted by the elastic methods may be very inaccurate. In recognition of the relative inaccuracy of the linear techniques, the acceptance criteria contained in the materials chapters have intentionally been set with some level of conservatism, in order to provide a reasonable level of confidence that overall structural performance to the desired level can be attained.

Buildings that have relatively limited inelastic demands under a design earthquake may be evaluated with sufficient accuracy by linear procedures, regardless of their configuration. If the largest component DCR calculated for a structure does not exceed 2.0, the structure may be deemed to fall into this category, for the particular earthquake demand level being evaluated.

For buildings that have irregular distributions of mass or stiffness, irregular geometries, or nonorthogonal lateral-force-resisting systems, the distribution of demands predicted by an LDP analysis will be more accurate than those predicted by the LSP. Either the response spectrum method or Time-History Method may be used for evaluation of such structures.

Section 2.9.1 provides guidance as to when a dynamic procedure should be used.

A linear procedure is deemed applicable unless the results derived from the analysis indicate large ductility demands and the presence of certain irregularities, which would invalidate the predicted distribution of demands. The user must first determine whether an LSP or LDP should be used. An LDP may always be used, in those cases where linear procedures are applicable. The LSP may be used unless either vertical or torsional stiffness or mass irregularities exist. Stiffness or mass irregularities in a structure produce mode shapes that can be significantly different from those typical for a regular structure. Consequently, structures with these irregularities present may have substantially different responses to earthquake ground motion than regular structures. Since the lateral forcing function used in the LSP is derived from the response of regular structures, it should not be used for structures with these irregularities.

The presence of mass or stiffness irregularities, or both, can often be determined only after some analysis. The *Guidelines* suggest that if a user is in doubt with regard to the presence of such irregularities, the LSP may be employed to determine if such irregularities exist. The pattern of displacements in the structure predicted by such an analysis will typically indicate the presence of these irregularities. If a vertical stiffness or mass irregularity is present, this will typically show up as a concentration of drift demand in the structure. In vertically regular structures, inter-story drifts will be distributed in a uniform manner up the structure. In vertically irregular buildings, some stories will exhibit significantly greater drift than others. Similarly, if torsional stiffness or mass irregularities are present, the displacement pattern predicted by the LSP will indicate significant twisting of the structure, in plan.

In addition to being recommended for irregular structures, the LDP is also recommended for structures with heights that exceed 100 feet and buildings with nonorthogonal lateral-force-resisting systems. LDPs are recommended for tall structures because their response is often dominated by higher modes, which are more accurately tracked by the dynamic procedure. Also, tall buildings are generally important structures and warrant the extra care in modeling required to perform a dynamic procedure. Similarly, buildings with nonorthogonal lateral-force-resisting systems typically experience complex patterns of lateral movement (i.e., twisting and translation in directions that are skewed relative to the principal axes), resulting in element stresses and deformations that are more difficult to predict. For such buildings, the more careful development of an analytical model typically required for a dynamic procedure is deemed appropriate.

Once a linear procedure, either static or dynamic, has been performed for a structure, it is possible to determine if the predicted response is sufficiently elastic or uniform to justify the procedure's use. This is done by examining the distribution of calculated DCR values for the critical actions of the controlling components of the primary elements. The critical actions for a component are the independent "weak link" actions that can limit the participation of the component in the structural system.

Table C2-1 lists the typical actions for common structural components. The concept of "critical actions" will be demonstrated by example, in this case the components of a single bay reinforced concrete portal frame. The components are the columns, the beam, and the joint between each column and the beam. As indicated in Table C2-1, the various actions that can limit the beam's capacity to participate in the lateral-force-resisting system include its shear capacity and the flexural capacity of the section at either end for positive and negative bending moments. For each of these actions, a DCR value is calculated, based on the results of the linear procedure. First, the DCR values for the beam flexural capacity are calculated. Next, the beam is evaluated to determine whether it is shear critical or flexurally critical. The flexurally limited shear is calculated using Equation C2-2.

$$V_f = \frac{(M_L + M_R)}{L} + V_D + V_L \qquad \text{(C2-2)}$$

Table C2-1 Typical Actions for Structural Components

Structural Component	Action
Brace	Member axial force Connection axial force
Steel or Timber Beam or Column	Member axial force Member end shear force Member end moment Connection axial force Connection shear force Connection moment
Reinforced Concrete or Masonry Beam, Column, or Pier	Axial force End shear force End positive moment End negative moment Joint shear capacity
Unreinforced Masonry Pier or Spandrel	Axial force End shear force End moment

where:

L = Length of the beam span between points of plastic hinging

M_L = Plastic capacity of the beam at the left end

M_R = Plastic capacity of the beam at the right end

V_D = Beam shear due to dead loads

V_f = Shear resulting from development of the beam's plastic flexural capacity, at each end

V_L = Beam shear due to live loads

If the value of V_f is less than the nominal shear capacity of the beam, then the beam is flexurally critical and the controlling DCR values for bending at either end of the beam are the critical values. If V_f is greater than the beam's shear capacity, then the beam is shear critical and the DCR value computed for beam shear is the critical value for the component. Next, critical DCRs are determined for the other frame components, including the columns and the beam-column joints.

Determination of the controlling components for an element can be done by simple comparison of the calculated DCR values for the critical actions of each of the various components. The controlling component is the one that will reach its capacity at the lowest level of lateral loading to the element. The component with the highest calculated DCR value for its critical action will be the controlling component. If this frame were proportioned such that under increasing lateral loads the columns reached their capacity in flexure (or shear, or axial load) prior to the beam reaching its critical capacity, then the columns would be the controlling components. In this case, the calculated DCR values for the critical column components would exceed those for the beam.

C2.9.2 Nonlinear Procedures

Nonlinear procedures generally provide a more realistic indication of the demands on individual components of structures that are loaded significantly beyond their elastic range of behavior, than do linear procedures. They are particularly useful in that they provide for:

- More realistic estimates of force demands on potentially brittle components (force-controlled actions), such as axial loads on columns and braces
- More realistic estimates of deformation demands for elements that must deform inelastically in order to dissipate energy imparted to the structure by ground motions
- More realistic estimates of the effects of individual component strength and stiffness degradation under large inelastic demands
- More realistic estimates of inter-story drifts that account for strength and stiffness discontinuities that may develop during inelastic response
- Identification of critical regions in which large deformation demands may occur and in which particular care should be taken in detailing for ductile behavior
- Identification of strength discontinuities in plan or elevation that can lead to changes in dynamic characteristics in the inelastic range

Two nonlinear procedures are contained in the *Guidelines*. These are a simplified Nonlinear Static Procedure (NSP) and a more detailed Nonlinear Dynamic Procedure (NDP). Nonlinear procedures may be used in the rehabilitation analysis of any structure. They should be used whenever the results of a linear procedure indicate that DCRs for critical actions of primary components are substantially in excess of 2.0, and in particular, when the distribution of these inelastic demands throughout the structure is nonuniform. An irregular distribution of DCRs based on a linear procedure indicates that the structure has the potential to form inelastic soft stories, or inelastic torsional instabilities. When such conditions exist, elastic analyses cannot predict the distribution of earthquake demands with any accuracy. A nonlinear procedure should be used in these cases.

C2.9.2.1 Nonlinear Static Procedure (NSP)

This static, sequential nonlinear procedure approach avoids many of the inaccuracies inherent in the linear methods by permitting direct, although approximate, evaluation of the inelastic demands produced in the building by the design earthquake. As with the linear procedures, a mathematical model of the building, representing both the existing and new elements, is constructed. However, instead of performing an elastic analysis of the response of the structural model to specified ground motion, an incremental nonlinear analysis is conducted of the distribution of deformations and stresses throughout the structure as it is subjected to progressively increased lateral displacements. Acceptance criteria include permissible deformation (for example, elongations, drifts, and rotations) and strength demands on common elements and components for different Performance Levels. By comparing the results of the incremental force-displacement analysis (“pushover”) with these acceptance criteria, it is possible to estimate limiting overall structural displacements at which each desired Structural Performance Level can be achieved. Overall displacement demands likely to be produced on the structure by the design earthquake(s) are then approximated using simplified general relationships between elastic spectral response and inelastic response. These relationships take into account, in an approximate manner, the effects of period lengthening, hysteretic damping, and soil structure interaction.

The NSP is generally a more reliable approach to characterizing the performance of a structure, at a given level of excitation, than are the linear procedures. However, it is not an exact approach. It cannot accurately account for the changes in dynamic response and in inertial load patterns that develop in a structure as it degrades in stiffness. Further, it cannot account for the effects of higher mode response in an accurate manner. For this reason, the *Guidelines* recommend that when the NSP is utilized on a structure that has significant higher mode participation in its response, the LDP should also be employed to verify the adequacy of the design. When this approach is taken, somewhat less restrictive criteria are permitted for the LDP than are normally associated with its use, recognizing the significantly improved knowledge of the building’s probable seismic response that is obtained by performing both analysis procedures.

Despite the above-noted limitations on the accuracy of the NSP, it is still generally considered to provide a better estimate of the probable performance of structures than the linear procedures alone. The inelastic force and displacement demands on structural components are directly—albeit approximately—calculated. Therefore, when using this approach it is possible to directly use test data contained in the literature or performed on a project-specific basis to set permissible levels of demand, rather than relying on the less accurately developed *m* values used as acceptance criteria in the linear procedures.

Since the nonlinear procedures more accurately predict demands on individual components than do the linear procedures, acceptance criteria have been developed with less inherent margin. Accordingly, it is expected that the application of this technique will often result in rehabilitation designs that require less remedial work to the building than do the linear procedures. Consequently, the nonlinear procedures are an excellent way to conduct the more detailed evaluations of a building suggested in FEMA 178 (BSSC, 1992a).

Although only a single Nonlinear Static Procedure (NSP) is presented in the *Guidelines*, a number of related approaches are currently in use. These include the Capacity Spectrum Method (Department of the Army, Navy, and Air Force, 1986) and the Secant Modulus Method (Kariotis et al., 1994). Several of these approaches can estimate the effects of higher modes and changing patterns of inertial forces at increasing response more easily than does the NSP. Such methods may provide more accurate evaluations of probable building response for some structures.

C2.9.2.2 Nonlinear Dynamic Procedure (NDP)

The NDP consists of nonlinear Time-History Analysis, a sophisticated approach to examining the inelastic demands produced on a structure by a specific suite of ground motion time histories. As with the NSP, the results of the NDP can be directly compared against test data on the behavior of representative structural components in order to identify the structure’s probable performance when subjected to a specific ground motion. Potentially, the NDP can be more accurate than the NSP in that it avoids some of the approximations made in the more simplified analysis. Time-History Analysis automatically accounts for higher mode effects and shifts in inertial load patterns as structural softening occurs. In addition, for a given earthquake record, this approach directly solves for the maximum global displacement demand produced by the earthquake on the structure, eliminating the need to estimate this demand based on general relationships.

Despite these advantages, it is believed that the NDP is currently limited in application for a number of reasons. First, currently available computer hardware and software effectively limit the size and complexity of structures that may be analyzed by this technique. At present, there is no general-purpose nonlinear analysis software that will permit practical evaluation of large structures that include elements with the wide range of inelastic constitutive relations actually present in the building inventory. Further, these analyses tend to be highly sensitive to small changes in assumptions with regard to either the character of the ground motion record used in the analysis, or the nonlinear stiffness behavior of the elements. As an example, two ground motion records enveloped by the same response spectrum can produce radically different results with regard to the distribution and amount of inelasticity predicted in the structure.

It is expected that the limitations of software and hardware available to perform these analyses will eventually be resolved. However, sensitivity of the analyses to basic assumptions will remain a problem. In order to reliably apply this approach to rehabilitation design, it is necessary to perform a number of such analyses, using varied assumptions. The sensitivity of the analysis approach to the assumptions incorporated is the principal reason why this method should be used only for projects for which independent review is provided by qualified third-party experts.

The NSP is generally applicable to most building configurations and rehabilitation strategies. The NDP is also suitable for general application, although independent third-party review is recommended.

C2.9.3 Alternative Rational Analysis

During the development of the *Guidelines*, a number of existing analytical techniques for use in seismic rehabilitation design—as well as some that were under development—were evaluated for their applicability to the *Guidelines*. Many of these were found to be applicable to only specific Model Building Types and others to only one Rehabilitation Objective, often different from those contained in the *Guidelines*. Rather than adopting and modifying a number of these individual procedures, the *Guidelines* writers chose to develop the four general-purpose procedures (Linear Static, Linear Dynamic, Nonlinear Static, Nonlinear Dynamic) contained in the *Guidelines* and make them broadly applicable to all Model Building Types and Rehabilitation Objectives. These general-purpose procedures are based largely on many of these other preexisting approaches as well as some under parallel development. The fact that a specific rehabilitation procedure has not been adopted verbatim into the *Guidelines* should not be taken as an indication that the procedure is invalid or should not be used. Such procedures may continue to be used; however, it should not be assumed, without thorough review, that the specific Rehabilitation Objectives of the *Guidelines* may be attained through the use of these alternative procedures.

It is anticipated that as computing technology and the knowledge of structural behavior improve, additional procedures will become available that some engineers will desire to use in seismic rehabilitation. Such use is encouraged. However, independent expert review is recommended as a condition of such use because, like all developmental approaches, these procedures may be limited in applicability; may lead to inappropriate designs in some instances; and may not be developed to a sufficient level of detail for general application. When applying alternative analytical procedures, special caution is advised with regard to the adoption of the acceptance criteria contained in the *Guidelines*. The acceptance criteria contained in the *Guidelines* are specifically intended for use with the analytical procedures contained in the *Guidelines*, and may produce incorrect or meaningless results when applied to alternative analytical approaches.

C2.9.4 Acceptance Criteria

No commentary is provided for this section.

C2.10 Rehabilitation Strategies

The rehabilitation strategy is the basic approach used in mitigating the deficiencies previously identified in the structure. In Simplified Rehabilitation, the strategy is one of mitigating deficiencies relative to FEMA 178 (BSSC, 1992a), often by highly prescriptive techniques, as for example a requirement that sill plates be bolted to foundations. However, in Systematic Rehabilitation, a wide range of strategies may be available, depending on the nature of the specific deficiencies involved. For a given building and set of Rehabilitation Objectives, some strategies will be more or less effective than others, and can result in widely different rehabilitation costs. Complete discussion of the alternative strategies available is beyond the scope of this document; however, the publication *NEHRP Handbook of Techniques for the Seismic Rehabilitation of Buildings* (BSSC, 1992b), provides good background material.

The *Guidelines* allude to the importance of providing redundancy in a structure's lateral-force-resisting system but provide no direct method to evaluate whether sufficient redundancy is present in a structure. Recently adopted codes for new buildings, including the 1997 *Uniform Building Code* (ICBO, 1997) and the *NEHRP Recommended Provisions* (BSSC, 1997) have adopted a specific redundancy coefficient, ρ, that is used to adjust the design seismic forces based on the percentage of the total lateral force resisted by any single component in the structure. This coefficient varies from a value of 1.0, for highly redundant structures, to a value of 1.5 for structures with very limited redundancy. The effect of this coefficient is to provide greater margin against failure for structures that rely heavily on the resistance provided by only a few elements. This concept was not specifically adopted by the *Guidelines*. However, it may be worth considering, particularly when rehabilitating buildings with nonredundant systems. The ρ coefficients adopted by the 1997 *UBC* (ICBO, 1994) and *NEHRP Recommended Provisions* (BSSC, 1997) documents could be directly used with the *Guidelines* to account for redundancy effects in an explicit, if not rigorous manner. For the linear procedures, this could be done by directly multiplying the base shear forces by the ρ coefficient. For the NSP, this could be done by multiplying the target displacement by this coefficient. For the NDP, it would be necessary to multiply the ground motion records by the coefficient.

C2.11 General Analysis and Design Requirements

This section provides guidelines for controlling important seismic performance attributes, such as continuity and interconnection of elements, that are not directly evident as potential deficiencies from an analytical evaluation. The requirements are mostly based on parallel provisions contained in the *NEHRP Provisions*.

C2.11.1 Directional Effects

This section requires that a building be demonstrated to be capable of resisting ground motion incident from any direction. For structures that are rectangular or nearly rectangular in plan, analysis of building response about the two principal orthogonal building axes is sufficient. For buildings of unusual shape, analyses of building response to applied ground motion incident from other directions may be required.

C2.11.2 P-Δ Effects

Earthquake-induced collapse of buildings that experience excessive drift can occur as a result of secondary stresses attributable to the P-Δ effect. Equation 2-14 in the *Guidelines* uses a first-order linear approximation of P-Δ effects. More accurate approaches, directly incorporating elastic stability theory, could also be employed.

C2.11.3 Torsion

The effects of torsion are much more important to seismic performance than they are to wind resistance. Engineers familiar with wind design but not with seismic design may overlook torsional effects by utilizing two-dimensional analysis techniques. This section reminds the engineer of the importance of capturing torsional behavior in the analysis.

C2.11.4 Overturning

In addition to creating lateral shear forces in structures, earthquake ground motion also results in a tendency for structures, and individual vertical elements of structures, to overturn about their bases. Although actual overturning of structures due to earthquake ground motion is very rare, overturning effects do have the potential to result in significant stresses in structures, which have caused local and even global failures. In the design of new buildings, earthquake effects, including overturning, are evaluated for lateral forces that are much lower (reduced by the factor *R*) than those the structure actually will experience. The designer typically evaluates the effects of overturning in one of two ways:

1. For elements that are provided with positive attachment between levels, such as reinforced concrete or masonry shear walls, or moment-resisting frames, the overturning effects are resolved into component forces, e.g., flexure at the base of a wall pier; and the component is then proportioned with adequate strength to resist these overturning effects at the reduced force levels.

2. Some elements, such as wood shear walls and foundations, may not be provided with positive attachment to lower levels. For these elements, an overturning stability check is typically performed. If the element supports sufficient dead load to remain stable under the overturning effects of the design lateral forces, then the design is deemed adequate. However, if it is determined that the element has inadequate dead load to remain stable against overturning, then hold-downs, piles, or other types of uplift anchors are provided to resist overturning effects.

In the linear procedures contained in the *Guidelines*, the lateral forces used to evaluate the performance of a structure have not been reduced by the *R*-factor, as they typically are in the design of new buildings. As a result, the computed effects of overturning will be more severe, if calculated in the typical manner, than is the case during the design of new buildings. Though the procedure used to design new buildings for earthquake-induced overturning is not completely rational, it has resulted in successful performance. Therefore, it was felt that it would be inappropriate for the *Guidelines* to require that structures and elements of structures remain stable for the full lateral forces used in the linear procedures. Instead, just as with new buildings, the designer must determine if positive direct attachment will be needed to resist overturning effects, or alternatively, if sufficient dead load is present on the element to resist these effects. If dead loads are used to resist overturning without supplemental positive direct attachment, then overturning is treated as a force-controlled behavior and the overturning demands are reduced to an estimate of the real overturning demands that can be transmitted to the element, considering the overall limiting strength of the structure. As with the design of new buildings, a stability evaluation is performed, and in addition, the element is evaluated for adequacy to resist bearing stresses at the toe, about which it is being overturned.

If it is determined that there is inadequate dead load on an element to resist overturning effects, then positive structural attachment must be provided to resist overturning effects. Examples of such attachment included piles or caissons with uplift anchors at foundations; dowels or reinforcing that extends between the boundary elements of a shear wall at one level to that in the level below; and hold-down hardware attached to the end stud of a timber shear wall in one level and that in the level below. The individual materials chapters provide guidance as to whether each of these elements is to be treated as deformation-controlled or force-controlled for evaluation and design purposes.

When nonlinear procedures are performed, the effects of overturning can be directly investigated in the mathematical model. This is accomplished by releasing the rotational restraint on elements, once the demands on the elements exceed the stabilizing forces. One of the principal benefits of the nonlinear procedures is that they permit a more realistic evaluation of overturning effects than do the linear procedures.

C2.11.5 Continuity

A continuous structural system with adequately interconnected elements is one of the most important prerequisites for acceptable seismic performance. The requirements of this section are similar to parallel provisions contained in the BSSC (1995) provisions.

C2.11.6 Diaphragms

The concept of a diaphragm chord, consisting of an edge member provided to resist diaphragm flexural stresses through direct axial tension or compression, is not familiar to many engineers. Buildings with solid structural walls on all sides often do not require diaphragm chords. However, buildings with highly perforated perimeter walls do require these components for proper diaphragm behavior. This section of the *Guidelines* requires that these components be provided when appropriate.

A common problem in buildings that nominally have robust lateral-force-resisting systems is a lack of adequate attachment between the diaphragms and the vertical elements of the lateral-force-resisting system to affect shear transfer. This is particularly a problem in buildings that have discrete shear walls or frames as their vertical lateral-force-resisting elements. This section provides a reminder that it is necessary to detail a formal system of force delivery from the diaphragm to the walls and frames.

Diaphragms that support heavy perimeter walls have occasionally failed in tension induced by out-of-plane forces generated in the walls. This section is intended to ensure that sufficient tensile ties are provided across diaphragms to prevent such failures. The design force for these tensile ties, taken as $0.4S_{XS}$ times the weight, is an extension of provisions contained in the 1994 *Uniform Building Code* (ICBO, 1994). In that code, parts and portions of structures are designed for a force calculated as C_pIZ times the weight of the component with typical values of C_p being 0.75 and Z being the effective peak ground acceleration for which the building is designed. The 1994 UBC provisions use an allowable stress basis. The *Guidelines* use a strength basis. Therefore, a factor of 1.4 was applied to the C_p value, and a factor of 1/(2.5) was applied to adjust the Z value to an equivalent S_{XS} value, resulting in a coefficient of 0.4.

C2.11.7 Walls

Inadequate anchorage of heavy masonry and concrete walls to diaphragms for out-of-plane inertial loads has been a frequent cause of building collapse in past earthquakes. Following the 1971 San Fernando earthquake, the *Uniform Building Code* adopted requirements for positive direct connection of wall panels to diaphragms, with anchorage designed for a minimum force equal to ZIC_pW_p. In this equation, the quantity ZIC_p represents the equivalent out-of-plane inertial loading on the wall panel and typically had a value that was 75% of the effective peak ground acceleration for the site. This section of the *Guidelines* imposes design provisions based on observations made following the 1994 Northridge earthquake. Failures occurred in a number of buildings meeting the requirements of the building code in effect at that time. Actual strong motion recordings in buildings with flexible diaphragms indicates that these diaphragms amplify the effective peak ground accelerations by as much as three times. For a site with an effective peak horizontal ground acceleration of 0.4g (S_{XS} = 1.0g), this would correspond to an inertial acceleration of the wall panels of 1.2g. The χ coefficients contained in Table 2-18 were derived from this relationship, providing for somewhat greater factors of safety at the Immediate Occupancy Performance Level and reduced factors of safety at the Collapse Prevention Performance Level. More thorough treatment of this subject may be found in Hamburger and McCormick (1994).

These failures also extended to walls of construction other than concrete and masonry, even though earthquake-induced collapse of such walls is rare. This can be considered a matter of collateral rehabilitation for wind-load resistance. Lack of adequate out-of-plane anchorage for wood stud walls has occasionally resulted in failures in tornadoes and high wind storms. Use of the *Guidelines* will reduce the vulnerability of wood buildings to such failures.

C2.11.8 Nonstructural Components

There is a tendency for structural engineers to address structural deficiencies but neglect nonstructural problems, which can have life safety implications as well important economic implications. This section serves as a reminder of the importance of addressing these issues.

C2.11.9 Structures Sharing Common Elements

Structures that share elements in common are particularly problematic. Where practical, the best approach for such structures may be to tie the buildings together, such that they behave as one structure. Alternate approaches could include ensuring that differential displacements of the two structures cannot result in a collapse condition, or providing redundant structural elements such that if failure of the shared element occurs, stability is still maintained.

C2.11.10 Building Separation

Buildings that have inadequate separation can impact each other, or “pound” during response to ground motion. This can drastically alter the buildings’ performance and should be considered in rehabilitation design. The first step is to determine if pounding is likely to occur. One approach to determining the likelihood of pounding is to take the absolute sum of the expected lateral deflections of each building at the location of potential impacts, and if the available separation of the buildings is greater than this amount, assume that pounding does not occur. The implicit assumption in such an approach is that at some point during the buildings' response to the ground motions, the structures will become completely out of phase and require a separation of the calculated amount.

An alternative approach to evaluating the potential for pounding, termed the spectral difference approach (Jeng et al., 1992), directly accounts for the incoherence of multimode response, and the fact that both structures are unlikely to experience the maximum response of all modes at the same instant, completely out of phase. This approach requires knowledge of the natural modes of both structures. Since such information is often not available for one of the structures, the *Guidelines* adopt a somewhat simpler approach of using a square root of the sum of the squares (SRSS) combination of estimated structural lateral deflections to check the adequacy of building separation. This approach requires only an estimate of the lateral deflection of the adjacent structure (which can be based on general rules of thumb), rather than performance of a modal analysis on each structure. However, it accounts for the fact that some incoherence of response is likely to occur and permits less than the full separation required if both structures are assumed to behave completely out of phase.

When two adjacent structures pound, this can drastically alter the dynamic response of both structures, resulting in a change in the effective mode shapes and period of each, as well as the pattern and magnitude of inertial demands and deformations induced on both structures. The *Guidelines* permit buildings rehabilitated to the BSO to experience pounding as long as the effects of such pounding are adequately accounted for in the design.

Approximate methods of accounting for these effects can be obtained by performing nonlinear Time-History Analyses of both structures (Johnson et al., 1992). Approximate elastic methods for evaluating these effects have also been developed (Kasai et al., 1990) and are presented in the literature.

One of the most dangerous aspects of pounding is the potential for local destruction of critical structural components at the point of impact. As an example, the floor slabs of one structure can create a knife-edge effect against the columns of an adjacent structure, resulting in potential for partial or total collapse. Where such behavior is plausible, consideration should be given to altering the response of both structures such that impacts do not occur, or providing redundant elements at a location away from the zone of impact to replace components that may fail due to the impact effects.

Buildings that are likely to experience significant pounding should not be considered to be capable of meeting Enhanced Rehabilitation Objectives. This is because significant local crushing of building components is likely to occur at points of impact. Further, the very nature of the impact is such that high-frequency shocks can be transmitted through the structures and potentially be very damaging to architectural elements, and mechanical and electrical systems. Such damage is not consistent with the performance expected of buildings designed to Enhanced Rehabilitation Objectives.

C2.12 Quality Assurance

This section indicates the minimum construction quality assurance (QA) measures that should apply to any seismic rehabilitation project, regardless of the Rehabilitation Objectives, project complexity, or costs. The intent of these requirements is to assure that those resources invested in seismic rehabilitation result in the intended improvement in seismic reliability. Failure to properly implement rehabilitation measures can result in no improvement in the existing building's seismic resistance, or worse, a lessening of its resistance. For some projects that are highly complex, use unusual technologies, have exacting construction tolerance requirements, or are intended to achieve Enhanced Rehabilitation Objectives, it may be appropriate to implement measures beyond those contained in the *Guidelines*. The structural design professional of record should establish these on a project-specific basis.

C2.12.1 Construction Quality Assurance Plan

The development of a Quality Assurance Plan (QAP) is the only design period quality assurance measure specifically prescribed by the *Guidelines*; however, it is not the only design period quality assurance measure that should be taken. In addition to development of a QAP, the design professional should also take a number of other precautions to maintain the quality of the project. These include ensuring that:

- An adequate understanding of the existing construction characteristics of the structure has been developed, prior to embarking on a rehabilitation design.

- The construction documents adequately represent the intent of the design calculations and analyses, and these analyses and calculations are accurate.
- The construction documents are clear with regard to the existing conditions of the structure and the modifications that are to be made to it as part of the rehabilitation work.
- The construction documents specify the construction of details that are constructible, and specify the use of materials and methods that can be readily performed to attain the desired results.

These measures are not specified in the *Guidelines*, as they are a function of individual design office practice. However, they are an important part of any project.

C2.12.2 Construction Quality Assurance Requirements

C2.12.2.1 Requirements for the Structural Design Professional

In addition to other inspections and observations that may be made during the construction period, the design professional in responsible charge of development of the seismic evaluation, analyses, and rehabilitation design for the building should make site observations during the construction process. This is even more important in rehabilitation construction than it is in new construction. Often it is not practical to fully investigate the existing structural conditions of a building during the rehabilitation design. Consequently, when selective demolition of finishes occurs during the construction period, it is commonly found that the configuration, condition, and strength of some components of the existing building are significantly different than assumed in the rehabilitation design. It is imperative that the design professional become aware of any such deviations from the design assumptions so that the validity of detailing contained on the construction drawings, and perhaps the overall design, can be confirmed or adjusted as appropriate. Adjustments that may be necessary can range from minor revisions of individual details to complete alteration of the design concept.

Structural observation by the design professional is also extremely important in rehabilitation projects because many of the details used for rehabilitation construction can be significantly different from those commonly used in the construction of new buildings. Therefore, there is somewhat greater potential for construction error in the implementation of the details. Structural observation is an important tool for assuring that construction work is performed in accordance with the design intent.

C2.12.3 Regulatory Agency Responsibilities

No commentary is provided for this section.

C2.13 Alternative Materials and Methods of Construction

This section provides guidance for developing appropriate data to evaluate construction materials and detailing systems not specifically covered by the *Guidelines*. The *Guidelines* specify stiffnesses, *m* coefficients, strength capacities, and deformation capacities for a wide range of element and component types. To the extent practical, the *Guidelines* have been formatted to provide broad coverage of the various common construction types present in the national inventory of buildings. However, it is fully anticipated that in the course of evaluating and rehabilitating existing buildings, construction systems and component detailing practices that are not specifically covered by the *Guidelines* will be encountered. Further, it is anticipated that new methods and materials, not currently in use, will be developed that may have direct application to building rehabilitation. This section provides a method for obtaining the needed design parameters and acceptance criteria for elements, components, and construction details not specifically included in the *Guidelines*.

The approach taken in this section is similar to that used to derive the basic design parameters and acceptance criteria contained in the *Guidelines* for various elements and components, except that no original experimentation was performed. The required story-force deformation curves were derived by the *Guidelines* developers, either directly from research testing available in the literature, or based on the judgment of engineers knowledgeable in the behavior of the particular materials and systems.

C2.13.1 Experimental Setup

The *Guidelines* suggest performing a minimum of three separate tests of each unique component or element. This is because there can be considerable variation in the results of testing performed on "identical" specimens, just as there is inherent variability in the behavior of actual components and structural elements in buildings. The use of multiple test data allows some of the uncertainty with regard to actual behavior to be defined.

A specific testing protocol has not been recommended by the *Guidelines*, as selection of a suitable protocol is dependent on the anticipated failure mode of the assembly as well as the character of excitation it is expected to experience in the real structure. In one widely used protocol (ATC, 1992), the specimen is subjected to a series of quasi-static, fully reversed cyclic displacements that are incremented from displacement levels corresponding to elastic behavior, to those at which failure of the specimen occurs. Other protocols that entail fewer or greater cycles of displacement, and more rapid loading rates, have also been employed. In selecting an appropriate test protocol, it is important that sufficient increments of loading be selected to characterize adequately the force-deformation behavior of the assembly throughout its expected range of performance. In addition, the total energy dissipated by the test specimen should be similar to that which the assembly is anticipated to experience in the real structure. Tests should always proceed to a failure state, so that the margin against failure of the assembly in service can be judged.

If the structure is likely to be subjected to strong impulsive ground motions, such as those that are commonly experienced within a few kilometers of the fault rupture, consideration should be given to using a protocol that includes one or more very large displacements at the initiation of the loading, to simulate the large initial response induced by impulsive motion. Alternatively, a single monotonic loading to failure may be useful as a performance measure for assemblies representing components in structures subject to impulsive motion.

C2.13.2 Data Reduction and Reporting

It is important that data from experimental programs be reported in a uniform manner so that the performance of different subassemblies may be compared. The data reporting requirements specified in the *Guidelines* are the minimum thought to be adequate to allow development of the required design parameters and acceptance criteria for the various Systematic Rehabilitation Procedures. Some engineers and researchers may desire additional data from the experimentation program to allow calibration of their analytical models and to permit improved understanding of the probable behavior of the subassemblies in the real structure.

C2.13.3 Design Parameters and Acceptance Criteria

The *Guidelines* provide a multistep procedure for developing design parameters and acceptance criteria for use with both the linear and nonlinear procedures. The basic approach consists of the development of an approximate story lateral-force-deformation curve for the subassembly, based on the experimental data.

In developing the representative story lateral-force-deformation curve from the experimentation, use of the "backbone" curve is recommended. This takes into account, in an approximate manner, the strength and stiffness deterioration commonly experienced by structural components. The backbone curve is defined by points given by the intersection of an unloading branch and the loading curve of the next load cycle that goes to a higher level of displacement, as illustrated in Figure C2-5.

C2.14 Definitions

No commentary is provided for this section.

C2.15 Symbols

No commentary is provided for this section.

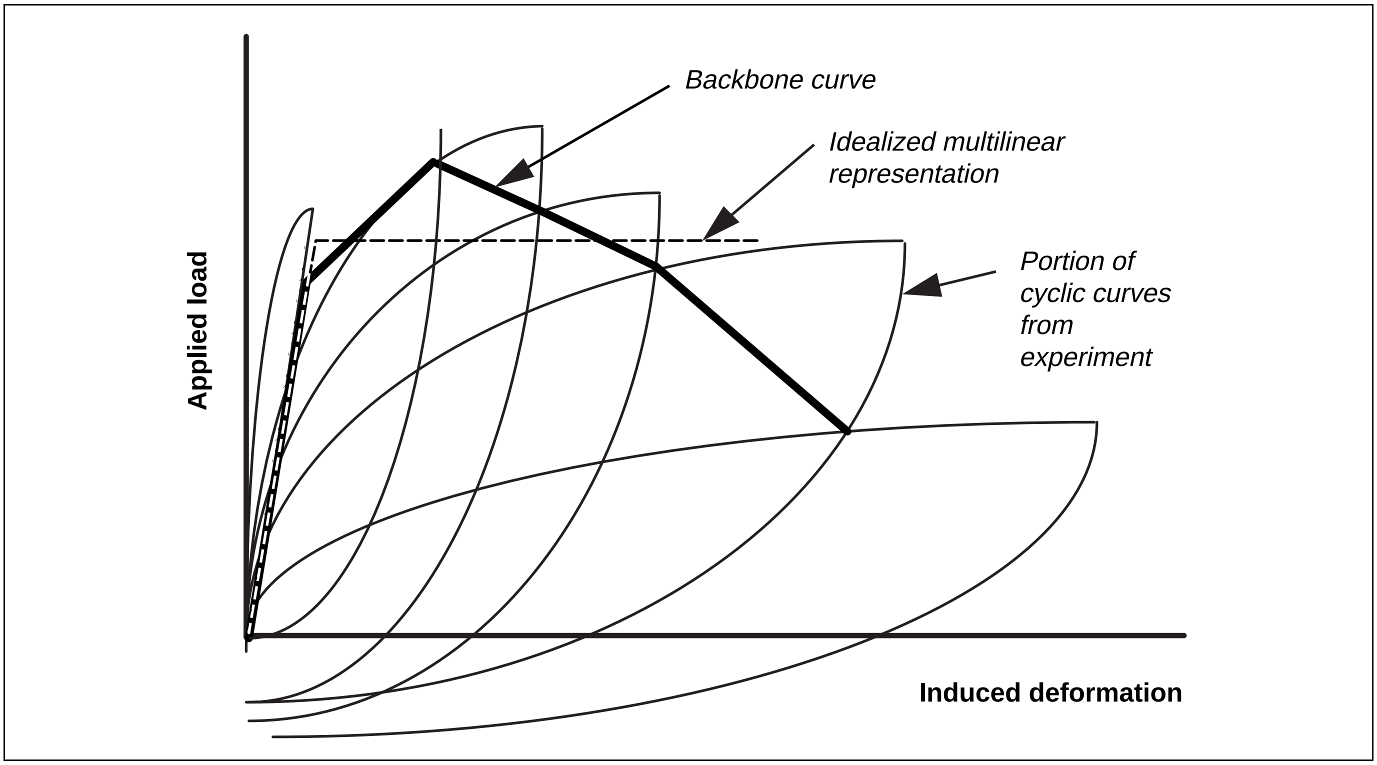

Figure C2-5 Idealized Force versus Displacement Backbone Curve

C2.16 References

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