Performance Based Seismic Engineering of Buildings

prepared by

Structural Engineers Association of California

Vision 2000 Committee

Chris Poland, Chair
James Hill, Vice Chair
Roland Sharpe, Vice Chair

delegates:

Vitelmo Bertero
Joseph Nicoletti
Ronald Hamburger
Nabih Youssef
Anthony Court
Scott Stedman
Ajit Virdee
Robert Chittenden
Richard Phillips

editor:

Jeffrey Soulages

ad hoc members:

Robert Thacker
Vilas Mujumdar
Charles Thiel, Jr.

ex-officio members:

Ronald Nelson
Arthur Ross
Jane Andersen

contractors:

Richard Fallgren

555 University Avenue, Suite 126
Sacramento, CA

April 3, 1995

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PART 1: INTERIM RECOMMENDATIONS

Part 1 of the Vision 2000 Report presents interim recommendations for engineering procedures to obtain buildings of predictable and defined seismic performance. The volume includes a definition of performance based engineering, definitions of a standard set of performance levels, earthquake hazard levels, and performance objectives for the seismic resistive design and evaluation of buildings, and recommended uniform procedures for performance based engineering within the State of California using currently available technology and design practices.

PART 2: CONCEPTUAL FRAMEWORK

Part 2 of the Vision 2000 Report presents a conceptual framework for performance based engineering to be incorporated into future seismic design codes. The conceptual framework outlines the proposed scope of structural design needed to achieve structures of predictable performance. The volume includes a number of possible design approaches as well as possible acceptability analysis procedures.

PART 3: PRELIMINARY NORTHRIDGE LESSONS

Part 3 of the Vision 2000 Report presents preliminary lessons learned about the performance of buildings in the Northridge Earthquake. The goal of this information is to calibrate the performance based engineering provisions developed by Vision 2000. The volume includes a database of summaries about lessons learned from the earthquake extracted from existing published reports, as well as a database of post-earthquake observations of specific buildings conducted by various consulting engineering offices and public agencies.

PART 4: MOVING THE BLUE BOOK TOWARD PERFORMANCE BASED ENGINEERING

Part 4 of the Vision 2000 Report presents recommendations on how to incorporate new performance based engineering concepts and lessons from Northridge into future editions of the SEAOC Blue Book and building code. The volume includes results of short-term efforts such as the 1994 UBC Emergency Code Change, 1995 Blue Book changes, and 1996 UBC Supplement changes, as well as a "road map" of how performance based engineering concepts proposed by Vision 2000 could be incorporated into future design practice using the SEAOC Blue Book.

Structural Engineers Association of California

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Part 1: Interim Recommendations

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California Office of Emergency Services

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SEAOC Vision 2000 Committee

In recognition of the need for a new generation of design procedures, the Structural Engineers Association of California (SEAOC) has formed a special committee to look into the future and develop a framework for procedures that will yield structures of predictable seismic performance. This committee, called Vision 2000, first started their deliberations in 1992 and continues today with a multifaceted effort that is aimed at both the final goal and at the process that will deliver the profession to that end. While Vision 2000 intends to identify the framework, others are expected to translate it into design guidelines and codes as appropriate. Within SEAOC, the Seismology Committee will incorporate the new concepts, as they are available and appropriate, into future editions of the SEAOC Blue Book.

The first step of the Vision 2000 Committee was to establish the project parameters. These project parameters were incorporated into a Goal/Mission Statement. The goal of the Vision 2000 is as follows:

> **The goal of the Vision 2000 Committee is to develop the framework for procedures that yield structures of predictable seismic performance.**

This framework will explicitly address life-safety, damageability, and functionality issues. The intent will be to produce structures whose seismic performance is predictable with a reasonable degree of confidence provided the actual materials and construction accurately reflect the design.

The second step was to establish a work plan and timeline for both the short-term phase of the project and for the mid-term and long-term phases of the project. A copy of this Workplan is available from the SEAOC Office.

The third step was to create the Vision 2000 Report which includes the results from all phases of the short-term work of the committee. This report includes: performance based engineering definitions, interim recommendations on how to apply performance based engineering concepts using today's available standards, a conceptual framework for a future performance based design code, preliminary lessons from the Northridge Earthquake, and a road map on moving the SEAOC Blue Book towards performance based engineering concepts. This report is divided into four parts as follows:

Part 1: Interim Recommendations
Part 2: Conceptual Framework
Part 3: Preliminary Northridge Lessons
Part 4: Moving the Blue Book Towards Performance Based Engineering

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1. INTRODUCTION

1.1 Purpose

This report presents interim recommendations for engineering procedures to obtain buildings of predictable and defined seismic performance. It was prepared by the Structural Engineers Association of California (SEAOC), Vision 2000 Committee, under funding provided by the California Office of Emergency Services. The interim recommendations are intended to be used by practicing structural engineers engaged in the design of new buildings and the evaluation, repair and upgrade of existing buildings. Portions of this report may also be useful for building officials, public policy makers, regulatory agencies, and building owners and tenants.

1.2 Intent

The intent of these interim performance based engineering recommendations is as follows:

1. Define a series of standard performance levels for the seismic resistive design and construction of buildings. A performance level is a limiting damage state. Damage to structural elements, nonstructural elements, building contents, and site utilities are considered in the performance level definitions. Performance levels should be selected based upon the safety, economic and societal consequences of the given damage state occurring.

2. Define a series of reference earthquake hazard and design levels. Within a defined period of time, a given seismic source zone may produce many small magnitude events, several moderate magnitude, and a few large magnitude events. The goal of performance based engineering is to control the seismic risk associated with a building to pre-determined levels of acceptability. This risk is a result of the probability that an earthquake of given severity will affect the building coupled with the consequences of the building's response to that earthquake. The reference earthquake design levels are discrete events, within the entire spectrum which may affect a building site, that should be used to check the risk level inherent in a building design.

3. Define a series of uniform design performance objectives for buildings of different occupancies and uses. These design performance objectives consist of a series of standard performance levels for each of the reference earthquake design levels. Buildings designed, constructed and maintained to achieve these performance levels will have a quantifiable seismic risk.

4. Recommend uniform engineering procedures for performance based engineering within the State of California, using currently available technology and design practices.

1.3 Background

Almost all building construction in the United States today is regulated under the provisions of a building code that includes provisions intended to provide for seismic resistive construction. The major building codes currently in effect include the *Uniform Building Code* (ICBO-1994), *BOCA Code* (BOCA-1993) and *Standard Building Code* (SBCCI-1993). Each of these in turn is based either on the SEAOC Blue Book (SEAOC-1990) or NEHRP Provisions (BSSC-1991), resource documents which themselves are closely tied to ATC 3-06 (ATC-1978). Each of these codes is therefore very similar with regard to seismic resistive design and construction provisions.

The primary goal of these codes is to protect life-safety by providing for buildings with sufficient integrity, strength and toughness to resist collapse or the generation of large falling debris, in very severe, but relatively infrequent earthquakes. Secondary goals include control of property damage and maintenance of function in more moderate events, which are expected to occur more often. These codes have been empirically developed, based on observations of actual damage which has occurred to structures in past earthquakes and extensive research at various universities. Following each major earthquake, engineers have observed the damage sustained by buildings, with particular interest in the performance of structures conforming to the then current code. Where unacceptable damage has been observed, the building code provisions have been modified to prevent the recurrence of such damage in future events. As a result, current building codes enforced in seismically active regions of the United States are believed to provide good levels of life-safety protection in buildings which are properly designed and constructed. However, these codes appear less reliable with regard to the secondary goals of minimizing property damage in moderate and small events for some types of construction.

In recent years, California, due to its active seismicity and relatively modern inventory of buildings, has provided a good laboratory in which to judge the effectiveness of these building codes in meeting their intended performance goals. In 1989, the M7.1 Loma Prieta Earthquake affected the San Francisco Bay area. Although this was a relatively large magnitude event, the location and fault rupture characteristics were such that strong ground motion had a relatively short duration and the most densely developed regions of the Bay Area experienced ground accelerations that were 1/4 to 1/2 that which would be anticipated in the largest expected earthquake. Despite the relatively limited levels of ground shaking produced, the Loma Prieta Earthquake caused more than $7 billion in damage with some modern structures, designed to recent editions of the building code, damaged so severely that extended loss of use occurred. Although no loss of life occurred in modern buildings designed to recent building codes, the economic loss which occurred was judged both by the structural engineering profession and public policy makers as too large for this moderate event. A need was identified for new building design and construction procedures, which could better meet society's requirement that property and business interruption losses in moderate earthquakes be controlled to acceptable levels. Also identified was the need to expand the scope of procedures used to evaluate and rehabilitate existing buildings.

SEAOC has been actively engaged in the development of the seismic design and construction provisions contained in the UBC Code for more than 40 years. Much of this work is done through the voluntary efforts of its membership on the SEAOC Seismology Committee. It was realized that the development of a new, performance based methodology for seismic design and construction would require a massive and lengthy effort, beyond the capabilities of the Seismology Committee, given their continued responsibility to maintain and support the existing UBC Code. Therefore, in 1992, the SEAOC board of directors established the Vision 2000 Committee to develop a framework for a next generation, performance based, seismic resistive building code.

The Vision 2000 Committee was constituted from a number of past chairs of the Seismology Committee, as well as SEAOC members who had been active in development of both the Blue Book and NEHRP Provisions. During the period 1992-93, the committee developed a preliminary framework for a next generation, performance based building code. This framework incorporated the adoption of newly developing analytical techniques and methodologies for characterizing the behavior of structural systems subjected to strong ground motion, including energy balance and non-linear analysis methods. At the same time, the committee understood the need for simplified, but reliable procedures for the design and construction of the most common structures in our building inventory. A draft multi-year plan was laid out to pursue the development of this next generation code.

In January of 1994, the M6.7 Northridge Earthquake occurred, resulting in even more severe losses, estimated at approximately $20 billion, than the Loma Prieta Earthquake. Engineers and public policy makers alike again determined that it is unacceptable to experience this magnitude of loss in these relatively frequent and moderate events. Faced with a need to repair and reconstruct many hundreds of buildings, the California Office of Emergency Services contracted with SEAOC to develop recommendations for performance based design and construction procedures which can be used today. This report presents those recommendations and refers to them as Performance Based Engineering.

1.4 Interim Recommendations

Performance Based Seismic Engineering involves the complete design and construction support activities necessary to permit buildings to be constructed that will resist earthquakes of different severities within specified limiting levels of damage. While this is done to various degrees of success now, there is no uniform, written statement of procedure. These interim guidelines are intended to provide such a written statement that will hopefully form the basis for performance based guidelines that will be written in the future.

Performance Based Engineering is a process that begins with the first concepts of a project and lasts throughout the life of the building. It includes selection of the performance objective, determination of site suitability, conceptual design, preliminary design, final design, acceptability checks during design, design review, quality assurance during construction, and maintenance during the life of the building. Each step is critical to the success of the design and must be addressed to a level suitable to the performance objective selected. Performance Based Engineering, not Performance Based Design, is the most suitable title for this process since it encompasses all aspects of the effort and not just those related to design.

The Vision 2000 committee, in conjunction with these interim recommendations, has developed a conceptual framework for performance based engineering that fully defines each of the key aspects of the process, suggests a basic organizational structure and a series of approaches for each. This work is published as Part 2 of the Vision 2000 report.

These interim recommendations have been developed to permit the immediate application of the Performance Based Engineering concepts using available guidelines and standards. The basic definitions of performance objectives in terms of performance levels and earthquake hazards are contained in Chapter 2: Recommended Performance Objectives. The interim recommendations for Performance Based Engineering are contained in Chapter 3.

1.5 Limitations

The practice of performance based seismic engineering presumes the ability of an engineer to predict building performance, given the occurrence of a defined earthquake ground motion. The current state of knowledge and available technology is such that our ability to accurately predict the earthquake performance of a specific building subjected to a defined earthquake ground motion is quite limited, and subject to a number of uncertainties. Principal sources of these uncertainties include:

- definition of the ground motion including, intensity, duration, phasing, and frequency content,
- analysis of the distribution of deformations and stresses produced in the structure in response to the ground motion,
- knowledge of the actual configuration, strengths, deformations, and energy absorption and dissipation capacities of the structure in its as-constructed and maintained condition, and

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- determination of specific damage to structural and nonstructural components, in response to defined ground motions.

Researchers and practicing engineers are actively working to reduce these uncertainties through the development of improved analytical tools for the estimation of ground motion and structural response, laboratory testing of structural assemblies and our continued observations of the damage caused to real structures by earthquakes. While our ability to predict performance today is much better than it was just 10 years ago, reliability is still limited.

Recommendations contained in this report are intended to provide engineers with guidance on how to better achieve structures with predictable and defined seismic performance. There is, of course, no guarantee that specific structures designed in accordance with these recommendations will achieve the desired performance. Users of this document are cautioned to exercise judgment and care in the application of the recommendations contained herein. This document is not intended to be, and should not be used as, a substitute for currently prevailing building codes.

The Structural Engineers Association of California, its membership, and the Vision 2000 committee offer no warranty with regard to the performance of buildings designed in accordance with these recommendations.

2. RECOMMENDED PERFORMANCE OBJECTIVES

2.1 Introduction

Severe earthquakes are relatively infrequent events, which may or may not ever occur within the life of a building. While it may technically be feasible to design and construct buildings such that they do not experience any damage in the most severe earthquake events which could effect them, it is generally considered unnecessary and uneconomical to do so. Therefore, an engineering design philosophy has evolved over the years in which design is performed with the anticipation that severe earthquakes will cause some damage to most buildings. The intent of earthquake resistive design therefore has become one of attempting to limit the damage experienced by a building to levels which are deemed acceptable by structural engineers. Historically, damage which did not result in potential loss of life was deemed acceptable for most structures.

Under the concept of performance based engineering, the acceptability of varying levels of damage is determined based on the consequences of this damage to the user community and the frequency with which such damage occurs. If a structure is provided with enhanced earthquake resistance, this reduces costs over its life related to loss of life, business interruption, and repair/reconstruction following earthquakes. In an optimal situation, the costs related to providing initial earthquake resistance in a structure would be balanced against the costs related to damage in future earthquakes. The goal is to minimize earthquake related cost to the building owner over the life of the building. This is done by considering a set of performance objectives.

A design *performance objective* is an expression of the desired performance level for the building for each earthquake design level. Design performance objectives should be selected based on the building's occupancy, the importance of functions occurring within the building, economic considerations including costs related to building damage repair and business interruption, and considerations of the potential importance of the building as a historic or cultural resource. On a community level, public policy makers may wish to set performance objectives based on the combined effects of entire classes of buildings.

A *performance level* is an expression of the maximum desired extent of damage to a building, given that a specific earthquake design level affects it. The condition of structural elements, nonstructural elements and contents are all considered in the performance levels, as are availability of site utilities necessary to building function. Performance levels are expressed in two different, but related ways:

1. In qualitative terms meaningful to the lay public including building owners, occupants and public policy makers.

2. In technical terms useful for engineering design and evaluation, including the extent of deterioration and degradation permissible to individual structural and nonstructural elements, as well as overall behavior of the structural system.

Four individual performance levels are defined. Each specific performance level defines the limit to a range of damage states which meet basic user needs such as continuity of function, suitability for repair, safety, etc. The four performance levels are:

- Fully Operational
- Operational
- Life-safe
- Near Collapse

Earthquake hazards include ground shaking, ground fault rupture, soil liquefaction, lateral spreading, landsliding and differential settlement. Each of these effects can result in building damage, and therefore affect the performance level achieved by the building. The extent to which these hazards may affect building performance is dependent on their individual severity, which in turn is a function of earthquake magnitude, distance of the actual zone of fault rupture from the site, direction of fault rupture propagation, the geologic makeup of the region and the unique geologic conditions of the individual site. The effects of each of these hazards should be considered, and as appropriate, specifically investigated as part of the performance based engineering process.

For any site, there is a continuous spectrum of earthquakes which may affect it, ranging from distant, small magnitude events that produce negligible hazards, to local, large magnitude events which produce hazards that have the potential of being highly damaging. In general, there is a high probability that a site will experience low-hazard events within the lifetime of a building and a lower probability that it will experience high-hazard events. Performance based engineering seeks to control the levels of damage experienced by a building over the full spectrum of events which may occur. In order to permit practical application of this approach, it is necessary to select a series of discrete earthquake events from among the entire spectrum which may occur. These should represent the range of earthquake severity for which a particular building performance is desired. These discrete earthquake events are termed *earthquake design levels*. Although they are useful for purposes of design, the building may or may not ever be subjected to the exact earthquake represented by a particular earthquake design level.

2.2 Performance Level Definitions

This section defines the four design performance levels in terms meaningful to the building user community. Performance is expressed with regard to suitability of the building for function and occupancy, the extent to which life-safety is protected and the necessity/practicality of effecting repairs on the structure and restoring it to service. It should be noted that buildings are complex assemblies containing a number of interdependent systems. Therefore, within the intent of a given performance level, there is a wide range of potential damage states, which could qualify as meeting the intent of the level. The definitions given below, define each of these ranges in terms of the maximum damage and lowest performance which would qualify within the intent of the performance level. Figure 2-1 illustrates the entire spectrum of damage states which a building may experience when subjected to ground motions of increasing severity. This figure also relates the various ranges of damage which are permissible within the intent of each of the limiting performance level and for easy reference, defines a numerical damage index. These definitions are also suitable for cataloging the performance of buildings after earthquakes.

A. *Fully Operational*

A performance level in which essentially no damage has occurred. If a building responds to an earthquake within this performance level, the consequences to the building user community are negligible. The building remains safe to occupy and it is expected that post-earthquake damage inspectors utilizing the ATC-20 (ATC-1989) methodology would post the building with a green placard. The building is occupiable and all equipment and services related to the building's basic occupancy and function are available for use. In general, repair is not required.

B. *Operational*

A performance level in which moderate damage to nonstructural elements and contents, and light damage to structural elements has occurred. The damage is limited and does not compromise the safety of the building for occupancy. Post-earthquake damage inspectors utilizing the ATC-20 methodology would be expected to post the building with a green placard. It would be available for occupancy for its normal intended function, immediately following the earthquake, however, damage to some contents, utilities and nonstructural components may partially disrupt some normal functions. Back-up systems and procedures may be required to permit continued use. Repairs may be instituted at the owners' and tenants' convenience.

C. *Life-safe*

A performance level (damage state) in which moderate damage to structural and nonstructural elements, and contents has occurred. The structure's lateral stiffness and ability to resist additional lateral loads has been reduced, possibly to a great extent, however, some margin against collapse remains. No major falling debris hazards have occurred. Egress from the building is not substantially impaired, albeit elevators and similar electrical and mechanical devices may not function. In the worst case, post-earthquake damage inspectors, using the ATC-20 methodology, would be expected to post such a building with a yellow placard. In such cases the building would not be available for immediate post-earthquake occupancy. The building would probably be repairable, although it may not be economically practical to do so.

D. *Near Collapse*

An extreme damage state in which the lateral and vertical load resistance of the building have been substantially compromised. Aftershocks could result in partial or total collapse of the structure. Debris hazards may have occurred and egress may be impaired, however, all significant vertical load carrying elements (beams, columns, slabs, etc.) continue to function. In the worst case, post-earthquake damage inspectors, using the ATC-20 methodology, would be expected to post such a building with a red placard. The building will likely be unsafe for occupancy and repair may not be technically or economically feasible.

Table 2-6 describes permissible levels of damage to the various systems and subsystems in buildings, for each of the four performance levels. Also shown are typical transient (during the earthquake response) and permanent (after the earthquake response) drift levels which may be sustained by a building in meeting these performance levels. Permissible drift levels are functions of the structural as well as nonstructural systems. Some structural systems can withstand greater levels of drift without experiencing significant levels of damage than other systems. Similarly, some nonstructural elements may be detailed and installed in such a manner that they can withstand large interstory drifts while other such elements may not be. Therefore, the drift limits given in this table should be applied with caution and judgment.

Table 2-7 describes, for each performance level, permissible levels of damage to the various components which comprise the vertical elements of the typical building lateral force resisting systems. For purposes of classifying structural damage, structural components are classified either as primary or secondary. Primary components form part of the basic lateral load resisting system for the structure and are essential to controlling drift response and overall stability. Secondary components are not essential contributors to the structure's overall lateral stiffness or strength. Secondary components may be permitted to experience substantial strength and stiffness degradation without altering the acceptability of the structure's overall response to earthquake demands. Secondary components may be important to vertical load carrying systems within the building.

In general, secondary components may be permitted to experience more severe damage within the lower performance levels than primary components. This is because the secondary components are not important to the overall lateral response of the building. However, secondary elements which provide support for gravity loads may not be permitted to degrade so much that their ability to continue to support gravity loads is significantly compromised, unless redundant vertical load carrying capacity is available.

For the Fully Operational and Operational performance levels, the extent of damage to all components, be they primary or secondary, must be controlled to low levels. Otherwise, the visual appearance of damage in a secondary element may result in loss of use of the facility based on an apparent loss of structural strength.

Table 2-8 presents similar information for typical horizontal elements (diaphragms) of building lateral force resisting systems. For these elements, distinctions between primary and secondary elements are not made.

Tables 2-9, 2-10 and 2-11 summarize permissible levels of damage for architectural systems and components of buildings, mechanical, electrical and plumbing systems, and typical building contents respectively.

2.3 Earthquake Design Levels

Earthquake design levels are expressed in terms of a mean recurrence interval or a probability of exceedance. The mean recurrence interval (e.g. 475 years) is an expression of the average period of time, expressed in years, between the occurrence of earthquakes which produce effects of the same, or greater, severity. The probability of exceedance (e.g. 10% in 50 years) is a statistical representation of the chance that earthquake effects exceeding a given severity, will be experienced at the site within a specified number of years. Recurrence interval can be directly related to a probability of exceedance for a specified number of years.

It is recommended that the earthquake design levels defined in Table 2-1 be used for the performance based engineering of buildings. Suitable design parameters such as effective peak accelerations, elastic response spectra with appropriate damping levels, inelastic spectra, expected permanent differential vertical and lateral ground displacements, etc., should be calculated for each design event of importance to the performance objective.

Earthquake Design Level	Recurrence Interval	Probability of Exceedance
Frequent	43 years	50% in 30 years
Occasional	72 years	50% in 50 years
Rare	475 years	10% in 50 years
Very Rare	970 years[1,2]	10% in 100 years[1,2]

1. For sites located within 25 miles of an active fault or seismic zone, the hazard parameters for the "Very Rare Earthquake" need not exceed those calculated at a mean + 1 confidence level for the maximum deterministic event the fault or source zone is deemed capable of generating based on substantiated geologic data.

2. For sites located within zones of low seismicity, the hazard parameters for the "Very Rare Earthquake" should be based on a calculated Maximum Capable Earthquake for the region.

Table 2-1: Earthquake Design Levels

For each earthquake design level, the seismic hazard parameters indicated in Table 2-2 should be determined. The adequacy of the building's structural and nonstructural components to resist these hazards should be determined in accordance with the recommendations of Chapter 3. Specific mitigation measures should be taken to control the hazards to levels consistent with the permissible levels of damage stated in Tables 2-6 through 2-11 for the applicable performance levels.

Hazard	Parameter
Ground Shaking	Effective peak ground acceleration - Z Elastic response spectra Inelastic response spectra Suite of appropriate time histories
Liquefaction	Permissible foundation bearing pressure Expected vertical differential settlement Expected lateral soil displacement
Landsliding	Expected lateral and vertical soil displacement
Settlement	Expected differential settlement
Fault Rupture	Expected differential horizontal and vertical movement

Table 2-2: Seismic Hazard Parameters

2.4 Design Performance Objectives

Recommendations are provided in this section for minimum design performance objectives for buildings of three different occupancies and uses. Figure 2-2 summarizes these recommendations. Individual owners and tenants may desire, on a case by case basis, to specify design performance objectives which result in more or less damage at one or more earthquake design levels than recommended by this document for the specific building class.

It is recognized that for existing buildings, it may not be economically practical to fully achieve these recommended performance levels. Owners of existing buildings should be encouraged to upgrade their buildings, to the extent that is economically practical, to meet these recommended performance objectives. The occupancy of existing buildings, however, should not be changed to one which would result in either Safety Critical or Essential Facility service unless the building is seismically upgraded to fully meet these recommended performance objectives.

A. *Safety Critical Facilities*

Safety Critical Facilities are those which contain large quantities of hazardous materials, the release of which would result in an unacceptable hazard to wide segments of the public. Materials falling under this classification may include toxins, explosives, and radioactive materials. The presence of these materials in small quantity, as may be found in research laboratories is not intended to result in classification of a building as a "safety critical" facility. Table 2-3 summarizes the recommended earthquake performance objectives for safety critical buildings.

Safety Critical Facilities	
Earthquake Design Level[1]	Minimum Performance Level
Frequent	Fully Operational
Occasional	Fully Operational
Rare	Fully Operational
Very Rare	Operational

1. See Table 2-1 for definition.

Table 2-3: Recommended Performance Objectives for Safety Critical Facilities

B. *Essential/Hazardous Facilities*

Essential Facilities are those which are critical to post-earthquake operations. Examples include: hospitals, police stations, fire stations, communications centers, emergency control centers and shelters for emergency response vehicles. Hazardous Facilities are those which contain large quantities of hazardous materials, but where the release of those materials would be contained within the boundaries of the facility and the impact to the public would be minimal. Examples include: oil refineries, microchip manufacturing facilities and other similar types of facilities that deal with large quantities of hazardous materials. Table 2-4 summarizes the recommended earthquake performance objectives for buildings of these occupancies.

Essential/Hazerdous Facilities	
Earthquake Design Level[1]	Minimum Performance Level
Frequent	Fully Operational
Occasional	Fully Operational
Rare	Operational
Very Rare	Life-safe

1. See Table 2-1 for definition.

Table 2-4: Recommended Performance Objectives for Essential/Hazardous Facilities

C. *Basic Facilities*

Basic Facilities are buildings which are not classified as Safety Critical or Essential Hazardous Facilities. Table 2-5 summarizes the recommended performance objectives for buildings of this occupancy.

Basic Facilities	
Earthquake Design Level[1]	Minimum Performance Level
Frequent	Fully Operational
Occasional	Operational
Rare	Life-safe
Very Rare	Near Collapse

1. See Table 2-1 for definition.

Table 2-5: Recommended Performance Objectives for Basic Facilities

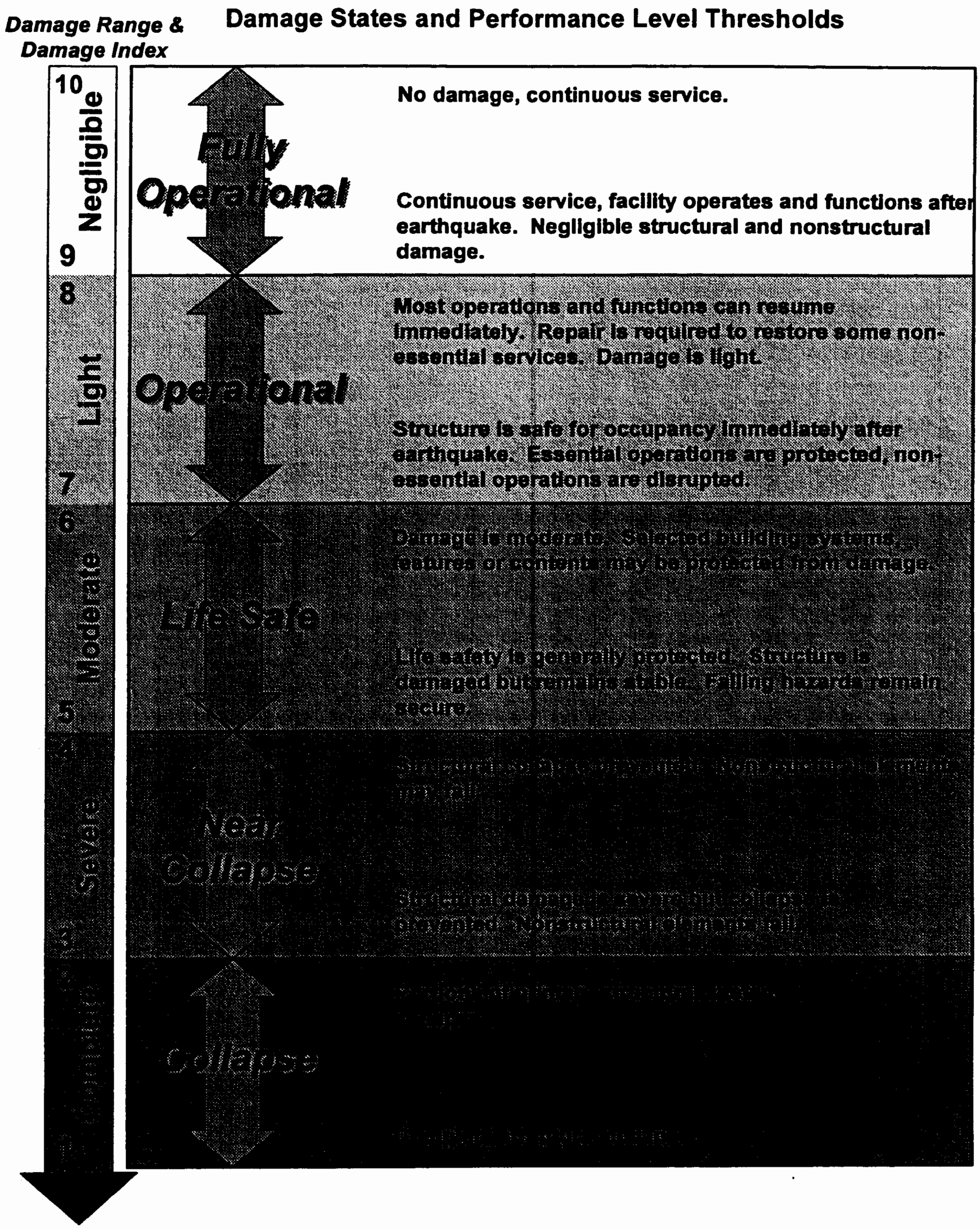

Figure 2-1: Spectrum of Damage States

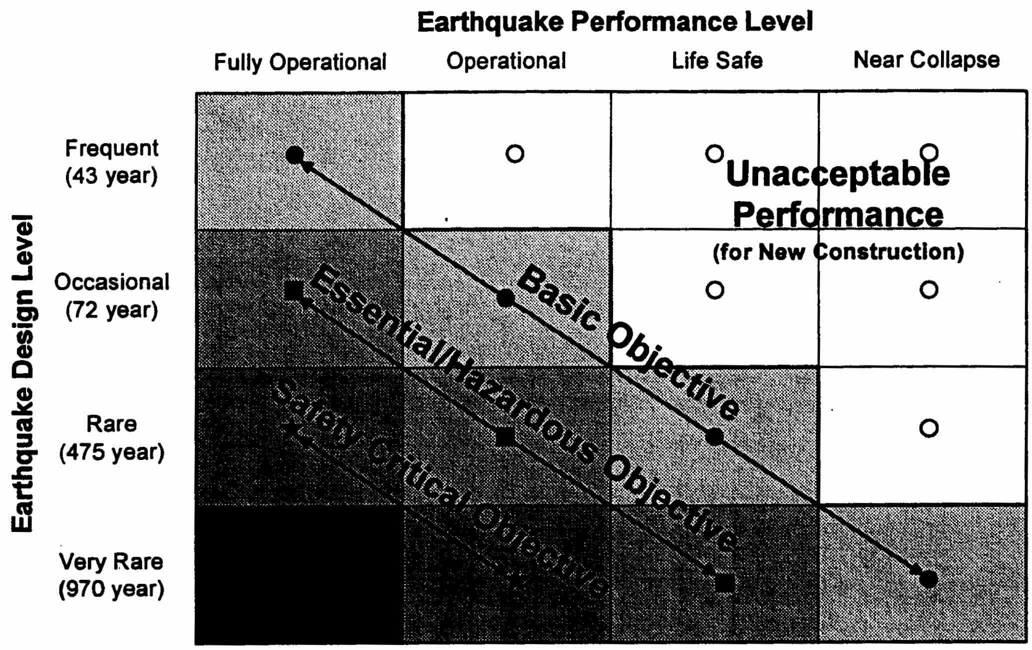

Figure 2-2: Recommended Performance Objectives for Buildings

System Description	Performance Level				
	10 Fully Operational 9	8 Operational · 7	6 Life Safe 5	4 Near Collapse 3	2 Collapse 1
Overall building damage	Negligible	Light	Moderate	Severe	Complete
Permissible transient drift	< 0.2%+/-	< 0.5%+/-	< 1.5%+/-	< 2.5%+/-	> 2.5%+/-
Permissible permanent drift	Negligible.	Negligible.	< 0.5%+/-	< 2.5%+/-	> 2.5%+/-
Vertical load carrying element damage	Negligible.	Negligible.	Light to moderate, but substantial capacity remains to carry gravity loads.	Moderate to heavy, but elements continue to support gravity loads.	Partial to total loss of gravity load support.
Lateral Load Carrying Element damage	Negligible - generally elastic response; no significant loss of strength or stiffness.	Light - nearly elastic response; original strength and stiffness substantially retained. Minor cracking/yielding of structural elements; repair implemented at convenience.	Moderate - reduced residual strength and stiffness but lateral system remains functional.	Negligible residual strength and stiffness. No story collapse mechanisms but large permanent drifts. Secondary structural elements may completely fail.	Partial or total collapse. Primary elements may require demolition.
Damage to architectural systems	Negligible damage to cladding, glazing, partitions, ceilings, finishes, etc. Isolated elements may require repair at users convenience.	Light to moderate damage to architectural systems. Essential and select protected items undamaged. Hazardous materials contained.	Moderate to severe damage to architectural systems, but large falling hazards not created. Major spills of hazardous materials contained.	Severe damage to architectural systems. Some elements may dislodge and fall.	Highly dangerous falling hazards. Destruction of components.
Egress systems	Not impaired.	No major obstructions in exit corridors. Elevators can be restarted perhaps following minor servicing.	No major obstructions in exit corridors. Elevators may be out of service for an extended period.	Egress may be obstructed.	Egress may be highly or completely obstructed.

Table 2-6a: General Damage Descriptions by Performance Levels and Systems

System Description	Performance Level				
	10 Fully Operational 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3	2 Collapse 1
Mechanical/Electrical/ Plumbing/Utility Systems	Functional.	Equipment essential to function and fire/life safety systems operate. Other systems may require repair. Temporary utility service provided as required..	Some equipment dislodged or overturned. Many systems not functional. Piping , conduit ruptured.	Severe damage and permanent disruption of systems.	Partial or total destruction of systems. Permanent disruption of systems.
Damage to contents	Some light damage to contents may occur. Hazardous materials secured and undamaged.	Light to moderate damage. Critical contents and hazardous materials secured.	Moderate to severe damage to contents. Major spills of hazardous materials contained.	Severe damage to contents. Hazardous materials may not be contained.	Partial or total loss of contents.
Repair	Not required.	At owners/tenants convenience.	Possible - building may be closed.	Probably not practical.	Not possible.
Effect on occupancy	No effect.	Continuous occupancy possible.	Short term to indefinite loss of use.	Potential permanent loss of use.	Permanent loss of use.

Table 2-6b: General Damage Descriptions by Performance Levels and Systems

Elements	Type	Performance Level			
		10 Fully Functional 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Concrete frames	Primary	Negligible.	Minor hairline cracking (0.02"); limited yielding possible at a few locations; no crushing (strains below 0.003).	Extensive damage to beams; Spalling of cover and shear cracking (< 1/8") for ductile columns. Minor spalling in non-ductile columns. Joints cracked < 1/8" width.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some non-ductile columns. Severe damage in short columns.
	Secondary	Negligible.	Same as primary.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some non-ductile columns. Severe damage in short columns.	Extensive spalling in columns (possible shortening) and beams. Severe joint damage. Some reinforcing buckled.
Steel moment frames	Primary	Negligible.	Minor local yielding at a few places. No observable fractures. Minor buckling or observable permanent distortion of members.	Hinges form; local buckling of some beam elements; severe joint distortion. Isolated connection failures. A few elements may experience fracture.	Extensive distortion of beams and column panels. Many fractures at connections.
	Secondary	Negligible.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.	Extensive distortion of beams and column panels. Many fractures at connections.	Extensive distortion of beams and column panels. Many fractures at connections.
Braced steel frames	Primary	Negligible.	Minor yielding or buckling of braces. No out-of-plane distortions.	Many braces yield or buckle but do not totally fail; Many connections may fail.	Extensive yielding and buckling of braces. Many braces and their connections may fail.
	Secondary	Negligible.	Same as primary.	Same as primary.	Same as primary.

Table 2-7a: Performance Levels and Permissible Structural Damage - Vertical Elements

Elements	Type	Performance Level			
		10 Fully Functional 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Concrete shear walls	Primary	Negligible.	Minor hairline cracking (0.02") of walls. Coupling beams experience cracking < 1/8" width.	Some boundary element distress including limited bar buckling; Some sliding at joints; damage around openings; some crushing and flexural cracking; Coupling beams - extensive shear and flexural cracks; some crushing, but concrete generally remains in place	Major flexural and shear cracks and voids; sliding at joints; extensive crushing and buckling of rebar; failure around openings; severe boundary element damage; Coupling beams shattered, virtually disintegrated
	Secondary	Negligible.	Minor hairline cracking of walls, some evidence of sliding at construction joints, Coupling beams experience cracks < 1/8" width, minor spalling.	Major flexural and shear cracks; sliding at joints; extensive crushing; failure around openings; severe boundary element damage; Coupling beams shattered, virtually disintegrated.	Panels shattered virtually disintegrated.
Unreinforced masonry infill walls	Primary	Negligible.	Minor cracking (< 1/8") of masonry infills and veneers. Minor spalling in veneers at a few corner openings.	Extensive cracking and some crushing but wall remains in place; no falling units; Extensive crushing and spalling of veneers at corners of openings.	Extensive cracking and crushing; portions of face course shed.
	Secondary	Negligible.	Same as primary.	Same as primary.	Extensive cracking and crushing; portions of face course shed.

Table 2-7b: Performance Levels and Permissible Structural Damage - Vertical Elements

Elements	Type	Performance Level			
		10 Fully Functional 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
URM bearing walls	Primary	Negligible.	Minor cracking (< 1/8") of masonry infills and veneers. Minor spalling in veneers at a few corner openings. No observable out-of-plane offsets.	Extensive cracking, noticeable in-plane offsets of masonry and minor out-of-plane offsets.	Extensive cracking. Face course and veneer may peel off. Noticeable in-plane and out-of-plane offsets.
	Secondary	Negligible.	Same as primary.	Same as primary.	Same as primary.
Reinforced masonry walls	Primary	Negligible.	Minor cracking (< 1/8"). No out-of-plane offsets.	Extensive cracking (< 1/4"), distributed throughout wall. Some isolated crushing.	Crushing; extensive cracking; damage around openings and at corners; some fallen units.
	Secondary	Negligible.	Same as primary.	Crushing; extensive cracking; damage around openings and at corners; some fallen units.	Panels shattered virtually disintegrated.
Wood stud walls	Primary	Negligible.	Distributed minor hairline cracking of gypsum and plaster veneers.	Moderate loosening of connections and minor splitting of members.	Connections loose, nails partially withdrawn, some splitting of members and panel; veneers shear off.
	Secondary	Negligible.	Same as primary.	Connections loose, nails partially withdrawn, some splitting of members and panel.	Sheathing shears off, let-in braces fracture and buckle, framing split and fractured.
Foundations	General	Negligible.	Minor settlement and negligible tilting.	Total settlements < 6 inches and differential settlements < 1/2 inch in 30 feet.	Major settlements, and tilting.

Table 2-7c: Performance Levels and Permissible Structural Damage - Vertical Elements

System	Performance Levels			
	10 Fully Functional 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Metal deck diaphragms	Negligible.	Connections between deck units and from deck to framing intact. Minor distortions.	Some localized failure of welded connections of deck to framing and between panels; minor local buckling of deck.	Large distortion with buckling of some units and tearing of many welds and seam attachments.
Wood diaphragms	Negligible.	No observable loosening, withdrawal of fasteners, or splitting of sheathing or framing.	Some splitting at connections, loosening of sheathing, observable withdrawal of fasteners, splitting of framing and sheathing.	Large permanent distortion with partial withdrawal of nails and splitting of elements.
Concrete diaphragms	Negligible.	Distributed hairline cracking and a few minor cracks (< 1/8") of larger size.	Extensive cracking (< 1/4") and local crushing and spalling.	Extensive crushing and observable offset across many cracks.

Table 2-8: Performance Levels and Permissible Structural Damage - Horizontal Elements

Element	Performance Level			
	10 Fully Operational 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Cladding	Negligible damage.	Connections yield; some cracks or bending in cladding.	Severe distortion in connections; distributed cracking, bending, crushing and spalling of cladding elements; some fracturing of cladding; falling of panels prevented.	Severe damage to connections and cladding; some falling of panels.
Glazing	Generally no damage; isolated cracking possible.	Some broken glass; falling hazards avoided.	Extensive broken glass; some falling hazard.	General shattered glass and distorted frames; widespread falling hazards.
Partitions	Negligible damage; some hairline cracks at openings.	Cracking to about 1/16" at openings; crushing and cracking at corners.	Distributed damage; some severe cracking, crushing and wracking in some areas.	Severe wracking and damage in many areas.
Ceilings	Generally negligible damage; isolated suspended panel dislocations or cracks in hard ceilings.	Minor damage; some suspended ceilings disrupted, panels dropped; minor cracking in hard ceilings.	Extensive damage; dropped suspended ceilings, distributed cracking in hard ceilings.	Most ceilings damaged; most suspended ceilings dropped, severe cracking in hard ceilings.
Light fixtures	Negligible damage; pendant fixtures sway.	Minor damage; some pendant lights broken; falling hazards prevented.	Many broken light fixtures; falling hazards generally avoided in heavier fixtures (> 20 lbs +/-).	Extensive damage; falling hazards occur.
Doors	Negligible damage.	Minor damage.	Distributed damage; some racked and jammed doors.	Distributed damage; many racked and jammed doors.
Elevators	Elevators operational with isolated exceptions.	Elevators generally operational; most can be restarted.	Some elevators out of service.	Many elevators out of service.

Table 2-9: Performance Levels and Permissible Damage - Architectural Elements

Element	Performance Level			
	10 Fully Operational 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Mechanical equipment	Negligible damage, all remain in service.	Minor damage. Some units, not essential to function out-of-service.	Many units non-operational; some slide or overturn.	Most units non-operational; many slide or overturn; some pendant units fall.
Ducts	Negligible damage.	Minor damage, but systems remain in service..	Some ducts rupture; some supports fail, but ducts do not fall.	Most systems out of commission; some ducts fall.
Piping	Negligible damage.	Minor damage. Minor leaking may occur.	Some pipes rupture at connections; many supports fail; few fire sprinkler heads fail.	Many pipes rupture; supports fail; some piping systems collapse.
Fire alarm systems	Functional.	Functional.	Not functional.	Not functional.
Emergency lighting systems	Functional.	Functional.	Functional.	Not functional.
Electrical equipment	Negligible damage.	Minor damage; panels restrained, isolated loss of function in secondary systems.	Moderate damage; panels restrained from overturning; some loss of function and service in primary systems.	Extensive damage and loss of service.

Table 2-10: Performance Levels and Permissible Damage - Mechanical/Electrical/Plumbing Systems

Element	Performance Level			
	10 Fully Operational 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Furniture	Negligible effects.	Minor damage; some sliding and overturning.	Extensive damage from sliding, overturning, leaks, falling debris etc.	Extensive damage from sliding, overturning, leaks, falling debris etc.
Office equipment	Negligible effects.	Minor damage; some sliding and overturning.	Extensive damage from sliding, overturning, leaks, falling debris etc.	Extensive damage from sliding, overturning, leaks, falling debris etc.
Computer systems	Operational.	Minor damage; some sliding and overturning. Mostly functional.	Extensive damage from sliding, overturning, leaks, falling debris etc.	Extensive damage from sliding, overturning, leaks, falling debris etc.
File cabinets	Negligible damage.	Minor damage; some sliding and overturning.	Extensive damage from sliding, overturning, leaks, falling debris etc.	Extensive damage from sliding, overturning, leaks, falling debris etc.
Bookshelves	Negligible damage.	Minor damage; some overturning and spilling.	Extensive damage from leaks, falling debris, overturning etc.	Extensive damage form leaks, falling debris, overturning etc.
Storage racks & cabinets	Negligible damage; overturning restrained.	Moderate damage; overturning restrained; some spilling.	Extensive damage from leaks, falling debris, overturning, spilling etc.	Extensive damage form leaks, falling debris, overturning, spilling etc.
Art works, collections	Minor damage; overturning restrained.	Moderate damage; overturning restrained; some falling.	Extensive damage from leaks, falling debris, overturning, spilling etc.	Extensive damage form leaks, falling debris, overturning, spilling etc.
Hazardous materials	Negligible damage; overturning and spillage restrained.	Negligible damage; overturning and spillage restrained.	Minor damage; overturning and spillage generally restrained.	Severe damage; some hazardous materials released.

Table 2-11: Performance Levels and Permissible Damage - Contents

3. UNIFORM PROCEDURES FOR PERFORMANCE BASED ENGINEERING

3.1 Introduction

This section presents interim recommendations for specific engineering procedures to be employed in the performance based engineering of buildings covering the basic structural systems and nonstructural elements. It is based on the conceptual framework for Performance Based Seismic Engineering of Buildings developed by Vision 2000 and reported in Chapter 2 of Part 2 of the Vision 2000 report.

As outlined in the Conceptual Framework for Performance Based Engineering, the following factors are critical to producing structures of predictable seismic performance:

a) selection of a suitable site,
b) selection of suitable structural materials and structural systems,
c) configuration and continuity of load path,
d) quality of detailing,
e) strength and stiffness,
f) quality and consistency of design,
g) quality of design review,
h) quality of construction, and
i) quality of inspection

The following sections of this chapter provide interim recommendations for Performance Based Engineering in terms of these factors and available design guidelines.

A number of standardized methodologies are currently available for the design of structures for earthquake resistance. Some of these are codified, such as the provisions of the ICBO, BOCA and SSBC codes, while others have been adopted as guideline standards by individual owners and government agencies such as the Department of Defense, Department of Energy, Veterans Administration, California State University System, etc. Appendix A contains summaries of some of the most commonly applied, standardized seismic design methodologies. Each summary includes:

1. a description of the performance objectives intended to be achieved by the standard methodology,

2. a description of the actual procedures employed in controlling design and construction of both structural and nonstructural components, and

3. a judgmental discussion of the apparent effectiveness of each standard methodology, based on observed earthquake performance of buildings designed to its standards, where possible.

The standard methodologies summarized in the appendix include the following:

- Uniform Building Code (ICBO-1994)
  - Standard Occupancy Facilities
  - Essential or Hazardous Facilities

- California Building Code (Title 24) (State of California-1991)
  - For Schools, Colleges, Hospitals and State Owned Essential Services Buildings
  - For other State Owned Facilities

- NEHRP Recommended Provisions (BSSC-1994)

- Seismic Design Guidelines for Essential Buildings (Department of Defense-1986)

- Uniform Code for Building Conservation, Appendix Chapter 1 (ICBO-1992)

- Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities (DOE-1994)

- Guidelines for Seismic Retrofit (CSU-1994)

- Standards of Seismic Safety for Existing Federally Owned or Leased Buildings (NIST-1994)

The recommendations contained in this chapter are based on application of selected portions of the standard methodologies noted above. Table 3-1 summarizes these recommendations for each performance objective.

While these interim recommendations tend to focus on one earthquake level and one performance level for each performance objective, it is expected that with the additional limitations given, the basic goals of the complete performance objective will be achieved. Each of these recommendations are purposely quite conservative and based on the consensus opinion of the committee. They may be superseded by appropriate advanced analysis/verification techniques. Any application that elects to supersede these recommendations should assure that all aspects of the performance based engineering framework are adequately and appropriately addressed.

3.2 Site Selection

Selection of the site for a building is one of the more important decisions which can be made with regard to controlling the building's probable behavior in future earthquakes. To the extent that the seismic hazard parameters indicated in Table 3-2 can be minimized, a building can be more economically and reliably designed and constructed to provide the desired performance. Conversely, on sites with extreme seismic hazards such as landsliding or liquefaction, extraordinary amounts of structural toughness may be required to assure acceptable earthquake performance for the building as a whole unless provisions are taken to mitigate the condition.

This section provides recommendations for selection of suitable sites for new building construction. For existing buildings, site related earthquake hazards such as liquefaction, landsliding, fault rupture, etc., must be directly considered in the design and appropriate mitigation measures taken to permit the building to perform within the design objectives.

3.2.1 Limitations Related to All Buildings

The following limitations apply to the selection of a site for any building project:

1. Buildings should not be constructed astride the known traces of active faults with surface expression unless mitigation measures are taken to resist all such adverse effects of such behavior.

2. Buildings should not be constructed within the probable debris zone or slide plane of sites subjected to landslide unless the effects of such hazard is mitigated.

3. Buildings should not be constructed within sites subjected to liquefaction and lateral spreading, unless mitigation measures are undertaken to resist all adverse affects of such behavior.

4. Buildings should not be constructed on sites adjacent to existing construction, the behavior of which may prevent the building from meeting its intended performance objectives.

In addition to consideration of the geologic hazards associated with a site, the likely earthquake behavior of adjacent construction should also be carefully considered when selecting a site for construction of a new building. The performance of a building in an earthquake can be adversely affected by neighboring construction due to pounding. In extreme cases, adjacent construction can collapse onto the new building, resulting in severe damage to a structure which might otherwise have performed well. Other potential adverse affects of adjacent construction include hazardous materials releases or initiation of fire.

3.2.2 Special Limitations Related to Higher Performance Objectives

The following limitations apply to the selection of a site for buildings designed to Essential/Hazardous or Safety Critical performance objectives:

1. Buildings designed to Essential/Hazardous or Safety Critical performance objectives should not be constructed within an Alquist-Priolo special study

zone unless adequate mitigation measures are taken to control the effects of this hazard. In California, an Alquist-Priolo special study zone is identified along many known active fault traces in the state. For any construction occurring in such a zone, trenching is required to locate the fault and assure that construction does not occur directly over the fault.

2. Buildings designed to Essential/Hazardous or Safety Critical performance objectives should not be constructed on sites subjected to liquefaction and/or lateral spreading, unless mitigation measures are taken to mitigate the effects these hazards have on the building, utilities and overall site access.

3. Buildings designed to Essential/Hazardous or Safety Critical performance objectives should not be constructed on type S3 or S4 sites as defined by the Uniform Building Code, unless mitigation measures are taken to ensure the survivability of critical utilities and access roads.

3.3 Materials and Systems Selection

A wide variety of materials and structural systems are available to the structural engineer for use in building construction. Each of these materials and systems has unique characteristics which affect their behavior in response to strong ground motion. Building codes have traditionally restricted the use of certain materials and structural systems based on building occupancy, building size, and site environmental characteristics.

Based on the uneven results of current standards, additional limitations are needed when applying current standards to performance based engineering. Structural materials and systems should be selected in accordance with the limitations of the 1994 Uniform Building Code (ICBO-1994-1), with the additional limitations contained in Table 3-2 below.

The use of seismic isolation is encouraged for structures designed to attain a Safety Critical or Essential Hazardous Facility performance objective, however, caution should be taken to accommodate the large displacements which can occur in such systems when located within a near source zone with a large magnitude earthquake potential.

3.4 Configuration and Load Path Continuity

Observation of damage to buildings in past earthquakes has clearly shown that structures with a regular distribution of mass and stiffness, and a direct and continuous path for lateral loads perform better and more reliably than do structures which do not possess these features. The Uniform Building Code, and Title 24 of the State of California Administrative Code contain specific restrictions on the design of structures with certain irregular features. In addition to the applicable restrictions contained in those codes, additional limitations are needed on irregularities to achieve higher performance objectives and these are contained in Tables 3-3 and 3-4.

3.5 Detailing

In order to attain reliable seismic performance, it is essential that elements of the structure be detailed in a manner which will permit anticipated and unanticipated inelastic deformations to occur without excessive damage. In order to achieve this, all structures, regardless of the intended performance objectives, should be detailed in accordance with the requirements contained in the Uniform Building Code for Seismic Zones 3 and 4. In addition, based on the Northridge earthquake experience, the following additional requirements should apply:

1. All masonry and concrete elements should be detailed as elements "participating" in the lateral force-resisting system.

2. Steel moment resisting frames should be detailed such that inelastic behavior of the frame is not concentrated at the beam/column joint.

3. Structural wood panels used to resist lateral forces should have a maximum height to width ratio of 2.0 for one-story walls and 1.5 for each story of two story walls.

4. Gypsum board and cement plaster should not be used as part of the lateral force resisting system for structures exceeding 1 story in height.

3.6 Strength and Stiffness

The lateral strength and stiffness of each building structural system should be evaluated and designed in accordance with the criteria contained in this section. These requirements are based on the most appropriate guidelines and standards currently available for each of the three performance objectives.

A. *Safety Critical Facilities*

Buildings designed to meet the Safety Critical performance objective should be designed in accordance with the procedure outlined in DOE Standard 1020-94, for Performance Category 3 facilities, as modified herein.

1. Ground motion used for evaluation shall be determined on a site specific basis for the "very rare" earthquake, as defined in Table 2-1 of this report.

2. Computed inter-story drifts should not exceed those indicated in Table 2-6 of this report for the Operational performance level.

3. For sites located within 5 miles of the known surface trace of a fault capable of producing earthquakes in excess of M6.5, or located above the horizontal projection of a known thrust plane, the effects of vertical ground motion shall be considered as follows:

   a) All elements of the building (participating in lateral force resisting system or not) shall be evaluated for the effects of response to vertical ground motion. Vertical ground motion response shall be assumed to produce either increases or decreases in gravity load stresses.

   b) Elements subject to forces resulting from both vertical and lateral earthquake response shall be evaluated for the following load combinations:

      i. $D_T = D_{NS} + D_{SV} + D_{SI}$
      ii. $D_T = D_{NS} - D_{SV} + D_{SI}$

      where: D_T is as defined in DOE Standard 1020-94

      D_{NS} is as defined in DOE Standard 1020-94

      D_{SV} is the calculated earthquake force due to vertical response, without modification factors

      D_{SI} is as defined in DOE Standard 1020-94

   c) Vertical ground motion shall be calculated for the "very rare" earthquake, using either:

      i. representative spectra obtained from instrumented sites at similar distances to past earthquakes with similar causative mechanisms, or

      ii. direct modeling of the fault rupture mechanism and transmission of ground motion through the geologic media.

B. *Essential/Hazardous Facilities*

Buildings designed to meet the Essential/Hazardous performance objective should be designed in accordance with the procedure outlined in Title 24 of the California Administrative Code, as modified herein:

1. The occupancy importance factor I shall be taken as 1.5.

2. Ground motion used for evaluation shall be determined on a site specific basis for the "rare" earthquake, as defined in Table 2-1 of this report.

3. For sites located within 5 miles of the known surface trace of a fault capable of producing earthquakes in excess of M6.5, or located above the horizontal projection of a known thrust plane, the effects of vertical ground motion shall be considered as follows:

   a) All elements of the building (participating in lateral force resisting system or not) shall be evaluated for the effects of response to vertical ground motion. Vertical ground motion response shall be assumed to produce either increases or decreases in gravity load stresses.

   b) Vertical ground motion shall be calculated for the "rare" earthquake, using either:

      i. representative spectra obtained from instrumented sites at similar distances to past earthquakes with similar causative mechanisms, or

      ii. direct modeling of the fault rupture mechanism and transmission of ground motion through the geologic media.

C. *Basic Facilities*

Buildings designed to meet the Basic performance objective should be designed in accordance with the Uniform Building Code, using an occupancy importance factor I = 1.0 and using a realistic assessment of the ground motion and other hazards resulting from the "rare" earthquake defined in Table 2-1 of this report.

3.7 Nonstructural Components and Contents

In order for a building to attain the Fully Operational or Operational performance levels, the nonstructural components and contents of the building must perform as reliably as the structure itself. In many past earthquakes, however, building structures have performed adequately while the contents and nonstructural components did not. This points to a general inadequacy in current practice and approaches to the design and installation of nonstructural components and contents. As indicated in ATC-29, this often occurs because responsibility for the design and installation of nonstructural systems is delegated to persons who are not trained in seismic engineering.

This section presents interim recommendations for the assumption of engineering responsibility for the design and construction quality assurance of the seismic performance of nonstructural features and contents, as well as the preferred procedures to be used in the execution of this engineering.

3.7.1 Seismic Performance Categories

There are three important concerns to be considered in the design of nonstructural components for seismic performance. One concern is that components may have inadequate attachment to the structure and could slide or fall, damaging themselves, adjacent components or resulting in potential injury. A second concern is that the components can fail structurally, resulting in similar hazards. A third concern is that the component may be undamaged structurally, but that motion transmitted to the component could cause the dysfunction of mechanical or electrical devices within the unit, resulting in loss of service. Table 3-5 defines four performance categories (A through D) for nonstructural components and indicates the particular design concerns which should be addressed for each.

3.7.2 Responsibility

The primary purpose of structural components of a building is to provide structural support. The Structural Engineer of Record therefore, has primary responsibility for the design of these structural components. However, the Structural Engineer of Record will not, in general, have primary responsibility for the design of nonstructural components or contents. Instead a design professional of record other than the structural engineer, for example a mechanical engineer, electrical engineer, or architect will have such responsibility, and in many cases, the component manufacturer will have ultimate responsibility for the design.

The apportionment of responsibility for seismic design of nonstructural components between the various project participants should be specified as follows:

A. *Structural Engineer of Record*

The Structural Engineer of Record should have responsibility for the following with regard to the seismic performance design of nonstructural components and contents:

1. Take the lead role in working with the architect, mechanical engineer, and electrical engineer in defining a seismic performance category for each nonstructural component.

2. Define for the other design professionals of record, those public utilities which are unlikely to be available immediately following each earthquake design level, so that provision of appropriate standby utilities may be made in the design.

3. Define for the other design professionals of record, the seismic environment (acceleration, velocity, differential displacement) that each nonstructural component in performance categories C and D will be subjected to.

4. Design and provide construction documents for support systems and anchorage for all components in performance categories B, C, and D:

   Exception: Support and anchorage systems for the following components need not be designed by the Structural Engineer of Record:

   - Piping and tubing
   - Conduit, cable trays and raceways
   - Ducting

   These should be designed by other design professionals of record as defined in the next section.

5. Review manufacturers submittals for all components for which anchorage or support has been designed by others, to ascertain that appropriate mounting and anchorage features are provided and that assumed weights and center of gravities are correct.

6. Review all specifications for nonstructural components which must be specifically qualified by the manufacturers for seismic resistance (see Section 3.7.3) to assure that appropriate seismic design data has been provided.

7. Review all certifications of seismic design compliance including test data and calculations submitted by manufacturers.

8. Perform site observations for the adequacy of installation of all nonstructural components in performance category D.

B. *Other Design Professionals of Record*

The architect, mechanical engineer, electrical engineer and other designers of record should have responsibility for the following with regard to design for seismic performance:

1. Coordinate with the Structural Engineer of Record to determine the seismic performance category for all building components for which they are responsible.

2. For systems which must remain operable in "moderate", "rare", or "very-rare" earthquakes, as defined in Table 2-1, consult with the Structural Engineer of Record to determine the probability that public utilities will be available immediately following these design earthquakes and provide for back-up utilities as required.

3. Obtain from the Structural Engineer of Record, environmental data on the seismic hazards (acceleration, velocity, differential displacement) for which nonstructural components in performance categories C and D must be designed.

4. Provide data to the Structural Engineer of Record on the weight, mounting provisions and center of gravity location for all components in performance categories B, C and D.

5. Include seismic environmental data obtained from the Structural Engineer of Record in all specifications for nonstructural components which must be specifically qualified by the suppliers for seismic resistance (see Section 3.7.3).

6. Provide manufacturers certification data on the seismic performance of equipment to the Structural Engineer of Record for review.

7. Include appropriate references to applicable seismic design standards in specifications for installation of piping, ducting, conduit, raceways and similar materials in which the installation contractor is primarily responsible for the design and installation of bracing systems.

3.7.3 Seismic Qualification of Components

All nonstructural components in performance categories B, C and D should be demonstrated capable of meeting the applicable performance concerns indicated in Table 3-5, unless designated as inherently rugged. Qualification of the adequacy of nonstructural components to meet the performance concerns indicated in Table 3-5 should be in accordance with following:

1. Performance category B - calculations of adequacy of attachment to structure, in accordance with procedures of Section 3.7.4.

2. Performance category C - calculations of adequacy of the component's structural system and attachment to building structure, in accordance with procedures of Section 3.7.4.

3. Performance category D - calculations of adequacy of the component's structural system and attachment to building structure, for mechanically and electrically inactive components; or specific qualification testing for mechanically or electrically active components, in accordance with the procedures of Section 3.7.4.

Certain classes of nonstructural components are classified as structurally rugged or functionally rugged, based on the observed performance of these components in repeated past earthquakes. Structurally rugged components, if fabricated in accordance with industry standard practice for the component, have been demonstrated to be able to withstand very strong levels of seismic excitation without structural failure. If properly installed, such components can be expected to withstand very rare earthquakes without failure that would result in a danger to adjacent occupants or components. However, structurally rugged components may not necessarily function following severe shaking. Table 3-6 lists nonstructural components which may be classified as structurally rugged if fabricated in accordance with industry standard practice. Seismic qualification of structurally rugged components shall include the following:

1. Performance categories B, or C - calculations of adequacy of attachment to structure, in accordance with procedures of Section 3.7.4.

2. Performance category D - specific qualification testing for mechanically or electrically active components, in accordance with the procedures of Section 3.7.4.

Functionally rugged components include those classes of components that have been demonstrated in past earthquakes to be capable of surviving very strong seismic excitation, without failure, if fabricated and installed in accordance with industry standard practice. Table 3-7 lists nonstructural components which may be classified as functionally rugged. Seismic qualification of functionally rugged components shall include the following:

1. Performance categories B, C, or D - calculations of adequacy of attachment to structure, in accordance with procedures of Section 3.7.4.

3.7.4 Design Procedures

The specific standardized design methodologies to be used in the qualification and design of nonstructural components and their attachments to the building structure should be used or specified as follows:

A. *Basic Facilities*

For buildings with fundamental periods of vibration, in each principal direction of the building, less than 0.5 seconds, it is recommended that nonstructural components be designed and installed in accordance with the applicable provisions of the Uniform Building Code. Any equipment which is required by the owner to function following moderate earthquakes and which is not functionally rugged, in accordance with Table 3-7, should be demonstrated by direct prototype testing in accordance with the procedures of IEEE 344, as being capable of post-earthquake function. The earthquake ground motion used for such qualification should be that calculated for the moderate earthquake defined in Table 2-1 of this report.

For buildings with a fundamental period in any direction of the building of 0.5 seconds or more, nonstructural components shall be designed in accordance with Chapter 3 of the NEHRP Provisions (BSSC-1994). When using that document, the following substitutions shall apply for design parameters specified in the document:

1. For performance category B or C equipment, $I_p = 1.0$. For performance category D equipment, $I_p = 1.5$.

2. C_a and C_v shall be taken equal to the value of the effective peak ground acceleration used for the balance of the structural design if a site specific hazard analysis was performed or may be taken equal to the Uniform Building Code zone coefficient "Z", otherwise.

3. Deflections of the structure δ_{xA}, and δ_{yA} shall be taken as the corresponding deflections of the building under the forces used for design in the Uniform Building Code, multiplied up by 1.4.

B. *Essential/Hazardous Facilities*

Nonstructural components in Essential/Hazardous Facilities buildings should be designed for the more restrictive provisions contained in a) Title 24 of the California Administrative Code, or b) Chapter 3 of the NEHRP Provisions. When using that document, the following substitutions shall apply for design parameters specified:

1. For performance category B or C equipment, $I_p = 1.0$. For performance category D equipment, $I_p = 1.5$.

2. C_a and C_v shall be taken equal to the value of the effective peak ground acceleration for a "rare" earthquake as defined in Table 2-1 of this report, based on a site specific hazard study.

3. Deflections of the structure δ_{xA}, and δ_{yA} shall be taken as the corresponding deflections of the building under the forces used for structural design, multiplied up by 1.4.

C. *Safety Critical Facilities*

Nonstructural components in Safety Critical Facilities should be designed in accordance with the provisions contained in Chapter 3 of the NEHRP Provisions. When using that document, the following substitutions shall apply for design parameters specified:

1. For performance category B or C equipment, $I_p = 1.0$. For performance category D equipment, $I_p = 1.5$.

2. C_a and C_v shall be taken equal to the value of the effective peak ground acceleration calculated for a very rare earthquake as defined in Table 2-1 of this report, based on a site specific hazard study.

3. Deflections of the structure δ_{xA}, and δ_{yA} shall be taken as the corresponding deflections of the building under the forces used for structural design. The value of C_d shall be taken as 1.0.

4. The coefficient R_p shall not be taken larger than 1.5, regardless of the type of component.

3.8 Quality Assurance for Design

Reliable seismic performance of buildings starts with properly executed designs. It is an unfortunate fact that many buildings are designed without an adequate design budget and by practitioners who do not have adequate training or understanding of the fundamentals behind the procedures they are employing. Also many building departments do not perform a plan check of adequate detail. As a result, buildings are sometimes constructed using poorly conceived systems or with inadequate detailing. There have been a number of cases reported in the literature where poor performance in buildings constructed under the provisions of recent codes has been attributed to a poorly executed design. Similarly, the excellent performance attained by most school and hospital structures in recent California earthquakes has been partially attributed to the careful third party design review provided by State of California plan check Structural Engineers, under state law.

This section presents interim recommendations to assure that buildings will have predictable and defined seismic performance by having a quality control check as an integral part of the design process. This check can be a third party design review or a building department plan review. Recommendations are given as to when these two reviews should be used as well as procedures that should be followed to insure complete and relevant reviews.

3.8.1 Requirements for Third Party Design Review

Independent, third party design review is an essential part of the design process for the following projects and should be implemented following the procedures in Section 3.8.2:

1. Buildings designed to the Safety Critical performance objective.

2. Buildings designed to the Essential/Hazardous performance objective.

3. Any project incorporating advanced or unusual technologies including seismic isolation or passive damping.

4. Seismic upgrade projects for existing buildings containing critical gravity or lateral load carrying elements the strength and deformation capacity of which can not be determined by reference to published data or standardized design methodologies.

Where the building permit process includes a plan check performed by the California Division of the State Architect, or the California Office of Statewide Health Planning and Development under the requirements of Title 24 of the California Administrative Code, this may be considered to fulfill the recommendations for independent third party review except for projects involving seismic isolation or energy dissipation systems.

3.8.2 Procedures for Third Party Design Review

While third party design review can be quite beneficial to achieving the design performance objectives of a project, if not performed properly or if the third party reviewer is not adequately qualified, it may not significantly add to the project's quality. The performance of third party design reviews for all applications noted in Section 3.8.1 should be based on the following criteria, schedule, and scope of work.

A. *Qualifications of Reviewer*

The third party reviewer should have the necessary registration to act as the engineer of record for the project. The reviewer should have recognized technical expertise and capability in all aspects of the design process relevant to the particular project including ground motion evaluation, geotechnical engineering, structural analysis, and structural detailing. The reviewer should have specific past experience in other projects which employed the same technologies and structural systems.

In some cases, in order to obtain expertise in all areas pertinent to the design, it may be necessary to assemble a team of reviewers comprised of members with individual expertise in unique areas important to the project such as geotechnical engineering, base isolation, or energy dissipation. In such cases, one person should be appointed to chair the review team. The responsibilities of the chair should include ensuring that all members of the team are provided with necessary information, to serve as the liaison between the engineer of record and the team members, and to state the official opinions of the review team. The review team chair should have the necessary registration to act as the engineer of record for the project and should have previous design experience for similar structures.

B. *Timing of Review*

To be effective, third party review should commence as early as practical in the project so that review comments may be incorporated into the project without undue cost or delay to the project. It is recommended that reviews be performed at the following project milestones:

- a completion of schematic design documents,
- a completion of design development documents,
- prior to procurement of major components such as isolation bearings, energy dissipation units, or structural steel, and

- at completion of construction documents.

C. *Scope of Review*

The purpose of the review is to provide an independent informed opinion to the engineer of record, building owner and building official as to whether basic design assumptions and design decisions, calculation procedures and final detailing of the building structure are appropriate to achieving the design performance objectives. The third party review is not intended to be a plan check, value engineering, or a constructability review, although concerns with regard to constructability should be noted in that the ability to construct a design as intended is critical to the capability of the design to achieve its objectives. It is not intended that the third party reviewer's judgments be binding on the engineer of record or that the reviewer assume responsibility for the design. All final design decisions are to be made by the engineer of record who retains basic responsibility for the performance of the design. It is recommended that the reviews at the various project milestones contain the following:

1. <u>Schematic Design Phase</u> - Review performance objectives, occupancy requirements, and the critical components (structural and nonstructural) necessary to building function. Review site hazards study. Review proposed building configuration and structural systems. Provide summary report indicating preliminary findings and recommendations for consideration in the design development phase.

2. <u>Design Development Phase</u> - Review preliminary drawings and calculations to identify the basic design approach and to identify any critical flaws which may be inherent in the design. Provide summary report indicating any recommendations for consideration in the construction documents phase.

3. <u>Construction Documents Phase</u> - Review final drawings, calculations and specifications. Particular attention should be given to evaluating the consistency of detailing with design assumptions. Provide a final report indicating the findings of the review. This report should be presented to the engineer of record, the owner and the building official.

3.8.3 Building Department Plan Review

Prior to issuing a permit for construction, the Building Official should ensure that suitable plans, specifications, calculations and design reports have been submitted under the seal of the design professional(s) of record for the project. The Building Official should perform (or have performed) an independent review of the submitted documents to determine that:

1. Design basis is in accordance with all applicable codes and ordinances governing building safety.

2. Construction documents clearly identify the work to be constructed.

3. Construction documents clearly identify all contractor submittals, special inspections and testing which are to be performed during construction.

4. Construction documents clearly indicate all observations to be made by the engineer of record during construction.

5. Calculations are performed in accordance with code requirements and standard practice and the calculations are consistent with plans and specifications.

3.9 Construction Quality Assurance

Regardless of the adequacy of a building design to meet performance objectives, reliable earthquake performance can not be attained unless the building is constructed in the manner and with the quality assumed in the design. The Uniform Building Code contains a number of provisions intended to ensure appropriate quality construction. These include requirements for inspections by the Building Official as well as independent inspections and tests performed by special inspectors. In addition, provisions within the code provide for direct observation of the construction in progress by the engineer of record. Despite these safeguards, observation of damaged structures from the Northridge Earthquake indicates that improper construction is a frequent contributing cause of the damage. These problems span the entire range of construction materials and types, from low-rise wood frame structures to high-rise steel frame buildings. The recommendations of this section are intended to improve the quality of construction and therefore, the reliability of building earthquake performance.

Under the provisions of the building code, the Building Official is responsible to perform certain inspections. It is recommended that the Building Official take necessary steps to ensure that personnel performing these inspections are performing their assigned tasks in a diligent manner and that they have adequate training to ensure that they are effectively identifying construction of inadequate quality.

The building code requires that certain specialized tests and inspections be performed by a special inspector. Special inspectors should be employed directly by the building owner, rather than the contractor. The building owner should consult with the Structural Engineer of Record in selecting a special inspector with appropriate qualifications to perform the required work. The Structural Engineer of Record should be furnished copies of appropriate certifications and qualification certificates indicating the individual inspectors' qualifications to perform the inspections and tests required, during the selection of inspection personnel. Special inspection reports indicating non-conformance of installed work should be submitted to the Structural Engineer of Record within 24 hours of discovery of the non-conforming condition so that timely corrective action may be taken.

The Structural Engineer of Record is often the person most qualified to determine if construction is occurring in the manner anticipated in the design. At one time it was common for the Structural Engineer of Record to make regular observations of construction work in progress. However, in recent years, engineers have become concerned that such site observation visits could result in the engineer incurring liability for construction accidents resulting from unsafe practices or inadequate contractor supervision of the construction process. Consequently, many engineers do not make job site visits of construction work in progress. Engineers should re-establish the practice of observing construction work in progress, to improve the overall quality of construction. In order to encourage this practice, it is recommended that the State of California enact legislation which would minimize potential liability to the engineer for construction site accidents.

The following sections recommend minimum actions for construction quality assurance by the Structural Engineer of Record, for projects with different performance objectives. The Structural Engineer of Record may delegate suggested responsibilities to qualified persons acting under his/her direct supervision.

A. *Basic Facilities*

In addition to the minimum requirements of the building code for inspections and special inspections, it is recommended that the Structural Engineer of Record:

1. Ensure that the construction documents clearly specify all required submittals of shop drawings, material data, certifications and other submittals important to assuring that the structural work is properly performed.

2. Ensure that the construction documents clearly designate all special inspections and tests which the engineer desires as part of the overall quality assurance program. The specifications should indicate the type and number of tests/inspections to be performed, the methods to be followed, and the acceptance criteria. The Structural Engineer of Record should consult with the owner in the selection of inspection personnel.

3. Ensure that all structural shop drawings, material data, certifications, inspection and test reports and other submittals specified by the construction documents are submitted.

4. Review all submitted structural documents for conformance with the project requirements and take timely corrective action when non-conforming work is reported.

B. *Essential/Hazardous Facilities*

In addition to the recommendations above for Basic Facilities, it is recommended that the Structural Engineer of Record:

1. Specify full time inspection, during construction of the structural system, by a special inspector(s) competent to inspect the work being performed.

2. Specify that the special inspector prepare a daily report noting the progress of construction, noting any non-conforming conditions, and notifying the Structural Engineer of Record.

3. Ensure that the requirement for structural observations of critical stages of the work are noted on the drawings so that the contractor may provide adequate notification to the Structural Engineer of Record that observations can be made.

4. Make periodic visits to the construction site to observe the progress of the work and the general extent to which construction conforms with the design intent. Following each visit, the Structural Engineer of Record should prepare a written report to the Owner noting any non-conforming conditions observed.

C. *Safety Critical Facilities*

In addition to the recommendations above, for Basic Facilities, it is recommended that the Structural Engineer of Record:

1. Provide full time observation of the construction work in progress.

2. Prepare daily reports to the owner indicating the construction progress and noting any non-conforming conditions observed.

Design Parameter	Performance Objective			
	Basic		Essential/Hazardous	Safety Critical
	T < 0.5 seconds	T $\geq$0.5 seconds		
Design code	1994 Uniform Building Code	1994 Uniform Building Code	1991 California Building Code - Title 24	1994 DOE Standard 1020
Importance factor	I =1.0	I = 1.0	I = 1.5	Performance Category = 3
Design earthquake return period	475 year	475 year	475 year	1000 year
Structural system limitations (see also Table 6.2)	Minor: all structural systems permitted except bearing wall and masonry moment-resisting wall frame bldgs. > 3 stories and concrete SMRF w/ spans < or = to 25'.	Comprehensive: only EBF, plywood shear wall frames, conc. & masonry shear walls, conc. & steel SMRF, special steel concentric frames, dual systems w/ SMRFs and concrete SMRF w/ spans > 25' allowed.	Comprehensive: only EBF, plywood shear wall frames, conc. & masonry shear walls, conc. & steel SMRF, special steel concentric frames, and dual systems w/ SMRFs and concrete SMRF w/ spans > 25' allowed.	Same as Essential Facility except also eliminate plywood shear wall frames, masonry shear walls, and conc. & steel SMRF.
Configuration limitations (see also Table 6.3 & 6.4)	None.	No irregular buildings as defined in UBC Table 16-L or 16-M allowed.	No irregular buildings as defined in UBC Table 16-L or 16-M allowed.	No irregular buildings as defined in Table 6.3 allowed.
Detailing provisions	1994 UBC	1994 UBC	1994 UBC	1994 UBC
Nonstructural provisions	1994 UBC	1994 NEHRP	1994 NEHRP	1994 NEHRP
Peer review & Inspection requirements	Design review by building department plus special inspection and engr. inspection as req'd. by UBC.	Design review by building department plus special inspection and engr. inspection as req'd. by UBC.	Independent peer review or design review (OSA or OSHPD) plus full time special inspection and periodic inspection by SE.	Independent peer review plus full time inspection by SE of record or agent.
Verification requirements	None.	3D dynamic analysis w/ displacement spectra - story drifts vary by no more than 20% from adjacent story.	None.	Analysis required by DOE Standard 1020.

Table 3-1: Summary of Design Parameters for each Performance Objective

Basic System	Material System	Performance Objective			
		Basic		Essential/ Hazardous	Safety Critical
		T < 0.5 seconds	T ≥ 0.5 seconds		
Bearing Wall System	1. Light framed walls with shear panels				
	a. wood structural panels 3 stories or less				not permitted
	b. all other light framed walls		not permitted	not permitted	not permitted
	2. Shear Walls				
	a. Concrete	limit to 3 stories	limit to 3 stories	not permitted	not permitted
	b. Masonry	limit to 3 stories	limit to 3 stories	not permitted	not permitted
	3. Light steel framed bearing walls with tension only bracing			not permitted	not permitted
	4. Braced frames where bracing carries gravity load				
	a. Steel		special CBF, EBF	special CBF, EBF	not permitted
	b. Concrete	not permitted	not permitted	not permitted	not permitted
	c. Heavy timber			not permitted	not permitted
Building Frame System	1. Steel Eccentrically Braced Frame (EBF)				
	2. Light framed walls with shear panels				
	a. wood structural panels 3 stories or less				not permitted
	b. all other light framed walls		not permitted	not permitted	not permitted
	3. Shear Walls				
	a. Concrete				
	b. Masonry		not permitted	not permitted	not permitted
	4. Ordinary Braced Frames				
	a. Steel		not permitted	not permitted	not permitted
	b. Concrete	not permitted	not permitted	not permitted	not permitted
	c. Heavy Timber			not permitted	not permitted
	5. Special Conc. Braced Frames (CBF)				
	a. Steel				

Table 3-2a: Recommended Limitations on Structural Materials and Systems, by Performance Objective

Basic System	Material System	Performance Objective			
		Basic		Essential/ Hazardous	Safety Critical
		T < 0.5 seconds	T ≥ 0.5 seconds		
Moment resisting frame system	1. Special Moment Resisting Frames (SMRF)				
	a. Steel				not permitted
	b. Concrete	spans ≤ 25 feet	spans ≤ 25 feet	spans ≤ 25 feet	not permitted
	2. Masonry Moment Resisting Wall Frame	≤ 3 stories	≤ 3 stories	not permitted	not permitted
	3. Concrete Intermediate Moment Frame	not permitted	not permitted	not permitted	not permitted
	4. Ordinary Moment Resisting Frame				
	a. Steel		not permitted	not permitted	not permitted
	b. Concrete	not permitted	not permitted	not permitted	not permitted
Dual Systems	1. Shear Walls				
	a. Concrete with SMRF				
	b. Concrete with steel OMRF		not permitted	not permitted	not permitted
	c. Concrete with concrete IMRF	not permitted	not permitted	not permitted	not permitted
	d. Masonry with SMRF				not permitted
	e. Masonry with steel OMRF		not permitted	not permitted	not permitted
	f. Masonry with concrete IMRF		not permitted	not permitted	not permitted
	2. Steel EBF				
	a. With Steel SMRF				
	b. With Steel OMRF		not permitted	not permitted	not permitted
	3. Ordinary Braced Frames				
	a. Steel with steel SMRF		not permitted	not permitted	not permitted
	b. Steel with steel OMRF		not permitted	not permitted	not permitted
	c. Concrete with concrete SMRF	not permitted	not permitted	not permitted	not permitted
	d. Concrete with concrete IMRF	not permitted	not permitted	not permitted	not permitted
	4. Special concentrically braced frames				
	a. Steel with steel SMRF				
	b. Steel with steel OMRF		not permitted	not permitted	not permitted

Table 3-2b: Recommended Limitations on Structural Materials and Systems, by Performance Objective

Irregularity	Performance Objective			
	Basic		Essential/ Hazardous	Safety Critical
	T < 0.5 Seconds	T ≥ 0.5 Seconds		
1. **Stiffness Irregularity - Soft Story**				
a. lateral stiffness of story below less than 90% of that of story above, or less than 90% of average of stiffness of 3 stories above. lateral stiffness of story below exceeds 150% of stories above.				not permitted
b. lateral stiffness of story below less than 70% of that of story above, or less than 80% of average of stiffness of 3 stories above.		not permitted	not permitted	not permitted
2. **Weight (mass) Irregularity**				
a. the effective mass of any story is more than 120% of the effective mass of an adjacent story. A roof which is lighter than the floor below need not be considered.				not permitted
b. the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof which is lighter than the floor below need not be considered.		not permitted	not permitted	not permitted
3. **Vertical geometric irregularity** - where the horizontal dimension of the lateral force resisting system in any story is less than 130% of that in adjacent story, excluding one story penthouses.		not permitted	not permitted	not permitted
4. **In-plane discontinuity in vertical lateral-force resisting element.** An in-plane offset of the lateral force elements greater than the length of those elements.		not permitted	not permitted	not permitted
5. **Discontinuity in Capacity - weak story**				
a. The story strength in a story is less than 100% of that in the story above or more than 200% of that in the story above.				not permitted
b. The story strength in a story is less than 80% of that in the story above.		not permitted	not permitted	not permitted

Table 3-3: Limitations on Vertical Irregularity

Irregularity	Performance Objective			
	Basic		Essential/ Hazardous	Safety Critical
	T < 0.5 Seconds	T $\geq$ 0.5 Seconds		
1. **Torsional Irregularity** - when diaphragms are not flexible, and the maximum story drift calculated including accidental torsion at one end of the structure transverse to an axis is more than 1.2 times the average of the story drift of the two ends of the structure.		not permitted	not permitted	not permitted
2. **Reentrant Corners.** - plan configurations of a structure and its lateral force resisting system contain reentrant corners where both projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.		not permitted	not permitted	not permitted
3. **Diaphragm Discontinuity**				
a. Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out or open areas greater than 15% of the enclosed area.			not permitted	not permitted
b. Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out or open areas greater than 50% of the enclosed area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.		not permitted	not permitted	not permitted
4. **Out-of-plane offsets.** - Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.		not permitted	not permitted	not permitted
5. **Nonparallel systems.** The vertical lateral-load resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force resisting system.		not permitted	not permitted	not permitted

Table 3-4: Limitations on Horizontal Irregularity

Performance Category	Description	Design Concerns
A	Components weighing less than 400 pounds, mounted on floor with center of gravity less than 4 feet above floor level, and not required to function in "moderate", "rare" or "very rare" earthquakes[1]	• None for buildings with T < 0.5 sec. • For buildings with T > 0.5 sec., provide attachment to structure to prevent sliding or overturning.
B	Structurally rugged components weighing 400 pounds or more, or mounted with center of gravity more than 4 feet above floor level, but not required to function in "moderate", "rare" or "very rare" earthquakes[1]	• Attachment to structure adequate to prevent sliding or overturning.
C	Components weighing 400 pounds or more, or mounted with center of gravity more than 4 feet above floor level, but not required to function in "moderate", "rare" or "very rare" earthquakes[1]	• Attachment to structure adequate to prevent sliding or overturning. • Structurally adequate to prevent failure resulting in impact on neighboring components or occupants.
D	Components which must function following "moderate", "rare" or "very rare" earthquakes	• Attachment to structure adequate to prevent sliding or overturning. • Structurally adequate to prevent failure resulting in impact on neighboring components or occupants • Mechanically adequate to function.

1. See Table 2-1 for definition

Table 3-5: Seismic Performance Concerns for Nonstructural Components

Structurally Rugged Equipment
• Air handlers and fans (without internal vibration isolators)
• Low and medium-voltage switchgear ($\leq$ 13.8kV)
• Instrumentation cabinets
• Distribution panels
• Battery chargers
• Motor control centers
• Instrument racks
• Inverters
• Chillers

Table 3-6: Structurally Rugged Equipment

Functionally Rugged Equipment
• Valves
• Engines
• Motors
• Generators
• Turbines
• Horizontal and vertical pumps
• Hydraulic and pneumatic operators
• Motor operators
• Compressors
• Small factory-manufactured boilers
• Transformers with anchored internal coils
• Batteries

Table 3-7: Functionally Rugged Equipment

4. REFERENCES

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APPENDIX A

Summary of Standard Methodologies

Standard:	**Uniform Building Code, 1994 edition**
Performance Objectives	*I=1.0 Standard Occupancy Facility:* Resist Minor Earthquakes without damage. Resist Moderate Earthquakes with architectural damage and potentially light structural damage. Resist Major Earthquakes without collapse or endangerment of life safety but with potentially severe structural damage. It is expected that most structures would be repairable. (Similar to Standard Objective.) *I=1.25 Essential or Hazardous Facility:* Resist Moderate Earthquakes without damage. Resist Major Earthquakes with architectural damage and potentially light structural damage. (Similar to Essential Facility Objective.)
Applicability:	Design of new structures and evaluation of existing structures with conforming detailing present in all elements participating in the lateral force resisting system.

Design Events:	A single Design Basis Earthquake is used as the basis of design. This is a severe event, estimated as having a 475 year return period.
Ground Motion:	An elastic acceleration response spectrum tied to an effective peak ground acceleration. Approximate formulas are provided for obtaining these spectra.

Design Procedures - Structural Elements

Design Approaches:	Two approaches are permitted: A *general approach* intended to be applicable to all structures and which includes a combination of prescriptive detailing and analytical provisions. A *simple approach,* applicable only to low-rise wood frame construction with extensive prescriptive detailing requirements and no analytical provisions.

General Approach

Design controls:

Provisions specify minimum criteria for:

a. Element connectivity and a complete lateral load path
b. Limits on configuration and distribution of stiffness
c. Permissible materials of construction
d. Element detailing
e. Elastic strength of lateral force resisting system
f. Elastic lateral stiffness of structure

Analytical Procedures:	Three analytical procedures are defined: Equivalent lateral force (ELF), Elastic Response Spectrum (RSA) and Elastic Time History (TH). In addition, alternative rational analyses are permitted.
ELF:	Limited to structures within specified limits of regularity and height, but applicable to most structures. Forces and displacements in the lateral force resisting system are based on static elastic analysis for an applied lateral forcing function. The forcing function is representative of the distribution of lateral inertial forces determined for elastic structures with uniform weight and stiffness distributions when analyzed for response to typical Western US ground motions. Total applied lateral forces are scaled down to a fraction of the theoretical elastic response to account for the beneficial effects of period lengthening, damping, soil structure interaction, overstrength, redundancy and other factors, based on the type of lateral force resisting system employed.
RSA:	Applicable to any structure. Forces and displacements in the lateral force resisting system are directly determined from an elastic response spectrum analysis and may be reduced to account for the beneficial effects of period lengthening, damping, soil structure interaction, overstrength, redundancy and other factors, based on the type of lateral force resisting system employed. Results can not be less than a fixed percentage of those determined from the ELF approach.
TH:	Applicable to any structure. Forces and displacements in the lateral force resisting system are directly determined from an elastic time history analysis, reduced to account for the beneficial effects of period lengthening, damping, soil structure interaction, overstrength, redundancy and other factors, based on the type of lateral force resisting system employed. Results can not be less than a fixed percentage of those determined from the ELF approach and need not be greater.
Acceptance Criteria:	A combination of strength and displacement criteria are employed.
Strength:	**Wood**- Permissible working stresses are specified. **Steel**- Permissible working stresses are specified, connections designed either on a strength basis - to exceed connected member strength, or on a working stress basis - for elevated forces intended to approximate the maximum forces likely to be delivered during an earthquake. Columns provided with sufficient axial strength to resist maximum force which can be delivered, calculated directly or by approximate methods. Capacity of moment-resisting beam-column connections to resist inelastic demands must be demonstrated by prototype test or approved calculations. **Reinforced Concrete**- Design strength of members must exceed factored combinations of dead, live and seismic loads determined from the analysis. Shear strength of flexural elements designed to exceed flexural strength. **Masonry**- Permissible working stresses are specified. Alternatively, for some elements, a strength approach in which the design strength of members must exceed factored combinations of dead, live and seismic loads determined from the analysis, is permitted.

	Displacement: Interstory drifts at force levels used in the analysis are controlled to approximately 0.5%, depending on the structural system.
Simple Approach	
Design Controls:	Provisions specify minimum criteria for: a. Configuration b. Connectivity to foundation c. Placement, size and type of vertical elements of the lateral force resisting system.

Design Procedures - Non-structural Systems

Cladding:	Out-of-plane attachments to structure designed for specified inertial force. Elements designed to withstand drift calculated for reduced forces, factored to approximate real "inelastic" displacements.
Mechanical/Electrical:	Anchored and braced to structure for specified inertial force.
Ceilings:	Prescriptive requirements for lateral bracing.
Furnishings:	No specific requirements.
Portable Equipment:	No specific requirements.

Construction Quality Assurance

Building Official:	Reviews design documents for conformance. Inspects construction for general conformance, at pre-determined intervals.. Reviews special inspection reports.
Design Professional:	Specifies special testing and inspections required beyond those mandated by building official. Periodic observation of job site conditions for general conformance. Review of special inspection and testing reports.
Testing Agency:	Conducts materials tests specified. Full time observation of selected work in progress. Inspection of selected work in progress and selected completed work. Report to building official and design professional.

Commentary on Effectiveness

I=1.0 Standard Occupancy Facility:

Structural Systems:

Highly effective in meeting performance objective for minor earthquakes (events producing response within elastic or near-elastic range of structure). However, the definition of this earthquake level is poorly defined and highly subjective.

Highly effective for "regular" structures in meeting performance objective for moderate earthquakes (events producing moderate inelastic response). However, the definition of this earthquake level is poorly defined and highly subjective. Moderately effective for irregular structures.

Generally effective in preventing life threatening damage for major earthquakes (largest events likely to be experienced by the structure). Limited effectiveness for irregular structures and structures incorporating brittle components, particularly when subjected to near-field ground motions from large events. Note that the effectiveness of the provisions for long duration events has not been observed and can not be objectively evaluated.

Specific Deficiencies:

Steel Moment-Resisting Frames. Questionable effectiveness for predicting their behavior in major earthquakes. Procedures rely on large ductility capacities in elements and connections, but do not prevent brittle failures in critical connections.

Flexible Diaphragms. Moderate effectiveness for preventing structural damage in moderate earthquakes. Procedures do not adequately consider high in-plane amplification of diaphragms at all levels of shaking. Although inelastic action may be concentrated in the diaphragms, load-reduction factors are generally based on the type of vertical element.

Out-of-Plane Wall Anchorage to Flexible Diaphragms. Limited effectiveness for preventing life-threatening damage in major earthquakes. Procedures do not consider high in-plane amplification of diaphragms or the limited strength and ductility of components of the wall anchorage system.

Nonstructural Systems:

Highly effective at controlling damage for minor earthquakes and generally effective for protecting life safety in moderate earthquakes, but untested for major earthquakes. Force-based design methodology appears to be appropriate for design of equipment anchorage. Limitations on effectiveness may be due to lack of enforcement and unclear responsibility for design of seismic resistive features between the various structural and engineering related disciplines.

Specific Deficiencies:

Precast Exterior Cladding Elements. Questionable effectiveness for preventing substantial damage in major earthquakes for all building types. Procedures do not directly consider real inter-story drifts that could occur in a major earthquake. In some cases, maximum possible drifts used in determining panel clearances may be two to three times less than actual drifts expected.

Furnishings: Not addressed.

Portable Equipment: Not addressed.

I=1.25 Essential or Hazardous Facility:

Structural Systems: Limited effectiveness in meeting performance objective for moderate and major earthquakes. The increase in design base shear of 25% alone does not ensure that the structure will remain essentially elastic in the more moderate event (perhaps half of the demand level of the Design Basis Earthquake). Structures designed with large implied ductilities in the DBE may still exhibit some nonlinear behavior in the more moderate event, and consequently damage. Provisions are more effective when coupled with highly regular building configurations, as well as enhanced design and construction quality assurance measures.
Moderately effective in limiting structural damage for major earthquakes (largest events likely to be experienced by the structure). Ductility demands in more flexible structures can be substantially reduced by the 25% increase in design base shear. Limited effectiveness for irregular structures and structures incorporating brittle components, particularly when subjected to near-field ground motions from large events. Note that the effectiveness of the provisions for long duration events has not been adequately observed and can not be objectively evaluated.

Specific Deficiencies: Steel Moment-Resisting Frames. Limited effectiveness for limiting structural damage in major earthquakes. Ductility demands on elements and connections are reduced, due to higher design base shear, but do not sufficiently to prevent brittle failures in critical connections.

Flexible Diaphragms. Moderate effectiveness for preventing structural damage in major earthquakes. Procedures do not adequately consider high in-plane amplification of diaphragms at low levels of shaking. Although inelastic action may be concentrated in the diaphragms, load-reduction factors are generally based on the type of vertical element.

Out-of-Plane Wall Anchorage to Flexible Diaphragms. Limited effectiveness for limiting structural damage in major earthquakes. Procedures do not consider high in-plane amplification of diaphragms or the limited strength and ductility of components of the wall anchorage system. Ductility demands are reduced by 25% design force increase, but not sufficiently.

Nonstructural Systems: Highly effective for moderate earthquakes and moderately effective for major earthquakes. Force-based design methodology appears to be appropriate for design of equipment anchorage. Enforcement is generally better for structures designed with I=1.25.

Ineffective for non-engineered nonstructural elements, such as suspended ceilings and fire-sprinkler piping.

Specific Deficiencies: Precast Exterior Cladding Elements. Questionable effectiveness for limiting damage in major earthquakes. Procedures do not directly consider real inter-story drifts that could occur in a major earthquake. Maximum possible drifts used in determining panel clearances are still 1.5 to 2.5 times less than actual drifts expected.

Furnishings: Not addressed.

Portable Equipment: Not addressed.

Standard: **California Building Code, 1991 edition** (also known as Title 24, California Code of Regulations)

Performance Objectives: *For Schools, Community Colleges, Hospitals, and State-owned or - Leased Essential Services Buildings:*

Resist Minor Earthquakes without damage.
Resist Moderate Earthquakes without structural damage and with light architectural (nonstructural) damage. Hospitals and essential services buildings are expected to be fully operational (functional), in so far as practile.
Resist a design level earthquake without collapse or endangerment of life safety, but with structural and architectural damage. Hospitals and essential services buildings are expected to be operational (functional), albeit in a faulted mode. The design earthquake is defined as having a 10% probability of being exceeded in a 50 year period or a recurrence interval of 475 years.

For some highly irregular structures, or structures incorporating unusual technology, there is also a requirement that structural stability be demonstrated for an Upper Bound or Maximum Capable Earthquake, with a 10% probability of exceedance in 100 years..

For other State-owned building structures:

Same as Uniform Building Code (UBC), i.e.:
Resist Minor Earthquakes without damage.
Resist Moderate Earthquakes with architectural (nonstructural) damage and with limited structural damage.
Resist Major Earthquakes without collapse or endangerment of life safety, but with potentially severe structural and architectural damage.

The upper bound earthquake is defined as a motion having a 10% probability of being exceeded in a 50 year period or a recurrence interval of 475 years.

Applicability: *State Chapters:* Design of new schools, community colleges, hospitals, and State-owned or -leased Essential Services buildings; evaluation and repair of existing structures with lateral force resisting elements conforming to code detailing requirements.

Other than State Chapters: Design of new structures other than schools, community colleges, hospitals, or Essential Services buildings; evaluation and repair of existing structures (other than schools, community colleges, hospitals, or Essential Services buildings) with lateral force resisting elements conforming to code detailing requirements.

Design Events:

For Schools, community colleges, Hospitals, and State-owned or -leased Essential Services buildings:

Most buildings are designed for a single event with a 10% probability of exceedance in 50 years (475 year return period), called a Maximum Probable Earthquake. For some highly irregular structures, or buildings incorporating unusual technology, stability must be demonstrated for an Upper Bound Earthquake, with a 10% probability of exceedance in 100 years.

For other State-owned building structures:

A single design event with a 10% probability of exceedance in 50 years (475 year return period) is used.

Ground Motion:

An elastic acceleration response spectrum tied to an effective peak ground acceleration. For the design of regular structures, standard spectral shapes may be scaled by zone coefficients to obtain the design spectrum. Site specific response spectra are required for the design of schools, community colleges, hospitals and State-owned or leased Essential Services buildings that are: a) irregular; b) located on sites with an S4 soil profile; or c) located in an Alquist-Priolo zone. Where design for an Upper Bound Earthquake is required, this must also be based on a site specific response spectrum representation. Scaling by zone coefficients is permitted with some limitations.

Design Procedures-Structural Systems and Elements

Design Approaches:

For Schools, community colleges, Hospitals, and State-owned or -leased Essential Services buildings:

All schools, community colleges, hospitals, and state-owned or -leased Essential Services buildings must be engineered. The general approach used is applicable to all structures and includes a combination of analytical provisions and prescriptive detailing requirements.

For other State-owned building structures:

Two approaches are permitted: 1) A *general* approach of engineered design intended to be applicable to all structures and includes a combination of analytical provisions and prescriptive detailing requirements; and 2) a *simple* approach applicable only to regular low-rise wood-frame construction with extensive prescriptive detailing requirements and no analytical provisions, i.e., a non-engineered structure.

General Approach: **Engineered design.**

For Schools, community colleges, Hospitals, and State-owned or -leased Essential Services buildings:

Design Controls: Provisions specify minimum criteria for:

a. Element connectivity and a complete lateral load path.
b. Limits on configuration and distribution of stiffness.
c. Permissible materials of construction.
d. Element detailing.
e. Elastic strength of lateral force resisting system.
f. Elastic lateral stiffness of structure.

Analytical Procedures: Three analytical procedures are defined: 1) the Equivalent Lateral Force (ELF) method; 2) the elastic Response Spectrum Analysis (RSA) method; and 3) the Time History (TH) method. In addition, alternative rational analysis are permitted.

Equivalent Lateral Force Method: The ELF method approximates the dynamic effects of strong ground motions on structures with an *equivalent* static force or ELF. The ELF method is limited to structures within specified limits of regularity and height; however, the method is applicable to most structures. Design forces and displacements in the lateral force resisting system are based on static elastic analysis for an applied lateral forcing function. The forcing function is representative of the distribution of inertial lateral forces typically obtained for elastic structures with a uniform distribution of mass and stiffness. The method assumes that the structure response will be dominated by the structure's fundamental mode shape. The structure is assumed to respond in an inelastic manner when subjected to the forcing function. Hence, the total applied lateral forces are scaled down, by a factor R_w to a fraction of the theoretical elastic response to account for the beneficial effects of inelastic behavior. The scaling factors are intended to account for period lengthening, damping, soil structure interaction, system and material overstrengths, redundancy, and other factors, and are based on the type of lateral force resisting system used.

Response Spectrum Analysis Method: Applicable to any structure. The structural forces and displacements used for design are calculated based on a scaled, elastic response spectrum analysis (RSA). The RSA method is applicable to all buildings except base-isolated structures. The structure is assumed to respond in an inelastic manner when subjected to the elastic response spectrum accelerations. Hence, the analysis results are scaled down to a fraction of the theoretical elastic response to account for the beneficial effects of inelastic behavior. The scaling factors are intended to account for period lengthening, damping, soil structure interaction, system and material overstrengths, redundancy, and other factors, and is based on the type of lateral force resisting system used. For irregular buildings, special limitations are placed on the extent the forces resulting from an RSA may be scaled down, based on the dynamic characteristics of the structure and the characteristics of the site specific spectrum.

Time History:

The elastic Time History method models the dynamic effects of strong ground motions on structures by integrating the equations of motion subjected to the applied time history. The TH method is applicable to all structures including base-isolated structures. Where discrete elements of the structure, such as base isolation systems, are have well defined inelastic characteristics, these inelastic properties may be used directly in the analysis. When an elastic analysis is performed, calculated forces and displacements in the lateral force resisting system are scaled down to a fraction of the theoretical elastic response to account for the presumed beneficial effects of inelastic behavior. The amount of scaling which may be performed is limited, in a similar manner to the RSA method.

Acceptance Criteria:

A combination of strength and displacement criteria are employed.

Strength:

Wood - Permissible working stresses are specified.
Steel - Permissible working stresses are specified.
Connections are designed either on a *strength* basis to exceed connected member strength, or on a *working stress* basis for elevated forces intended to approximate the maximum forces likely to be delivered during an earthquake. Columns are designed to have the axial strength to resist maximum force which can be delivered by the connected elements. This force can be calculated directly or by approximate methods.
Reinforced Concrete - Design strength of members must be greater than factored combinations of dead, live and seismic loads determined from the analysis. The shear strength of flexural elements is designed to exceed the flexural strength.
Masonry - Permissible working stresses are specified. Alternatively, for some elements, e.g., shear walls and wall frames, the strength design method in which the design strength of the elements must exceed factored combinations of dead, live, and seismic loads determined from the analysis, is permitted.

Displacement:

Interstory drifts at force levels used in the analysis are controlled to approximately 0.5%, depending on the structural system.

For other State-owned building structures,

Design procedures are similar to those contained in the Uniform Building Code.

Simple Approach:

Non-engineered prescriptive design.

For State-owned building structures (other than schools, community colleges, hospitals, and State-owned or -leased essential services buildings):

(Note: The simple approach is not allowed for schools, community colleges, hospitals, and State-owned or -leased essential services buildings)

Design Controls:

Provisions specify minimum criteria for
a. Configuration
b. Connectivity to foundation

c. Placement, size and type of vertical elements of the presumed lateral force resisting system.

Analytical Procedures: None. Prescriptive details only.

Acceptance Criteria: None.

Design Procedures - Nonstructural System

For schools, community colleges, Hospitals, State-owned or -leased Essential Services buildings, and for other State-owned building structures:

Cladding: Out-of-plane attachments to structure designed for specified inertial force. Elements designed to withstand drift calculated from reduced (scaled) forces and then factored to approximate real "inelastic" displacements. Generally designed by the static force or displacement method.

Mechanical/Electrical: Anchored and braced to structure for specified inertial force. The anchorage or support is generally designed by the static force or displacement method. The equipment itself is often not designed for seismic forces, but relies on its presumed inherent ruggedness for survival, e.g., a boiler or transformer.

Ceilings: Prescriptive requirements for lateral bracing and anchorage.

Furnishings: No specific requirements.

Portable Equipment: No specific requirements.

Construction Quality Assurance

For Schools, community colleges, Hospitals, and State-owned or -leased Essential Services buildings:

Building Official: Rigorous review of design documents is performed for conformance with code structural requirements by means of a detailed plan check. On-site inspection of construction for strict conformance with the approved drawings by a general inspector, supplemented with regular visits by a licensed structural engineer employed by the Building Official.

Design Professional: Structure design must be done by Architect (AOR) or Civil Engineer with authority to use the title *Structural Engineer (EOR). The AOR/EOR* specifies special testing and inspections required (T&I list), including those mandated by the building official. The AOR and/or EOR provide periodic observation of job for general conformance with the approved drawings, and review the special inspection and testing reports. Upon job completion, files a statement that to his/her knowledge, construction conformed with approved drawings.

Testing Agency:	Conducts materials tests and inspections specified in T&I list. Full time observation of selected work in progress, and completion inspection of other work. Prepares reports to Building Official and design professional.
	For other State-owned building structures:
Building Official:	Reviews design documents for conformance with code structural requirements by means of a cursory plan check.
Design Professional:	Similar to requirements for Schools, community colleges, Hospitals, and State-owned or -leased Essential Services buildings except that final statements are not required.
Testing Agency:	Similar to requirements for Schools, community colleges, Hospitals, and State-owned or -leased Essential Services.

Commentary on Effectiveness

For Schools, community colleges, Hospitals, and State-owned or -leased Essential Services buildings:

Structural Systems:

Highly effective in meeting performance objective for minor earthquakes (events producing response within elastic range of the lateral force resisting system).

Highly effective for *regular* structures in meeting the performance objective for moderate earthquakes (events producing moderate inelastic response in the lateral force resisting system). Moderately effective for *irregular* structures.

Generally effective for *regular* structures in preventing significant structural damage for major earthquakes (large events likely to be experienced by the structure and producing severe inelastic response in the lateral force resisting system). May have limited effectiveness for *irregular* structures and structures incorporating brittle components, particularly when subjected to near-field ground motions from large events.

Nonstructural Systems:

Schools and community colleges: Highly effective for minor and moderate earthquakes less effective for major events.
Hospitals and Essential Services Buildings: Highly effective for minor earthquakes; moderately effective for moderate earthquakes; and likely adequate though untested for major and Upper Bound earthquakes. The provisions do not adequately prevent damage to the large number of utilities, e.g., oxygen and vacuum, and nonstructural items installed in hospitals and essential services buildings.

Furnishings: Not addressed.

For other State-owned building structures:

Generally the same as above, except that reliability levels are lower due to the decreased quality of design and construction which may occur with the less rigorous quality assurance practices adopted for these buildings.

Standard:	**NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings - 1994**
Performance Objectives:	Minimize hazard to life for all buildings and non-building structures. Increase expected performance of higher occupancy structures as compared to ordinary structures. Improve capability of essential facilities to function during and after an earthquake.
Applicability:	Design of new buildings and non-building structures and portions thereof, additions to buildings and non-building structures, and nonstructural components and systems.

Design Events:	A single Design Basis Earthquake is used as the basis for design. This EQ is estimated to have a return period of 475 years.
Ground Motion:	An elastic acceleration response spectrum based on an effective peak ground acceleration selected from Seismic Ground Acceleration Maps (either zoned or contour maps may be used). The seismic coefficient is determined from site coefficients F_a and F_v which are a function of site soil profile and shaking intensity. The base shear V is determined by a formula $V = CsW$.

Design Procedures-Structural Elements

	Design requirements depend on the Seismic Performance Category which is selected based on the EPA and the Seismic Hazard Exposure Group. There are three Seismic Hazard Exposure Groups which are essentially Occupancy groupings.
Design Approaches:	Design is based on strength design approach. A general approach is given applicable to all buildings and non-building structures. Approved alternative procedures may be used to determine seismic forces and their distribution. Conventional Light Wood-Frame Construction which relies on prescriptive provisions is permitted with restrictive requirements as to height, seismic performance category, gravity dead load of roofs, exterior walls and floors.

General Approach:

Design Controls:

Minimum criteria are specified for:

a. Individual member sizing
b. Connections of all elements
c. Building deformations (story drift and P-delta effects)
d. Complete continuous load path(s)
e. Foundation designs with recognition of dynamic nature of forces and expected ground motions
f. Element and connection detailing
g. Elastic strength of lateral force resisting system
h. Interaction effects between rigid and less rigid elements
i. Vertical and horizontal irregularities
j. Orthogonal effects
k. Discontinuities in strength and stiffness
l. Vertical seismic forces on cantilevers and horizontal prestressed components.

Analytical Procedures:

Two analytical procedures are defined: Equivalent Lateral Force (ELF), and Modal Analysis (MA) both two- and three-dimensional response spectrum analyses. Alternative approaches such as more detailed modal analysis (several degrees of freedom per floor) and inelastic response time history analyses are permitted.

ELF:

Limited to structures with specified configuration restrictions and height limits, but applicable to most structures. Based on the approximation that the effects of yielding can be adequately accounted for by linear analysis of the lateral force resisting system for the design spectrum (elastic acceleration response spectrum reduced by R, the response modification factor). Base shear for moderate height buildings is limited to 2.5 Ca/R. An upper limit is specified for the approximate building period Ta based on the site coefficient Cv and EPA. The effects of soil-structure interaction may be considered in the analysis and design. The vertical distribution of seismic forces is a parabolic function based on a building with reasonably uniform weight and stiffness distribution.

MA:

Applicable to any structure. The MA methods defined are based on one degree of freedom per floor. The main difference between the ELF method and the MA is the distribution of lateral forces over the height of the building. In MA the distribution is based on properties of the natural vibration modes, which are determined from the actual mass and stiffness distribution over the height. If the lateral motions in the two orthogonal directions and the torsional motion are strongly coupled both the ELF and MA methods may be inadequate. For such cases, a more rigorous MA procedure or an inelastic time history analysis should be considered. Results cannot be less than 80 percent of the results from the ELF method.

Acceptance Criteria:

A combination of strength and deformation criteria are specified

Strength:

Wood: Strength design using 2.16 times allowable stresses for the material involved.

	Steel: LRFD or ASD specification. For ASD, allowable stresses converted into design strengths per AISC specifications. Seismic requirements are also defined for special concentrically braced frames, cold-formed steel members, steel deck diaphragms, and light framed walls (metal studs). **Reinforced Concrete:** Strength design is specified, provisions based on ACI 318-89 (revised 1992), and precast systems permitted (must emulate monolithic structure). Coupling beam provisions have been added. Plain concrete is permitted for foundation walls in selected performance category A structures. Appendix covering RC Structural Systems Composed From Interconnected Elements added for review, trial designs and comment. **Composite Steel and Concrete Structures:** Provisions for design and construction of composite systems combining structural steel and reinforced concrete in structures. Strength requirements same as for applicable sections of Structural Steel and RC chapters. **Masonry:** Allowable stress design specified for reinforced masonry and plain masonry construction. Strength design is permitted as an alternative using a reduction factor and 2.5 times the allowable working stress. **Passive Energy Dissipation Systems:** An appendix has been added covering new and relevant techniques for incorporating energy dissipation devices into EQR building designs and provide requirements necessary to ensure that the devices will perform as designed.
Displacement:	Interstory drifts are limited in accordance with the Seismic Hazard Exposure Group and type of building. Limits vary from 0.007 to 0.025 times story height.

Design Procedures - Nonstructural Systems

Cladding:	Attachments to structure designed for gravity loads and specified inertial forces; all fasteners designed for 4 times F_p. Elements designed to withstand amplified drift.
Mechanical/Electrical:	Anchored and braced or restrained to structure for specified inertial force, F_p. F_p varies depending on whether equipment is rigidly or flexibly attached to structure.
Ceilings:	Prescriptive or calculated inertial forces resisted by bracing to structure
Furnishings:	No specific requirements.
Portable Equipment:	No specific requirements.

Construction Quality Assurance

Building Official:	Not specified (NEHRP is a resource document).
Design Professional:	Develops Quality Assurance Plan for Regulatory review and approval. Periodic observation of construction conditions for general conformance.

	Reviews required special inspection and testing reports.
Testing Agency:	Conducts tests and inspections as specified in QAP. Report to Design Professional and Building Official.
Contractor:	Must submit statement of awareness of QAP and intent to comply prior to start of construction.

Commentary on Effectiveness

Structural Systems:	Highly effective for structures subjected to minor earthquakes. Should be effective for regular structures in moderate earthquakes. Moderately effective for irregular structures. Should be moderately effective in preventing life threatening damage for major earthquakes. Less effective for irregular structures. Duration of motion not considered explicitly, however, detailing requirements should help moderate influence of long duration strong motion.
Nonstructural Systems:	Should be highly effective for minor and moderate earthquakes, and moderately effective for major earthquakes.
Furnishings:	Not covered.
Portable Equipment:	Not covered, per se.

Standard:	**Seismic Design Guidelines for Essential Buildings, February 1986; Army TM 5-809-10-1, Navy NAVFAC P-355.1, Air Force AFM 88-3, Chap 13, SEC A**
Performance Objectives:	*Essential Buildings:* Remain elastic in Minor earthquakes. Remain nearly elastic in Moderate earthquakes. (Nearly elastic is interpreted as allowing some structural elements to exceed yield stresses, on the condition that the linear-elastic behavior of the overall structure is not substantially altered.) Remain functional in major earthquakes, with some structural damage possible. *Other Buildings*: Slightly reduced objectives, minimum objective equal to SEAOC/UBC.
Applicability:	May be used for any structure, but written for the design of new essential structures, high-risk structures, or irregular structures for the US Army, Navy, or Air Force. May also be applied to existing structures per TM5-809-10-2.

Design Events:	Two earthquakes are used as the bases for design. The first earthquake is a moderate level event, with a 50% probability of exceedance in 50 years (72-year return period). The second earthquake is a severe event, with a 10% probability of exceedance in 100 years (approximately a 1000-year return period).
Ground Motion:	The ground motion is presented in the form of an elastic acceleration response spectrum, which is a function of an effective peak ground acceleration. The ground motion is specified using 1) ATC 3-06 standard spectra with adjustments for damping and return period or 2) Guidelines for developing site-specific response spectra.

Design Procedures - Structural Elements

Design Approaches:	Emphasis is on an approach with two levels of design earthquake: 1) An elastic design for the 72-year earthquake and 2) A post-yield verification analysis for the 1000 year earthquake. For the post-yield analysis two methods are presented: an elastic analysis method using inelastic demand ratios (IDR's) and an approximate inelastic method using a pushover and response spectra, the Capacity Spectrum Method (CSM). Single level and modified two level alternatives are presented for non-essential buildings (structures classed as either "High Risk Buildings" or "All Other Buildings").

Design Controls:
(All Categories)

Provisions specify criteria for:

a. Element connectivity and continuous load path(s).
b. Adequate strength and stiffness of the structure and its members.
c. Permissible combinations of loading.
d. Permissible materials of construction.
e. Detailing of structural and nonstructural components.
f. Elastic and inelastic strengths of lateral force resisting system.
g. Elastic and inelastic stiffnesses of the structure.
h. Drift limits.

Analytical Procedures:

Three analytical procedures are defined an elastic modal response spectrum analysis (RSA), an elastic design method using inelastic demand ratios (IDR) and the Capacity Spectrum Method (CSM).

Response Spectrum Modal Analysis (RSA):

Required for essential facilities, (optional for others), a modal analysis is prescribed using a linear elastic model subjected to the 72-year return period earthquake response spectrum at a designated damping. The demands determined from the analysis, in conjunction with gravity loads, are compared to the elastic capacities of the structural components on the basis of the yield strengths of the materials. A limited number (e.g., 10% or 20%) of structural components are allowed to exceed their elastic flexural capacities by a limited amount for qualified lateral force resisting systems (e.g., ductile framing systems or other framing systems). These limitations were established as a means to satisfy the "nearly elastic behavior" criterion, which is "interpreted as allowing some structural elements to slightly exceed specified yield stresses on condition that the elastic-linear behavior or the overall structure is not substantially altered."

Inelastic Demand Ratio (IDR) Method:

The IDR method is an elastic analysis procedure that is essentially the same as the RSA procedures described above, with exceptions that allow its use for the "post yield" evaluation for the 1000 year earthquake. Some exceptions include the use of increased damping, longer building period and demand-capacity ratios (DCR's) (i.e., the demands divided by capacities can exceed unity). This method attempts to limit the inelastic demands on the actual structural components by limiting the maximum allowable forces in the elastic model to levels that would not produce unstable plastic rotations or displacements. Stability control is achieved through the ratio of the maximum force in the elastic model to ultimate strength of each component. A table of acceptable IDR values is given for various structural elements such as beams, columns, and walls for each building category (i.e., essential, high risk, and others). The acceptance criteria are satisfied if the DCR's are less than the IDR limits, the drift limits are not exceeded and a list of other conditions are met. These conditions include absence of unsymmetrical yielding in a horizontal plane, hinging of columns at a single story, discontinuities that cause instability and unusual distribution of DCR's relative to IDR's.

Capacity Spectrum Method (CSM):

The CSM is a procedure that approximates the inelastic response of a structure by means of a graphical representation of both structural capacity and earthquake demand on the same plot. The capacity curve is constructed from a pushover analysis that takes into account the dynamic characteristics of the structure (e.g., mode shapes, participation factors and effective modal weights). Using structural dynamics principles, the spectral acceleration (S_a) and spectral displacement (S_d) are calculated from the values of base shear and roof displacement, for the different stages of the pushover. A capacity curve of S_a vs S_d is plotted*. The 1000 year earthquake response spectra for a range of damping values are plotted on the same graph. An inelastic "demand" spectrum is formed by constructing an approximate transition curve between the elastically damped spectrum (at the elastic limit of the structure) to the equivalent inelastic damped spectrum (at the inelastic capacity of the structure). The intersection of the inelastic demand spectrum curve with the building's pushover capacity curve defines the approximate inelastic response of the structure. If the final data point of capacity curve is above the demand curve, the acceptance criteria is satisfied.

* In the Seismic Guidelines for Essential Facilities, S_a vs. T (structural period) is plotted, instead of S_a vs. S_d. S_a vs. S_d has proven to be a convenient set of coordinate axis, as the values of T can be overlaid as radial lines centered at the origin.

Acceptance Criteria:

Criteria for appropriate strengths and maximum displacements are specified.

Strength:

Wood-Frame Construction - In the absence of a strength design criteria working stress design is used, however the allowable stresses may be increased by 100%.
Structural Steel Construction - Allowable stress design with allowable stresses increased by 70%, or load resistance factor design, with unfactored loads are allowed for steel construction.
Reinforced Concrete Construction - Strength design per ACI, without load factors, is used.
Masonry Construction - Working stress design is used with an increase in allowable stresses of 70%.
Connections - Connections that do not develop the strength of the connecting members will have a strength reduction factor of 75% applied to the post-yield strength of the connecting material.

Displacement:

Interstory drifts under design earthquakes for essential buildings are limited to 0.005 x the story height for elastic design under the 72 year earthquake and 0.01 x the story height for analysis under the 1000 year earthquake. High risk buildings and all other buildings are allowed 40% and 50% greater drifts for the 72 year and 1000 year earthquakes respectively when using a two level design approach.

Design Procedures - Nonstructural Systems

General:

Apply to nonstructural elements which must remain functional after a major seismic event. Requires that all nonstructural elements and their anchorages be designed to resist forces and displacements caused by the motion of the building. Under the 72 year earthquake, elements and anchorages must remain elastic. Under the 1000 year earthquake, elements must not collapse, endanger life or cease to remain functional, if deemed necessary for continued function. The effects of the nonstructural elements on the performance of the building must also be investigated.

Analytical Procedure:

For the 72 year earthquake 1) Rigid equipment ($T < 0.05s$) is designed to resist the maximum floor acceleration obtained from the elastic modal analysis of the building and 2) Flexible equipment is designed to resist the force obtained from an appropriate floor response spectra. Guidelines are given for obtaining the floor response spectra. The methodology is similar for the 1000 year earthquake, except the maximum floor acceleration and maximum floor spectral accelerations are limited to the maximum that can be derived based on A) the maximum elastic capacity of the structure or B) the maximum acceleration of the structure based on the post-yield response.

Cladding:

Attachment to building must accommodate calculated story drifts under design earthquakes or 1/2" whichever is greater. System also must be capable of resisting forces equal to its weight times the design acceleration.

Mechanical/Electrical:

Equipment certification for postulated displacements and accelerations is required. The following requirements apply to essential systems. Fire protection equipment is governed by NFPA No. 13 and NFPA 20. Each facility must have an emergency power system with separate emergency circuits and an alternate on site power source. The elements of the emergency power system and mechanical systems must be braced.

Ceilings: Minimum force levels are specified for the design of suspended ceiling systems. Bracing against lateral and vertical motion is required and detailing is prescribed. Ceiling shall be isolated from the walls or partitions to allow movement, and walls shall not support ceilings.

Furnishings: Essential or potentially hazardous furnishings are required to be braced.

Portable Equipment: Essential portable equipment, under the general category of essential support equipment, is required to be: checked for ability to resist its inertial force, braced in a manner convenient for daily use, and checked to insure it will not be damaged by excessive deformations or movement of adjacent elements.

Construction Quality Assurance

Construction quality assurance is enforced through a quality control program developed by the federal government which is directed by the construction contractor.

Commentary on Effectiveness

Structural Systems: High effective in meeting performance objective for minor level earthquake.

If designed using two level approach, highly effective in meeting performance objectives for moderate events because structures are designed to remain elastic for a moderate (72 year return period) event. If designed using a modification of the two level or alternatively a single level approach, the performance would likely not be as good, but as a minimum would be equivalent to the ELF method of the UBC.

For major earthquakes, the performance has not been tested, but would possibly be much better than a UBC approach because the designer is forced to examine the building in the post-elastic range thus identifying the potential for undesirable structural behavior.

Nonstructural Systems:

Cladding: Highly effective in meeting performance objective for minor or moderated events. Although cladding systems designed to this standard have not been subjected to major seismic events, performance would likely be good based on the fact that the design forces are derived from floor accelerations under the 1000 year earthquake.

Mechanical/Electrical: High effective in meeting performance objective for minor or moderated events. Although performance is unknown for major seismic events, independent back-up power systems are required which reduces the risk of complete loss of power.

Ceilings: Highly effective in meeting performance objective for minor or moderated events. Performance when subjected to a major seismic

	event, is not known but would likely be satisfactory, as lateral and vertical bracing requirements are stringent and light fixtures are required to have independent means of support.
Furnishings:	Highly effective in meeting performance objective for minor or moderated events. Performance in a major seismic event is unknown.
Portable Equipment:	Highly effective in meeting performance objective for minor or moderated events. Performance of portable equipment performance in a major seismic event is not known.

Standard: **Uniform Code for Building Conservation, Appendix Chapter 1: Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings, 1991**

Performance Objectives: Provisions are intended to reduce risk of life loss or injury by reducing the risk of structural collapse and falling hazards. Provisions are intended to provide life safety performance for median estimates of ground motion from 475 year earthquakes, as approximately defined by the seismic zone coefficients contained in the Uniform Building Code. Structures are designed to be at the limit state for protection of life safety, at this design level and little margin may remain for more severe seismic demands. Life threatening damage may occur in some structures when subjected to very strong shaking.

Applicability: May be used for the evaluation of unreinforced masonry bearing wall buildings for life safety hazards and reduction of risks associated with these hazards through structural strengthening. Provisions are intended primarily for use on clay brick masonry structures containing lime mortar but can be adapted to apply to bearing wall structures incorporating other types of unreinforced masonry.

Design Events: Design is performed for a single "major earthquake" event, producing strong ground motion approximating that which would occur, with median probability, one time in a 475 year period.

Ground Motion: Uniform Building Code "Z" factors are used to reflect the effective peak ground acceleration for the design event.

Evaluation Procedures-Structural Elements

Evaluation Approaches: Two approaches are permitted: A General Procedure applicable to any unreinforced masonry bearing wall building, and a Special Procedure intended for buildings with relatively rigid shear walls and relatively flexible diaphragms.

The General Procedure is based on the UBC equivalent lateral force procedure and specifies a minimum design base shear approximately equal to 75% of the UBC value for conforming masonry bearing wall construction. Lateral forces are distributed to the structure assuming a first mode response of the shear walls.

The Special Procedure is based on research which explicitly modeled the limit states of URM bearing wall buildings and accounts for the typical inelastic behavior of rigid shear wall - flexible diaphragm systems. It assumes that the diaphragms will yield and decouple the response of the diaphragm and tributary elements from the response of the shear walls. Inertial loads in shear walls assume rigid body response of the walls, without amplification of base motion. Diaphragm response is limited based on yield capacity of diaphragm and damping provided by interior partitions.

General Procedure:

Design Controls: Provisions specify criteria for:

a. Evaluating capacity of existing materials
b. Out-of-plane adequacy of walls
c Anchorage of walls to diaphragms
d. Determining shear capacity of walls
e. Bracing parapets
f. Provision of redundant gravity load support
g. Diaphragm design (similar to UBC)

Analytical Procedures: Static lateral forces are applied to the building in a method similar to the Equivalent Lateral Force Technique contained in the UBC. Forces are scaled to 75% of the base shear requirements for conforming bearing wall construction under the UBC.

Acceptance Criteria: Limiting values are given for:

a. Masonry quality, as determined by mortar shear strength and percent mortar fill in collar joints
b. Slenderness (h/t) of walls as a measure of out-of-plane capacity
c. Permissible diaphragm stresses
d. Permissible shear loads in masonry piers, controlled either by masonry shear stress or rocking behavior of pier
e. Wall to diaphragm tension and shear anchor systems.

Displacement: Drift not directly addressed.

Special Procedure:

Design Controls: Provisions specify criteria for:

a. Configuration of building and lateral force resisting system
b. Evaluating capacity of existing materials
c. Out-of-plane adequacy of walls
d Anchorage of walls to diaphragms
e. Determining shear capacity of walls
f. Bracing parapets
g. Provision of redundant gravity load support
h. Diaphragm design to control out-of-plane deformation of masonry walls

Analytical Procedure: A static lateral force procedures is used to design shear walls. Forces are calculated assuming each wall is subjected to a uniform acceleration related to the effective peak acceleration coefficient "Z". Diaphragms are assumed to load shear walls based on a similar acceleration, but not exceeding their shear capacity. When extensive interior partitions ("cross walls") with appropriate characteristics to dissipate energy are present within the building, force levels may be reduced.

Acceptance Criteria:

Limiting values are given for:
a. Masonry quality, as determined by mortar shear strength and percent mortar fill in collar joints
b. Slenderness (h/t) of walls as a measure of out-of-plane capacity
c. Diaphragm shear demand/capacity ratios, based on expected shear deformation and tolerance of walls for out-of-plane deflection
d. Permissible shear loads in masonry piers, controlled either by masonry shear stress or rocking behavior of pier
e. Wall to diaphragm tension and shear anchor systems..

Displacement:

Presumed to occur primarily through diaphragm shear deformation and controlled through limits on diaphragm demands.

Design Procedures - Non-structural Systems

Parapets:

Parapets exceeding h/t ratios of 2.5 must be braced.

Suspended ceilings:

Plaster systems must be anchored to perimeter walls or joists/rafters above.

Other:

Not addressed.

Construction Quality Assurance

Building Official:

Reviews design documents for conformance. Inspects construction for general conformance. Reviews special inspection reports.

Design Professional:

Specifies initial materials capacity evaluation for building. Specifies construction period special testing and inspections required beyond those mandated by building official. Observes construction for general conformance with design documents. Reviews special inspection and testing reports.

Testing Agency:

Conducts material tests (masonry mortar strength). Provides continuous inspection of selected work. Reports to building official and design professional.

Commentary on Effectiveness

Structural Systems:

Highly effective in reducing risk in minor, moderate, and major earthquakes.

Moderately to highly effective in preventing life threatening damage in short duration major earthquakes.
Limited to moderate effectiveness in preventing life-threatening damage in major earthquakes.
Effectiveness for long duration events has not been tested and cannot be evaluated.

Specific Deficiencies: Building corners, narrow piers, buildings with flexible frames at open storefronts, areas with poor mortar, and areas around reinforced masonry openings were areas of concentrated damage in the Northridge earthquake.

Plan irregularities: Irregularities are not addressed in the provisions and have contributed to unanticipated damage.

Building response to ground motions which exceed the design level are not addressed, and consequently, building performance in rare to very rare events may be unacceptable.

Non-structural Systems: Except for parapets, generally not addressed in the provisions.

Specific Deficiencies: Poorly anchored veneer and non-bearing walls with high h/t ratios were badly damaged in Northridge earthquake.

Furnishings: Not addressed in provisions.

Equipment: Not addressed in provisions.

Standard:	**DOE Standard Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities DOE-STD-1020-94, April 1994 (formerly UCRL-15910)**
Performance Objectives	*Category 1:* Maintain occupant safety in an event with a mean return period of 500 years. (This is intended to be equivalent to model building codes.) This is similar to the Basic Objective. *Category 2:* Maintain occupant safety and minimal interruption of operations in an event with a mean return period of 1,000 years. This is similar to the Essential/Hazardous Facility Objective. *Category 3:* Maintain occupant safety, maintain continued operation, and maintain confinement of hazardous materials in an event with a mean return period of 2,000 years (1,000 years for facilities near tectonic plate boundaries). This is similar to the Safety Critical Objective, but is somewhat less restrictive for facilities near tectonic plate boundaries. *Category 4:* Maintain occupant safety, maintain continued operation, and maintain confinement of hazardous materials in an event with a mean return period of 10,000 years (5,000 years for facilities near tectonic plate boundaries). This is somewhat more restrictive than the Safety Critical Objective.
Applicability:	Design of new facilities and evaluation of existing facilities (possible modification of annual exceedance probabilities by a factor of approximately 2 if necessary for existing facilities).
Design Events:	A single, but separate Design Basis Earthquake is used as the basis of design for each performance category. Performance categories are mutually exclusive. The level of ground acceleration is determined from a site-specific probabilistic seismic hazard analysis. The average return period is based on the performance category, as indicated above.
Ground Motion:	A uniform-hazard elastic response spectrum based on the probabilistic seismic hazard analysis.

Design Procedures - Structural Elements

Design Approaches: The design approach is dependent on the performance category. For facilities in categories 1 and 2, the equivalent static or dynamic elastic analysis procedures in the Uniform Building Code are permitted. For facilities in categories 3 and 4, a dynamic elastic analysis according to the Tri-Services manual for seismic design of essential buildings (with modifications) is required. In either approach, the demands are based on the design event and ground motion defined above. *Only the latter requirements are discussed below.*

General Approach:

Design controls: The design detailing provisions of the UBC apply to all facilities. The DOE standard provides the methodology to determine the minimum required elastic strength of elements to meet the specified performance objectives.

Analytical Procedure: Dynamic analysis is required, either by the response-spectrum method or by the time-history method. Two horizontal components and one vertical component of ground motion are to be considered. Recommended criteria for dynamic analysis (including combination of demands along different axes) are specified in ASCE 4. Forces and displacements in the lateral force resisting system are directly determined from an elastic analysis. Demand/capacity ratios are calculated for all elements. When a significant number of DCRs exceed unity (including inelastic behavior) forces may be reduced to account for system damping, based on specified system damping values at various levels of response. Soil-structure interaction may be explicitly computed using soil springs or finite-element models. As an alternative, the criteria present conservative SSI reduction factors to be applied to the input ground motion. Inelastic energy absorption factors (F_{μ}) are employed to reduce element force levels from earthquake effects, based on presumed permissible levels of inelastic demand. These range from 1.0 for brittle systems to 3.0 for ductile systems. For facilities in category 4, the response is scaled up by a factor of 1.25 (or as computed based on the slope of the hazard curve) to ensure a conservative estimate of ground motion at extremely low probability levels.

Acceptance Criteria: A combination of strength and displacement criteria are employed.

Strength: The general philosophy of the provisions is that the forces derived from the response-spectrum analysis are divided by the inelastic energy absorption factors, added to the imposed gravity loads, and compared with element strengths. Connections are to be designed to develop the strength of the members or with inelastic energy absorption factors limited to 1.0. Inelastic energy absorption factors for columns are limited to 1.5 for flexure and 1.0 for shear and compression.

Element strength for concrete is defined by UBC Chapter 26, while element strength for steel is defined by UBC Chapter 27 (LRFD design).

	Code defined strength-reduction (phi) factors are retained. Strength of masonry elements is not defined, but is presumed to be similarly based on LRFD concepts. Wood elements are not anticipated to be employed in the design of DOE facilities.
Displacement:	Interstory drifts based on unreduced spectral demands are controlled to 1.0%, except when drifts are dominated by shear distortion. In this case, the total drift is limited to 0.4%.

Design Procedures - Non-structural Systems

There is only one set of requirements that applies to all structures, systems, and components. Non-structural elements are to be analyzed and designed according to the dynamic procedures specified above. In-structure elastic response spectra are to be employed for elements supported above the ground level. Relative motion between supports is to be considered explicitly. Evaluation of elements may also be done by testing or through the use of experience data.

Ceilings:	Prescriptive requirements for lateral bracing from the UBC apply.
Furnishings:	No specific requirements.
Portable Equipment:	No specific requirements.

Construction Quality Assurance

Criteria Enforcement:	No single entity acts as the criteria-enforcement agency. The construction project is generally overseen by the DOE contractor responsible for the site. DOE personnel also review documents and observe construction, but not on a predictable basis.
Peer Reviewer:	Reviews design documents for conformance with criteria. Periodic observation of job site conditions for general conformance.
Design Professional:	Specifies special testing and inspections required. Periodic observation of job site conditions for general conformance. Review of special inspection and testing reports. May or may not be the designer of record for the project, due to separate bidding design and construction administration contracts.
Testing Agency:	Conducts materials tests specified. Full time observation of selected work in progress. Inspection of selected work in progress and selected completed work. Report to DOE site contractor and design professional.

Commentary on Effectiveness:

Performance Categories 3 and 4:

Structural Systems: Presumed to be moderately effective for meeting both category 3 and 4 performance objectives. The hazard level is well defined and is in fact incorporated into the selection of the DBE. The use of inelastic energy absorption factors for reduction of elastic design forces and correlating them directly to expected damage is inexact. However, the factors are small and very little inelastic demand is permitted relative to the UBC approach. The procedures are more effective for structures employing low inelastic energy absorption factors. The procedures are highly effective for regular structures, in which the inelastic deformations will be more uniformly distributed, leading to more predictable force-vs.-damage relationships. The procedures rely heavily on the detailing provisions of the UBC to ensure structural regularity, and thus predictable performance.

The procedure is expected to be used primarily for the evaluation of existing structures and equipment, rather than for new design. However, there are few guidelines for treating elements of structures that do not comply with currently accepted detailing and proportioning practices. This will tend to limit the procedure's effectiveness.

Nonstructural Systems: Highly effective for meeting both category 3 and 4 performance objectives. The use of in-structure, elastic response spectra for analysis of equipment adds additional conservatism to the design, due to the expected inelastic behavior of the structure. Because the equipment in category 3 and 4 facilities is expected to remain functional, inelastic demands are not permitted and the analysis represents a conservative envelope of the actual forces and deformations expected in the equipment.

Furnishings: Not applicable.

Portable Equipment: Not applicable.

Standard: **CSU Guidelines for Seismic Retrofit**

Performance Objectives: The performance objectives are intended to provide reduced risk of loss of life during seismic events. Buildings may experience damage beyond economic repair but are expected to provide reasonable protection against global collapse. There are no provisions for damage control of contents.

Applicability: The provisions are for use in the retrofit of existing CSU facilities.

Design Events: A single Design Basis Earthquake is used for the seismic performance evaluation and seismic design. The event is a ground motion having a 20% chance of exceedance in 50 years. A survey of campuses was conducted and the zero period acceleration anchor for the design spectrum is provided for each of the campus locations.

Design Procedures - Structural Elements

Design Approach: Two basic approaches are provided. For simple buildings the designer is allowed to use an Equivalent Lateral Force (ELF) approach for both the evaluation and design. The evaluation is based upon the unreduced response with design capacity measured by element inelastic demand ratios. New elements would be apportioned and designed in accordance with California Title 24 regulations.

More complex buildings are evaluated on a displacement demand basis using dynamic analysis techniques and the unreduced response spectrum as the demand. Maximum allowable drifts are based upon the ability of the elements to deflect as established by principles of mechanics using strain limits and assuming plane sections to remain plane. Since the approach is not prescriptive and is based in some manner on the maximum story drift appropriate for the specific building, the approach is in principle a performance based seismic design. The approach could also be modified to suit more complex criteria such as little or no damage under events with a likely occurrence and life safety or damage control under events characterized by stronger shaking.

Since it is the actual displacements which are being controlled, the ground motion used is the unreduced design response spectrum or an unreduced acceleration history scaled to provide the equivalent demand. In as much as the principal parameter controlled is displacement, and the story drifts calculated are a direct result of the assumed stiffness, it is absolutely critical than the building system be correctly modeled. The success of the method also depends upon the designer being able to correctly assess appropriate deformation limits of the various substructures. For this reason the use of peer review is required to use this design approach. A quasi dynamic approach is also recognized whereby the post yield performance of the building is assessed using a pushover technique. The maximum allowable story drifts are established using the same principles as when applying the dynamic analysis procedure.

Again new elements are apportioned to provide drift control but are designated and detailed to conform to current California Title 24 requirements.

Acceptance Criteria: A combination of strength and displacement criteria are employed.

Strength: Strength criteria are applied for simple, small, or stiff building types. For wood buildings code strength values are used with an appropriate inelastic demand ratio to relate to the demand calculated as the expected value and not reduced by R factors. For unreinforced masonry buildings the 1991 UCBC would be used for both demand and capacity. The UCBC procedure is a codified method that appropriately measures demand and capacity in terms of the expected demand and the expected capacity. The ZPA of the UCBC would be replaced by the Campus specific design value. Reinforced masonry and reinforced concrete in stiff, shear controlled buildings would use inelastic demand ratios as defined in the Tri-Services Design Manual. Steel buildings would use plastic capacities modified to reflect an appropriate inelastic deformation demand. All IDRs are reviewed and approved by the peer reviewer.

Displacement: For concrete and masonry buildings controlled by flexural yielding, the maximum deformations are calculated assuming plane sections remain plane and a maximum compressive strain of 0.003 for unconfined material. Length of hinge region, superimposed axial loads, and maximum strains for partially confined material are subject to peer review.

For flexural steel buildings, the maximum global deformation is based on the ability of the connections to develop plastic hinging, and the system capacity to avoid forming local or global collapse.

Design Procedures - Non-structural Systems

No specific requirements are specified for nonstructural elements. California Title 24 would be the default criteria for anchorage and cladding. There are no expectations for contents.

Construction Quality Assurance

Design Professional: Specifies testing and inspections he requires to perform the evaluation and repair. The code is not prescriptive and allows the designer the option to choose the method and procedures he feels will best serve the performance objectives.

Peer Reviewer: A peer review is mandatory for all projects conducted under these provisions. The peer review is initiated at the beginning to ensure the design effort will proceed in a mutually agreed manner. Peer reviewer writes a summary report accepting the solution.

Commentary on Effectiveness:

Advantages: The approach will be particularly useful for existing buildings where performance objectives can be realized without having to meet arbitrary prescriptive standards.

Disadvantages: The design approach is more complex than the conventional technique and the results are sensitive to modeling assumptions. Improper assumptions can yield problem designs. The displacement based approach requires the use of sophisticated computer modeling and while usually less expense to construct can prove to be more costly to design.

Two principal elements of the process require additional development before this technique will see widespread application. These are defining a reliable method for predicting the post yield performance of materials and simplified methods for predicting building response during inelastic demands.

As the approach is new, it has not yet been tested. The approach is currently undergoing calibration.

Research Needs: Develop affordable, user friendly 3D nonlinear time history programs for performance assessment.

Define techniques for reliably predicting nonlinear behavior of building elements

Identify simplified methods for estimating nonlinear performance.

Standard: **ICSSC RP-4: Standards of Seismic Safety for Existing Federally Owned or Leased Buildings, February 1994**

Performance Objectives: Minimum specified performance objective: *Substantial Life-Safety:* Performance objective where earthquake may cause significant building damage that may not be repairable, though it is not expected to significantly jeopardize life from structural collapse, falling hazards or blocked routes of entrance or egress.

Other performance objectives are suggested and available criteria specified using previously defined standards already in use by the Federal government.

Applicability: Evaluation and mitigation of seismic hazards in existing owned or leased Federally-owned buildings.

Design Events: Minimum requirements based on FEMA 178: A single earthquake ground motion is used as the basis of design. This design earthquake has a 10% probability of being exceeded in 50 years, or a return period of 475 years.

Ground Motion: An elastic effective peak acceleration response spectrum (ATC-3 style) tied to a peak velocity-related acceleration coefficient. Approximate curves are provided.

Evaluation Procedures-Structural Elements

Evaluation Approaches: Two approaches are permitted in FEMA 178: A general approach intended to be applicable to all structures and which includes basic checklists and statements for the building system, vertical systems resisting lateral forces, diaphragms and connections. A *building-type approach* may be used as an alternative to the general approach and which includes checklists and statements for each of the 15 building types.

Using the *building-type approach,* the checklist is tailored to each building type. If the building being evaluated is composed of a combination of structural systems, more than one checklist may be used. Using the *general approach,* the final checklist for each building must be distilled from the general list of statements.

General Approach:

Design Controls:

The FEMA 178 provisions specify evaluation statements for:
a. basic building system
b. vertical systems resisting lateral forces
c. diaphragms and horizontal bracing systems
d. structural connections

Analytical Procedures:

The FEMA 178 evaluation process includes the following steps:
a. visit the site and collect data
b. select all pertinent statements and complete checklist
c. complete quick check of building lateral force resisting system
d. complete detailed evaluation of all statements found to be false
e. finish report

The evaluation process includes a simplified equivalent lateral force method which yields base shears equal to approximately 75% of forces required for new buildings by FEMA 222 (NEHRP). The quick check of building's lateral force resisting system includes a check of average shear stress of drift of a particular story.

Acceptance Criteria:

The total demand, Q is calculated using load combinations, factored loads and adjustments for lack of ductility. The capacity, C, is calculated for each element and checked based on the normal design procedure as specified. The basic acceptance criteria is: Q must be less than or equal to C. If all significant elements meet the basic acceptance criteria, no further analysis is needed. Elements that do not meet the acceptance criteria are the deficiencies that should be addressed in further analytical studies or in a rehabilitation program.

Building-Type Approach

Design Controls:

The FEMA 178 provisions specify evaluation statements for:

a. each of the 15 types of building system
   1. Wood, Light Frame
   2. Wood, Commercial and Industrial
   3. Steel Moment Frame
   4. Steel Braced Frame
   5. Steel Light Frame
   6. Steel Frame with Concrete Shear Walls
   7. Steel Frame with Infill Shear Walls
   8. Concrete Moment Frame
   9. Concrete Shear Walls
   10. Concrete Frame with Infill Shear Walls
   11. Precast/Tilt-Up Concrete Walls w/ Lightweight Flexible Diaphragm
   12. Precast Concrete Frames with Concrete Shear Walls
   13. Reinforced Masonry Bearing Walls w/ Flexible Diaphragms

14. Reinforced Masonry Bearing Walls w/ Precast Concrete Diaphragm
15. Unreinforced Masonry Bearing Wall Buildings

Analytical Procedures:

The FEMA 178 evaluation process includes the following steps:
a. visit the site and collect data
b. select all pertinent statements and complete checklist
c. complete quick check of building lateral force resisting system
d. complete detailed evaluation of all statements found to be false
e. finish report

The evaluation process includes a simplified equivalent lateral force method which yields base shears equal to approximately 75% of forces required for new buildings by FEMA 222 (NEHRP). The quick check of building's lateral force resisting system includes a check of average shear stress or drift of a particular story.

Acceptance Criteria:

The total demand, Q is calculated using load combinations, factored loads and adjustments for lack of ductility. The capacity, C, is calculated for each element and checked based on the normal design procedure as specified. The basic acceptance criteria is: Q must be less than or equal to C. If all significant elements meet the basic acceptance criteria, no further analysis is needed. Elements that do not meet the acceptance criteria are the deficiencies that should be addressed in further analytical studies or in a rehabilitation program.

Evaluation Procedures - Nonstructural Systems

General:

The evaluation of nonstructural elements follows the checklists and procedures in Chapter 11 of FEMA 178. However, RP4 requires only items in the FEMA 178 nonstructural checklist deemed by the assessing engineer as serious threats to life-safety need to be considered. A list of items considered life-safety hazards by RP4 is given.

Cladding:

Specific evaluation statements to the following life-safety hazards:
a. connections of external cladding, glazing, and veneer to structure
b. unbraced parapets, cornices, ornamentation, and appendages

Mechanical/Electrical:

Specific evaluation statements for the following life-safety hazards:
a. mechanical equipment suspended from the ceiling without bracing
b. elevators with counter weights not adequately braced or secured
c. hazardous materials

Ceilings:

Specific evaluation statements for the following life-safety hazards:
a. unreinforced masonry partitions 2.5m (8 feet) tall or more and hollow clay tile partitions, not already excluded by

	FEMA 178
Furnishings:	Specific evaluation statements for the following life-safety hazards: a. nonstructural furnishings or elements that may fail or impede egress
Portable Equipment:	No specific requirements.

Evaluation Procedures - Geologic/Site Hazards

General:

The information in FEMA 178 on the evaluation of foundations and geologic/site hazards is fairly general and thus is supplemented with the procedure from ATC-26-1, *Rapid Evaluation Procedure for Geologic Hazards*. This procedure was developed for use by an engineer without extensive geotechnical or geologic experience. As the ATC-26-1 procedure covers a broad range of potential hazards, the evaluating engineer is cautioned to carefully consider the life-safety implications of any potential geologic hazard.

Foundations:

Specific evaluation statements in FEMA 178 for:
a. condition of foundations
b. capacity of foundations

Geologic/Site Hazards:

Specific evaluation statements in ATC-26-1 and FEMA 178 for:

a. soil liquefaction*
b. slope failure/landslides*
c. surface fault rupture
d. differential compaction*
e. tsunami-induced flooding*

* May not pose a risk to life-safety - final decision left to evaluating engineer

Evaluation Procedures - Adjacency Hazards

General:

FEMA 178 specifically considers potential hazards related to pounding of adjacent buildings, but not hazards dealing with common walls or falling hazards posed by one building onto an adjacent building. The evaluating engineer is directed to complete FEMA 178 for both buildings and consider the issues identified in Section 3.4 of FEMA 178 as well as consider other adjacency issues as addressed in RP4. Detailed evaluation methods are to be determined by the evaluating engineer. As the damage due to adjacent buildings can be difficult or impossible to mitigate, the evaluating engineer is warned to make certain that the adjacency condition poses a significant threat to life-safety before recommending mitigation.

Adjacency:	Specific evaluation requirements in FEMA 178 for: a. pounding Explanation of potential hazard given in RP4: a. common walls b. falling hazards

Commentary on Effectiveness

General: Most of the comments on RP4 are really grounded in the effectiveness of FEMA 178. As the procedures in FEMA 178 are improved, so will the procedures in RP4. However, an effort has been made to address perceived deficiencies in the FEMA 178 document regarding a number of issues including: provisions for nonstructural elements, geologic/site hazards, and adjacency hazards.

Structural Systems: FEMA 178 focuses on building systems, historic record of deficiencies, and permits non-ductile systems in a systematic way. It is a third-generation document which has developed from a large consensus process into a refined and reliable document for evaluating existing buildings.

Nonstructural Systems: Unlike FEMA 178, RP4 specifically designates those nonstructural elements that can pose a significant threat to life-safety. This results in a much shorter list than FEMA 178 to both evaluate and possibly mitigate.

Geologic/Site Hazards: RP4 contains a unique treatment of geologic and site hazards not found in FEMA 178. The incorporation of the evaluation contained in ATC-26-1, originally developed for the US Postal Service but never officially published, is a large step forward in evaluating these hazards.

Adjacency Hazards: Adjacency hazards are difficult to evaluate and even more difficult to mitigate. Only significant life-safety hazards due to adjacency should end in mitigation. Because the building owner in the case of RP4 is always the Federal government, the approaches presented in RP4 for dealing with adjacency hazards are probably not practical to private building owners.

Structural Engineers Association of California

VISION 2000

Performance Based Seismic Engineering of Buildings

Part 2: Conceptual Framework

Prepared for:

California Office of Emergency Services

Prepared by:

Structural Engineers Association of California
Vision 2000 Committee

April 3, 1995
Final Report

Performance Based Seismic Engineering of Buildings

Part 2: Conceptual Framework

prepared by

Structural Engineers Association of California

Vision 2000 Committee

Chris Poland, Chair
James Hill, Vice Chair
Roland Sharpe, Vice Chair

delegates:

Vitelmo Bertero
Joseph Nicoletti
Ronald Hamburger
Nabih Youssef
Anthony Court
Scott Stedman
Ajit Virdee
Robert Chittenden
Richard Phillips

editor:

Jeffrey Soulages

ad hoc members:

Robert Thacker
Vilas Mujumdar
Charles Thiel, Jr.

ex-officio members:

Ronald Nelson
Arthur Ross
Jane Andersen

contractors:

Richard Fallgren

555 University Avenue, Suite 126
Sacramento, CA

April 3, 1995

SEAOC Vision 2000 Committee

In recognition of the need for a new generation of design procedures, the Structural Engineers Association of California (SEAOC) has formed a special committee to look into the future and develop a framework for procedures that will yield structures of predictable seismic performance. This committee, called Vision 2000, first started their deliberations in 1992 and continues today with a multifaceted effort that is aimed at both the final goal and at the process that will deliver the profession to that end. While Vision 2000 intends to identify the framework, others are expected to translate it into design guidelines and codes as appropriate. Within SEAOC, the Seismology Committee will incorporate the new concepts, as they are available and appropriate, into future editions of the SEAOC Blue Book.

The first step of the Vision 2000 Committee was to establish the project parameters. These project parameters were incorporated into a Goal/Mission Statement. The goal of the Vision 2000 is as follows:

> **The goal of the Vision 2000 Committee is to develop the framework for procedures that yield structures of predictable seismic performance.**

This framework will explicitly address life-safety, damageability, and functionality issues. The intent will be to produce structures whose seismic performance is predictable with a reasonable degree of confidence provided the actual materials and construction accurately reflect the design.

The second step was to establish a work plan and timeline for both the short-term phase of the project and for the mid-term and long-term phases of the project. A copy of this Workplan is available from the SEAOC Office.

The third step was to create the Vision 2000 Report which includes the results from all phases of the short-term work of the committee. This report includes: performance based engineering definitions, interim recommendations on how to apply performance based engineering concepts using today's available standards, a conceptual framework for a future performance based design code, preliminary lessons from the Northridge Earthquake, and a road map on moving the SEAOC Blue Book towards performance based engineering concepts. This report is divided into four parts as follows:

Part 1: Interim Recommendations
Part 2: Conceptual Framework
Part 3: Preliminary Northridge Lessons
Part 4: Moving the Blue Book Towards Performance Based Engineering

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Executive Summary

Introduction

This report presents a methodology for performance based seismic engineering, as the first stage of a three stage initiative to develop performance based engineering provisions. It was prepared by the Structural Engineers Association of California, Vision 2000 Committee with funding by the California Office of Emergency Services following the January 17, 1994 Northridge earthquake.

The Performance Based Engineering Methodology is presented in the following three sections as a conceptual framework, possible design approaches and possible acceptability analysis procedures. The conceptual framework outlines the proposed expanded scope of structural design needed to achieve structures of predictable performance. Included in the framework is an outline defining the process needed for setting the design criteria, validating the acceptability of the site, developing a design approach and an acceptability analysis that result in construction documents, provisions for needed quality assurance programs, and recognition of the need for maintenance of the building during its life.

The heart of this performance based engineering process resides with the design and acceptability analysis procedures. There are many potential approaches; some more ready for application than others. These possible design approaches are contained in Chapter 3 and the related acceptability procedures are defined in Chapter 4. Because of wide variety of building types and the uniqueness of their response to earthquakes, no one design approach or acceptability check is expected to be appropriate for all buildings. Multiple approaches need to be developed and refined for use on the subset of building types that are best represented by their inherent assumptions.

Conceptual Framework

Performance based seismic engineering encompasses the full scope of engineering tasks necessary to create structures with predictable seismic performance. This recommended scope of seismic engineering is significantly expanded from current practice to include selection of performance design objectives, seismic hazard analysis, seismic design and analysis for the structural and nonstructural components, plus design and construction quality assurance, structural maintenance after construction and monitoring of function and occupancy. The performance based engineering conceptual methodology includes:

> *Performance objectives*, which are selected and expressed in terms of expected levels of damage resulting from expected levels of ground motion.
>
> *Site suitability and seismic hazard analyses*, which are performed to identity site hazards that may render the site unsuitable for building and to identify and characterize specific levels of ground motion for design.

A *design approach,* which includes the conceptual, preliminary and final designs and acceptability analyses. In the *conceptual design,* the key design decisions are made regarding selection, layout, and configuration of the structural and nonstructural systems and designation of a system of ductile elements to control the inelastic response. The *preliminary design* establishes initial member sizes based on initial design criteria. The *final design* refines the sizes and completes the ductile detailing. *Acceptability analyses* are used after each stage of design to verify that the acceptability criteria defined by the performance objectives are met.

Quality assurance, which includes expanded design review and construction quality assurance, both of which are essential to performance based engineering. *Structural maintenance* which includes safe guarding against deterioration and alterations to the earthquake resisting systems that will reduce their effectiveness.

Performance based seismic engineering brings with it risk and uncertainty. Sources of uncertainty must be identified and the levels of risk or reliability must be quantified as part of the design guideline development.

Design Approaches

Several existing design approaches once modified and several new approaches still needing development are available for performance based design. The *comprehensive design, displacement based design* and *energy based design* approaches are potential design approaches needing development. Each promises to directly address the inelastic response of the structure and should be useable with any performance objectives.

The *general force/strength, simplified force/strength* and *prescriptive* approaches are modifications and enhancements of current practice using elastic design procedures. These approaches when used for performance based design should have restricted application and generally should not be developed for use on complex irregular buildings with enhanced performance objectives.

Acceptability Analysis Procedures

Acceptability analyses are performed at each design step to verify that the selected performance objectives are met. Acceptability criteria are set in terms of limiting values of structural response parameters (drift, stress, ductility demand, etc.) associated with the defining damage levels for each performance objective. The acceptability analysis may employ various elastic or inelastic analysis techniques to measure the structural seismic response in comparison with the limiting acceptability criteria.

The *elastic analysis procedures* presently in common use, i.e., the equivalent lateral force, response spectra and modal analysis procedures, have limited application in determining the inelastic response. The *component based elastic analysis* procedure, currently in use on a limited basis, uses inelastic demand ratios to gauge the ductility demand on structural elements.

The newest inelastic analysis procedures include the *capacity spectrum* and *pushover methods*. These methods analyze an incrementally loaded structure as it yields by developing a series of plastic hinges, until an estimated ultimate drift demand is reached. The *drift demand spectrum* method is a developing technique for analyzing the near field demands on structures.

Dynamic nonlinear time history analysis provides the most comprehensive analysis of the dynamic seismic response. This technique attempts to replicate and record the actual performance of a building under a design time history record.

Application

The conceptual framework outlined herein can be developed into engineering guidelines in the next phase of the performance based engineering initiative. This will be best done by expanding the scope of current practice to include the selection of performance objectives, validation of site suitability, seismic design of structural and nonstructural components, expanded quality assurance in both design and construction, and long-term maintenance of the systems.

As design approaches are developed, they should embrace capacity design principles in order to predictably control the inelastic behavior. Analysis procedures should directly address inelastic seismic response. In this regard, the displacement based design and energy based design approaches are particularly promising new approaches.

The process of transitioning from the current practice of single performance level, elastic based design to performance based engineering will only be successful if there is a smooth transition that is properly calibrated and slowly introduced into the current standards. The Vision 2000 committees work during 1994 produced three other reports aimed at these transitions. Part 1: Interim Recommendations, outlines a process for performance based engineering using currently available design guidelines and standards. Part 3: Preliminary Northridge Lessons, offers basic building performance data that will be useful in calibrating design approaches and acceptability checks to that experience. Part 4: Moving the Blue Book Toward Performance Based Engineering, defines a transition process for accomplishing the goal.

1. INTRODUCTION

1.1 Purpose

This report presents a conceptual framework and methodology for performance based engineering of earthquake resistant structures, as the first stage of a three-stage initiative to develop performance based seismic engineering provisions. It includes basic descriptions of several proposed design approaches and several acceptability analysis procedures, applicable for development of structures with predictable and controllable seismic performance, within definable levels of risk. This report was prepared by the Structural Engineers Association of California (SEAOC), Vision 2000 Committee, with funding provided by the California Office of Emergency Services.

It is intended that the framework presented will be developed into a seismic engineering guideline document and then into code provisions in subsequent stages. The report is intended to provide guidance to the guideline development and to accelerate the process of incorporating performance based engineering concepts into the current code seismic provisions as they are developed. The report may also be used to introduce the concepts of performance based engineering to practicing engineers, architects, owners and other interested members of the community.

1.2 Background

Seismic design practice has developed over the course of this century from rudimentary beginnings with simple design concepts and limited performance objectives. It evolved from its early stages in the first part of this century to the 1970s, essentially as a development and refinement of the equivalent lateral force concept in which an earthquake resistant structure was designed to resist a small fraction of its own weight acting as a static lateral force against a vertical frame. The ATC 3-06 project in the 1970s embraced a broad range of recent research and development advances and gave new direction to seismic engineering practice, taking account of the dynamic nature of seismic loading and response, the influence of soil-structure interaction, and the importance of ductile behavior and ductile detailing in absorbing the inelastic demands on the structure. The ATC 3-06 concepts have now been significantly incorporated into seismic design codes.

The result has been greatly improved seismic design in many respects. Yet, the 1994 Northridge earthquake demonstrated, as have other recent earthquakes, that room for improvement remains. As expected, many older structures were damaged, but also a number of newer concrete, steel, masonry and wood frame structures were severely damaged. The losses due to nonstructural damage and loss of service were significantly greater even than the losses due to structural damage.

The life-safety protection offered by the modern code and tested in Northridge and other recent earthquakes has been relatively good. Yet had the timing of the Northridge earthquake been different, the consequences would have been significantly worse. The property damage losses, while generally consistent with the implied performance objectives of the code, were significant and appear to be arguably greater than society expects in such an earthquake.

The performance of structures in recent earthquakes points to the need for improved seismic design approaches capable of achieving more favorable and predictable results. Furthermore, the evolution of the seismic codes from their beginnings until today, as they incorporate the ATC 3-06 concepts, has resulted in an increasingly cumbersome and entangled set of design requirements, obscuring the fundamental concepts of seismic design.

A clear need has arisen to both reformulate seismic design guidelines to enable the achievement of predictable seismic performance and to restate the guidelines so that the fundamental seismic design concepts are transparently clear. In response to that need, the Vision 2000 Performance Based Engineering development initiative has been launched.

1.3 The Goals of the Vision 2000 Initiative

The workplan for the Vision 2000 Initiative includes a concept phase, guideline phase and design provision phase of development. The short-term goal, to which this document contributes, is to develop a conceptual framework for performance based seismic engineering of structures, to be completed in 1995. The mid-term goals are to develop the conceptual framework into a guideline document for performance based seismic engineering and to begin incrementally incorporating the concepts into the existing seismic design codes by 1998 to 2000. The long-term goal is to draft the guidelines into code form and to complete the overhaul of seismic design practice by the year 2000 to 2005. These goals and timetables are admittedly ambitious, considering the recent history with ATC 3-06.

The short-term project has been divided into nine tasks, reported in four Parts. This document, Part 2, is the end work product of the Task A.2 Subcommittee. It describes the conceptual framework for Performance Based Engineering and describes design approaches and acceptability analysis procedures. Part 1 is the outgrowth of the work of the Task A.1 subcommittee which defines performance based engineering, performance levels and objectives and seismic hazard levels, and recommends interim procedures for applying existing design provisions to performance based engineering. The results of the other tasks are reported in Parts 3 and 4.

1.4. Performance Based Engineering Definition

Performance based engineering is defined as "consisting of the selection of design criteria, appropriate structural systems, layout, proportioning, and detailing for a structure and its nonstructural components and contents, and the assurance and control of construction quality and long-term maintenance, such that at specified levels of ground motion and with defined levels of reliability, the structure will not be damaged beyond certain limiting states or other usefulness limits".

The limiting damage states referred to are termed "performance levels"; four specific levels are defined as described in Section 2.2 of this report. The performance levels are coupled with specified levels of probable ground motion to define the "performance objectives" for which structures are designed. The achievement of performance objectives is never guaranteed but is expected, with defined levels of risk and reliability.

1.5 Performance Based Engineering Framework Document

This report presents the conceptual framework for Performance Based Engineering of earthquake resistant structures. Chapter 1 is this introductory Chapter which places the current phase of performance based engineering development into context and outlines the overall goals of the initiative project.

Chapter 2 presents an overview of the conceptual framework organized into a step-by-step methodology. The conceptual framework and methodology outlined apply to all types of structures, in all seismic regions with any performance objectives, and to both new and existing structures. The conceptual framework is structured to identify the full range of key engineering issues that must be addressed to achieve predictable seismic performance in earthquake resistant structures. The framework presented includes a design approach, several of the most promising are summarized in Chapter 3, and a design acceptability analysis procedure, several of which are summarized in Chapter 4.

Chapter 3 presents six possible design approaches that may become part of the overall methodology. Each of these six design approaches can be developed and used independently. Some apply to more types of structures and performance objectives than others.

Chapter 4 summarizes several promising procedures for performing acceptability analysis, which are used to verify that the structural design is accomplishing its performance objectives, within defined levels of reliability. The acceptability analysis procedures presented generally consist of analyzing the elastic and inelastic response of the structure and comparing various structural response parameters to target values of those parameters, linked to the performance objectives for the project.

Chapter 5 presents the conclusions regarding the performance based engineering concept and recommendations as to the direction that future research and development should take to accomplish the mid-term and long-term goals of this performance based engineering initiative.

The Appendices to this report contain more expansive discussion of the several design approaches and acceptability analysis methods presented, plus additional reference information. This information is compiled as a resource document for the future phases of the performance based engineering development initiative.

2. CONCEPTUAL FRAMEWORK FOR PERFORMANCE BASED ENGINEERING

2.1 Overview

A conceptual framework for performance based engineering is presented in this chapter in a generalized step-by-step methodology. The framework encompasses the full range of seismic engineering issues to be addressed in designing structures for predictable and controllable seismic performance, within established levels of risk. The methodology is illustrated in a flow chart in Figure 2-1.

The framework presented herein must be developed first into guidelines and then into code provisions before it can be readily applied in building design. Some of the concepts are easily developed; others will require considerable research and trial design applications before they can be developed into a useable form. In the transitional phase from current seismic design practice to performance based engineering design, it is anticipated that new provisions will be incrementally developed from the existing provisions to assure that a smooth transition is made.

The performance based engineering methodology will apply to all types of structures in all seismic regions. It has been developed in the context of general building design but may also be applied to bridges and other non-building structures. It is focused primarily on new construction but also applies to existing structures with some limitations. Application to existing buildings is also being developed in concurrent guideline development projects, ATC-33 and ATC-40/ Proposition 122.

Performance based engineering begins with the selection of performance objectives and identification of seismic hazards, continues with conceptual, preliminary and final designs, design acceptability checks and design review, and concludes with quality assurance during construction and building maintenance after construction. Each step is pursued to a greater or lesser extent, depending on the rigor of design required to meet the selected performance objectives. Abbreviated methodologies can be used for simple structures with modest performance objectives.

The "design" and "acceptability check" steps vary depending on the design approach selected and the performance objectives. Building design approaches are outlined in Chapter 3 and include Comprehensive Design, Displacement, Energy, General Force/Strength, Simplified Force/Strength, and Prescriptive Approaches. Acceptability analysis procedures are outlined in Chapter 4 and include the General Elastic Analysis Procedures, Component Based Elastic Analysis Procedure, Capacity Spectrum Procedure, Pushover Analysis Methods, Dynamic Nonlinear Time History Analysis and the Drift Demand Spectrum Analysis.

The capacity design philosophy is adopted as an underlying principle of appropriate seismic design. In capacity design, the inelastic behavior of the structure is controlled by proportioning the structure to yield at predetermined ductile "fuse" locations. Those "fuses" are detailed to accommodate the ductility demand imposed by the earthquake while the rest of the structure remains elastic.

The seismic design process involves multiple sources of uncertainty including the seismic hazard, the analysis methods and models, and the variability in the construction materials and workmanship. Performance based engineering techniques must identify and quantify those uncertainties so that the reliability of the design can be estimated and acknowledged.

Each major step in the Performance Based Engineering methodology is described in greater detail in the following subsections. These descriptions are general in nature and will be more fully developed in subsequent sections of the report. Each is written as a general description of the end product, with some reference to particular applications. Each will need to be fully developed in a guideline document in the next stage of performance based engineering development.

2.2 Selection of Performance Objectives

The first step in performance based engineering is selection of seismic performance objectives for the design. This selection is made by the client in consultation with the design professional based on consideration of the client's expectations, the seismic hazard exposure, economic analysis and acceptable risk. Performance objectives will range typically from code minimums, usually based on life-safety in a rare earthquake, to fully operational in a maximum capable earthquake.

A performance objective is a coupling of expected performance level with expected levels of seismic ground motions. A performance level is a damage state, a distinct band in the spectrum of possible seismic damage states as illustrated in Figure 2-2. Performance levels are defined in terms of damage to the structure and nonstructural components and in terms of consequences to the occupants and functions carried on within the facility. Four performance levels are identified in this report and are described in detail in Part 1 of this report. These performance levels are as follows:

- *Fully Operational* - Facility continues in operation with negligible damage
- *Operational* - Facility continues in operation with minor damage and minor disruption in non-essential services.
- *Life Safe* - Life-safety is substantially protected, damage is moderate to extensive.
- *Near Collapse* - Life-safety is at risk, damage is severe, structural collapse is prevented.

Tables 2-1 through 2-6 further define these performance levels in terms of the various components of the building.

The seismic hazard at a given site is represented as a set of earthquake ground motions and associated hazards with specified probabilities of occurrence. Four levels of probabilistic events are proposed as follows:

Event	Recurrence Interval	Probability of Exceedance
Frequent	43 years	50% in 30 years
Occasional	72 years	50% in 50 years
Rare	475 years	10% in 50 years
Very Rare	970 years	10% in 100 years

Performance objectives typically include multiple goals, for example, fully operational in the 43 year event, life-safety in the 475 year event and collapse prevention in the 970 year event. For this report, a set of Minimum Objectives and Enhanced Objectives are identified:

> *Minimum Objectives:* The Basic Objective is defined as the minimum acceptable performance objective for typical new buildings. Essential/ Hazardous Facility and Safety Critical Objectives are defined as minimum objectives for facilities such as hospitals and nuclear material processing facilities, respectively. These three Minimum Objectives are illustrated in Figure 2-3 as diagonal lines in the Performance Objective Matrix.
>
> *Enhanced Objectives:* Other objectives providing better performance or lower risk than the Minimum Objectives may be selected at the client's discretion. These objectives are termed Enhanced Objectives.

The selection of performance objectives sets the acceptability criteria for the design. The performance objectives represent performance levels, or damage levels, expected to result from design ground motions. The performance levels are keyed to limiting values of measurable structural response parameters, such as drift and ductility demand. When the performance objectives are selected, the associated limiting values become the acceptability criteria to be checked in later stages of the design. Limiting values of the response parameters which correlate with the defined performance levels must be established through research. Acceptability criteria are discussed further in Section 2.6.

2.3 Site Suitability and Design Ground Motions

Before structural design begins, a site suitability and seismic hazard analysis must be undertaken considering the proposed performance objectives for the project. The building site is analyzed for suitability for the proposed structure, considering any site hazards that may exist. The ground motion design criteria are established and characterized in a form suitable for the anticipated structural analysis and design methods.

Site suitability analysis includes consideration of the site seismicity, soil type and potential site hazards. Analysis will typically include determination of soil profile and topography, identification of seismic sources and source mechanisms, and identification of liquefaction potential, tsunami exposure and other hazards such as flooding and fire hazards from adjacent properties.

Seismic hazard analysis will determine the design ground motion for the specified design events considering all critical seismic sources. The ground motions are represented as time histories, acceleration response spectra, displacement response spectra, drift demand spectra, or by other means as required for the design and analysis methods. Response spectra may be elastic or inelastic as required for the design and analysis. Damping may be accounted for as 2% critical for purely elastic response, 5% critical for traditional elastic analysis assuming inelastic response, or possibly 10% to 20% critical for inelastic analysis or for specially damped systems. Duration effects and energy content of the expected ground motion must be further studied and appropriately accounted for in the design.

For simple, regularly configured buildings with modest performance objectives, this step will follow procedures similar to the current code. The site suitability analysis may be based on publicly available maps or commonly known information. The seismic hazard analysis may be nothing more than a seismic zone factor assigned by the building official.

2.4 Conceptual Design

Once the performance objectives are selected and the site suitability and seismic ground motions are established, the structural design can proceed. The structural design begins with overall conceptual design of the facility including layout, configuration, selection of structural systems and materials, selection of foundation systems and, to varying degrees, selection of nonstructural systems. It is in the conceptual design stage that fundamental decisions are made that determine the ultimate viability of the design.

Modern seismic resistant design is founded upon reliance on the inelastic response of the structure to dissipate most of the input energy of the earthquake; therefore, it is imperative that the post-elastic behavior of the structure be addressed at this stage of the design. Capacity design principles should be applied to provide an overview of the inelastic response and to designate the ductile links or "fuses" in the lateral force resisting system. The designated "fuses" will be counted upon to yield and to dissipate the input energy of the earthquake while the rest of the system remains elastic. The conceptual overview obtained provides the design team with a clear conceptual understanding of the intended inelastic response of the structure and focuses the design and quality assurance programs on the critical links in the system. At the final design and detailing stage, these critical links will be rigorously detailed to provide the ductility required.

In making the fundamental design decisions at the conceptual design stage, the following guidelines to good seismic design should be considered:

- Use simple, symmetrical and regular configurations where possible.
- Use lighter building weight rather than heavier.
- Avoid high slenderness ratios.
- Provide redundancy and ductility to overcome seismic design uncertainties.

- Provide sufficient stiffness to limit drift related damage.
- Provide sufficient flexibility to limit acceleration related damage.
- Provide toughness (energy dissipation capacity) and stability in strength and stiffness in post-elastic cyclic behavior.
- Provide uniform and continuous distribution of strength, stiffness, ductility and toughness.
- Provide structural strength and stiffness compatible with the foundations and soils type.
- Use relatively short spans and close column spacings.
- Tie vertical elements together at each level including the foundation.
- Use capacity design principles to control the inelastic behavior by identifying and providing a system of ductile links to absorb the inelastic response.
- Consider use of energy dissipation devices as a design strategy.
- Consider use of seismic isolation devices as a design strategy.

Since conceptual design is closely tied to the ultimate performance of a building, guidelines specifying appropriate limitations on the materials, configuration, and structural system are needed for each performance objective. These must be defined in terms that are usable at the conceptual design stage. The level of restriction should increase in severity with the increase in performance objective and should reflect the excellent historic performance of regularly configured structural systems composed of well detailed ductile materials.

2.5 Preliminary and Final Design Steps

With the performance objective selected, the ground motion for critical earthquakes established and the conceptual design resolved, the preliminary and final designs can proceed. The design procedures will differ depending upon the performance objectives and the design and analysis approaches taken, however the basic steps and issues considered are similar. Chapters 3 and 4 outline several of the possible design and analysis approaches.

A major challenge to performance based engineering is to develop simple yet effective methods for designing, analyzing and checking the design of structures so that they reliably meet the selected performance objectives. The possible approaches to the design problem outlined in this report include familiar force/strength approaches, displacement approaches, energy approaches and prescriptive design approaches plus various elastic and inelastic analysis procedures to be used for acceptability checks.

The preliminary and final design stages involve sizing and detailing the structural framing system and the nonstructural systems so that the performance objectives can be met. The performance objectives must be translated into engineering terms as acceptability criteria - structural response parameters tied to expected damage states. The acceptability criteria, limiting values in the response parameters, become targets for the design. They include drift and deformation limits, acceleration and force limits, yield limits, ductility limits and energy dissipation that must be met in order for the structural response to be consistent with the performance objectives. These acceptability criteria are discussed in Section 2.6.

In the preliminary design, the structural framing elements are sized and checked against selected criteria. The sizing will be accomplished using a systematic design approach, possibly one of those outlined in Chapter 3. This involves designing to meet two criteria, one for functionality and one for life-safety or collapse prevention. As part of the preliminary element sizing, the elements should be proportioned to permit the expected post-elastic yield patterns to be consistent with the capacity based conceptual design.

In the final design and detailing of the structure, all selected performance goals must be considered. As appropriate, both the elastic and inelastic responses should be considered and compared to the limiting response parameters.

For capacity designed structures, the ductile link regions should be detailed to assure acceptable inelastic behavior. The details must provide suitable ductility to extend the structural response from the elastic limit to the inelastic limit imposed by the design ground motions. The rest of the structure will be designed to remain elastic while the ductile links develop their inelastic capacity.

The nonstructural elements and attachments should be designed to accommodate the elastic and inelastic deformations of the structure. The nonstructural elements may be designed to be isolated from the structural system or to be integral parts, participating in the building's inelastic response. In either case, these elements must be designed and checked to meet the performance objectives.

2.6 Acceptability Checks at Each Design Step

At each stage of the design, an acceptability check is performed to verify that the selected performance objectives are being met. The specific extent and methodology of the check will vary with the performance objectives and the design approach used. However, the general concept of the acceptability checks remains the same.

The structural response as measured by certain quantifiable parameters must be consistent with the performance objectives and associated acceptability criteria. The acceptability criteria consist of limiting values in structural response parameters, associated with selected performance levels or damage levels for specified levels of ground motion. Within a particular building, the design of specific components may be controlled by the same or different response parameters for the same or different performance objectives. Typical response parameters checked may include:

- stress ratios
- drift and deformation ratios
- structural accelerations
- ductility demand ratios
- energy dissipation demand vs. capacity

Typical limiting values for these response parameters must be established for each performance level through research which will include laboratory testing of specific components, as well as calibrating the limiting values by analyzing buildings that have experienced measurable damage in past earthquakes and for which strong motion records are available. Then, in a specific design, the appropriate parameters must be checked at the governing performance levels. Typically, the design will at least be checked at the Fully Operational level and at the Life Safe level, after both the preliminary and final design. Some structures, such as those in areas of infrequent seismicity, may require design checks at the near Collapse level as well. All selected objectives will typically be checked at the final design stage. The checks will include both the structural and the nonstructural components. In many cases, nonstructural components can be pre-qualified by being prescriptively designed to meet the target parameters of the structural design.

For certain simple or prescriptive design approaches, no formal acceptability checks other than plan review by the building official may be performed. In such cases, acceptability checks will need to be incorporated in the basic design requirements.

Acceptability check procedures will involve both elastic and inelastic analysis methods. Elastic analysis procedures for checking stress ratios and drift are currently well known and widely used. Several simplified inelastic approaches, such as the pushover analysis, are outlined in Chapter 4.

2.7 Design Review

An important quality assurance step in the design process is competent independent design review. This design review includes both independent peer review and plan review by the building official.

Schools and hospitals in California are designed to very similar standards as other buildings in California, yet they tend to perform much better in earthquakes, largely because of the rigor of the plan review during design and the quality assurance provided during construction.

For simpler buildings and for prescriptive designs, the building official will provide the only independent review of the design. It is very important, however, that the building official understand the intent of the design and be able to provide a thorough and competent review of the plans.

For all other buildings, including specialized, irregular or important structures, independent peer review should be required. Such review should be undertaken at the end of the conceptual design and final design stages in order to provide independent professional assessment of the design, the assumptions, modeling, analysis, and effectiveness of the design in meeting the performance goals.

2.8 Quality Assurance During Construction

Regardless of the quality of the design and sophistication of the analysis, performance based engineering will not be successful without adequate quality assurance during construction.

The quality assurance process involves a team of players, including the design professional, peer reviewer, building official, special inspectors, testing agencies and the contractor. The design professional must be involved to assure that the design intent is properly interpreted and that the critical elements in the building system are recognized, properly constructed, and properly inspected and tested. The building official is legally involved in all projects and must verify that the project is constructed in accordance with the building codes and that the specified inspection, testing and structural observation are completed.

The special inspector must, at the direction of the design professional, inspect the critical elements of the system and must assure that those elements are constructed in strict compliance with the plans. The testing agency must verify the quality of the construction materials and the qualifications of the special inspector.

The scope of the quality assurance program may be limited in simple structures with modest performance objectives or in prescriptively designed structures. The quality assurance program must be rigorous for complex structures with demanding performance objectives. In each case, the quality assurance program should be defined by the Structural Engineer of Record.

2.9 Building Maintenance and Function

Performance based engineering does not end with the completion of construction. The responsibilities merely shift. The building condition, configuration and use will evolve over the life of the structure. The owner and the building official must remain vigilant to assure that the earthquake resistant system is not adversely altered through future remodels and renovations. Further, the owner must assure that the structure is maintained, that elements are not allowed to corrode or deteriorate and that change of use does not result in higher static live loads such as storage inventory that might significantly change the mass and thereby the dynamic response of the building without proper analysis and validation.

Maintenance requirements will vary with the type of construction and the selected performance objectives. Seismic designs using sophisticated technologies such as active or passive energy dissipation devices or relatively degradable materials must be properly maintained on a regular basis to perform as expected. Prescriptive designs using traditional building materials may have lower maintenance requirements.

2.10 Uncertainty, Risk and Reliability

Performance based engineering yields structures with predictable performance within defined levels of risk and reliability. There are multiple sources of uncertainty in the seismic design process. There is uncertainty regarding the seismic ground motions, the design and analysis techniques and modeling assumptions, the variability in the materials, workmanship and construction quality, and the changes to the structure over its life due to material deterioration, wear and structural or nonstructural modifications.

It is an important task of performance based engineering to recognize, identify and quantify those uncertainties so that the levels of reliability and risk can be established and acknowledged by the design professional, the client, the legal profession and the public.

Performance based engineering will, of necessity, start with a set of seismic design guidelines, incrementally improved over present code, and with modest claims of reliability in performance prediction. As new design and analysis techniques are developed and can be proven accurate in prediction of the seismic performance observed in recorded earthquakes, the design guidelines will be modified and the reliability will be progressively improved.

2.11 Research Needs

The development of this conceptual framework into a guideline for performance based engineering will require considerable new basic research, applied research, calibration studies, trial designs and actual guideline preparation. Each facet must be embraced and supported in a balanced manner if the goal is to be achieved.

In general, research related to this conceptual framework is needed in the following general areas:

- Validate the appropriateness of the four performance levels and the specific parameters used to define their minimum performance states.
- Validate the appropriateness of the four seismic hazard definitions through consideration of the social, economic and political impact seismic design and seismic performance has on communities in various seismic regions.
- Establish appropriate acceptance criteria for site performance in terms of permissible settlement, lateral spreading, liquefaction, faulting, etc. for each performance objective.
- Identify appropriate ground motion parameters needed for the possible design approaches and analysis checks that yield the best and most appropriate characterization of the seismic hazard.

- Develop guidelines for conceptual design that address all facets of performance based engineering from the project start perspective.

- Identify and develop appropriate design approaches and analytical checks that are fully compatible with the entire structural/earthquake engineering design effort and within bounds of the project funding.

- Identify the key characteristic of a proper design review based on current and successful efforts and develop appropriate guidelines. Determine if there is a need to augment and/or change the basic functions of the traditional building department as they relate to achieving the various performance objectives.

- Based on the performance of buildings in past earthquakes, determine the significance that construction quality has on performance. Establish guidelines for appropriate construction quality assurance programs for buildings built to each of the performance objectives.

- Develop code provisions that will provide for the maintenance of a buildings structural system for the life of the building.

- Develop techniques for systematically defining the levels of risk and reliability inherent in the guidelines and provisions written for performance based engineering. Develop an acceptability criteria for each and monitor the performance of buildings designed under the guidelines to assure that they are yielding structures of predictable performance within defined levels of risk and reliability.

These research recommendations are somewhat general and only lend an indication to the level of research and development that is needed in each of the key areas of performance based engineering. In each case, it is important that sufficient work is done to verify the accuracy of the techniques purposed, their compatibility with the overall conceptual framework as defined, and their consistency with the historic performance of buildings in past earthquakes.

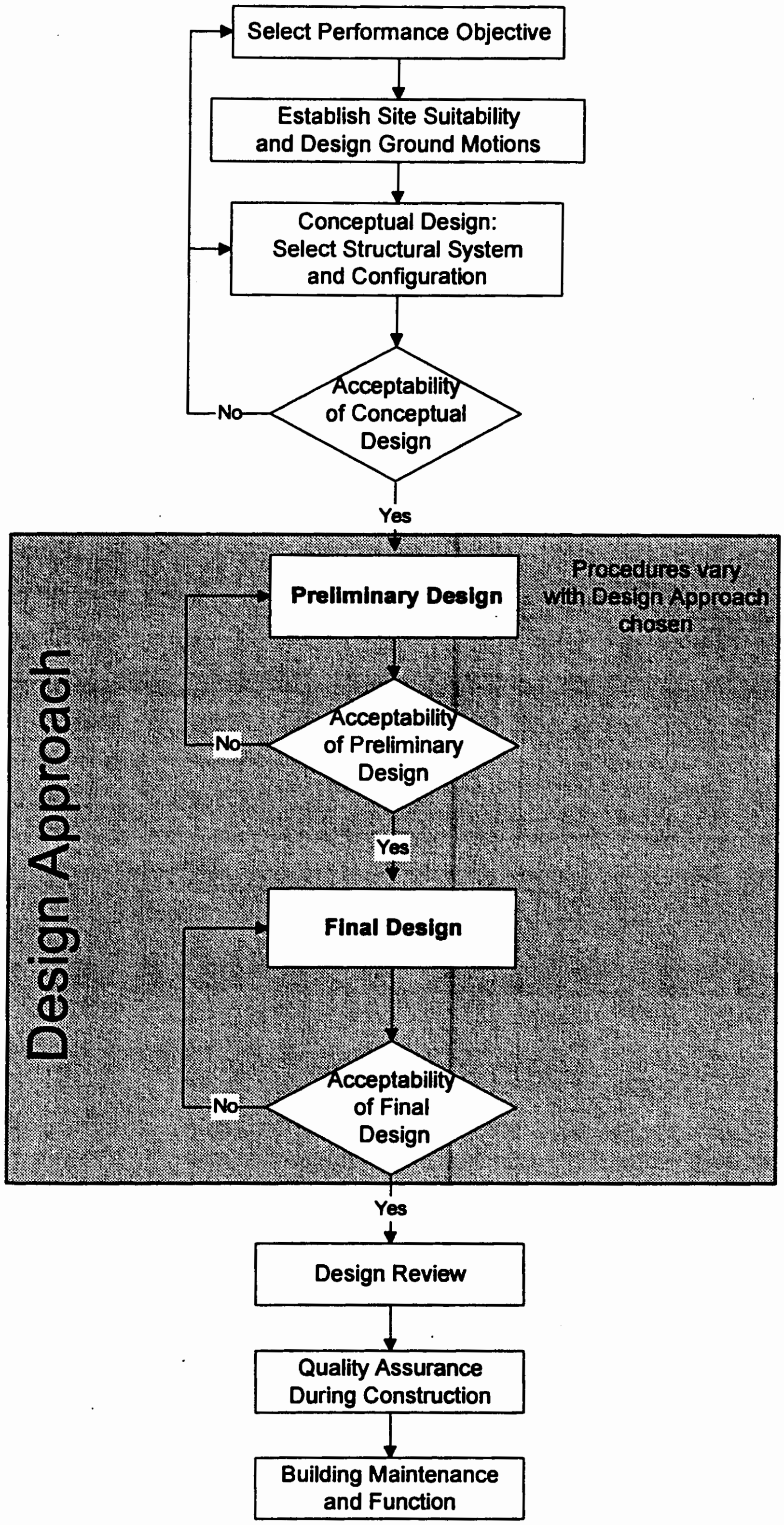

Figure 2-1: Methodology for Performance Based Engineering

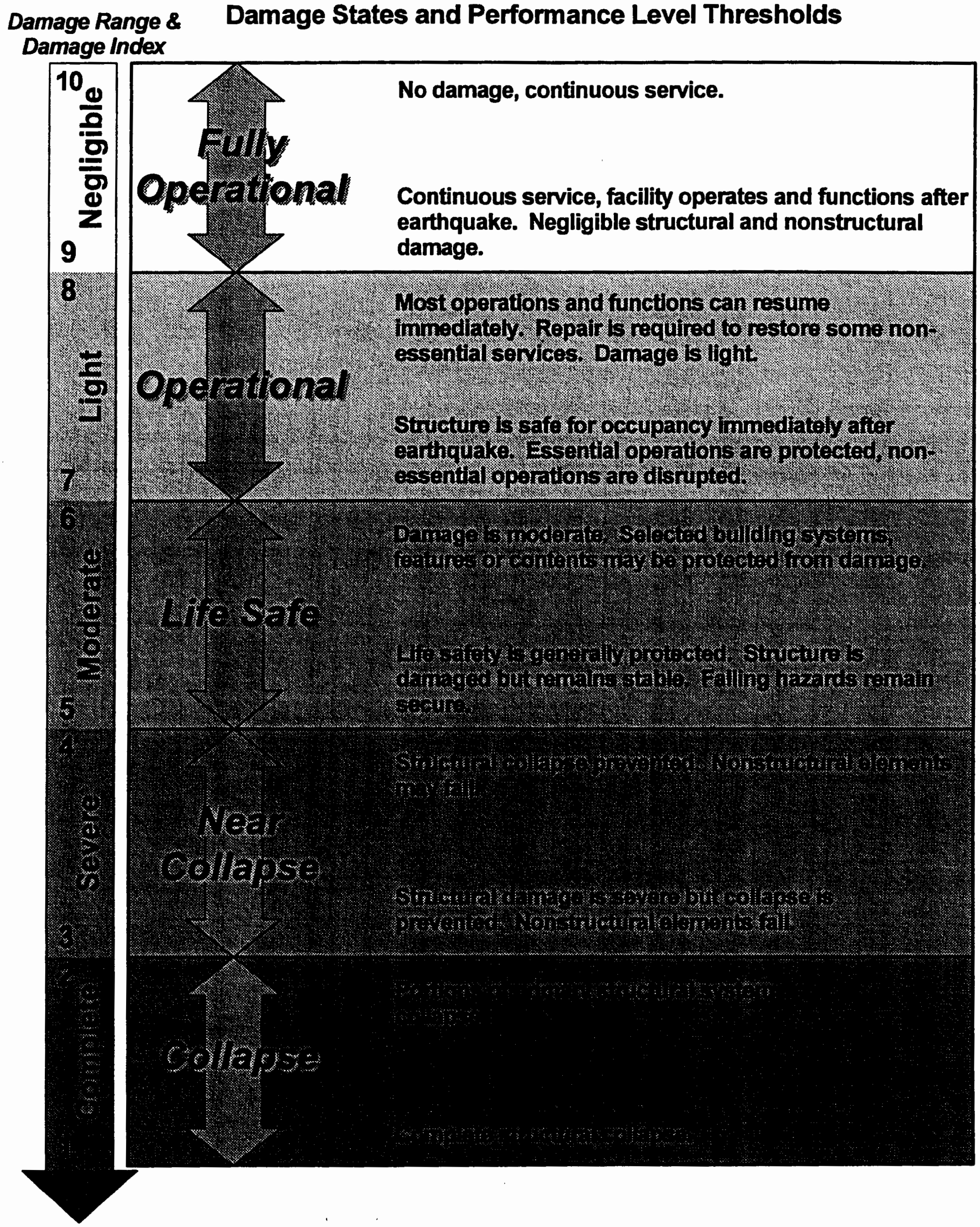

Figure 2-2: Spectrum of Seismic Damage States

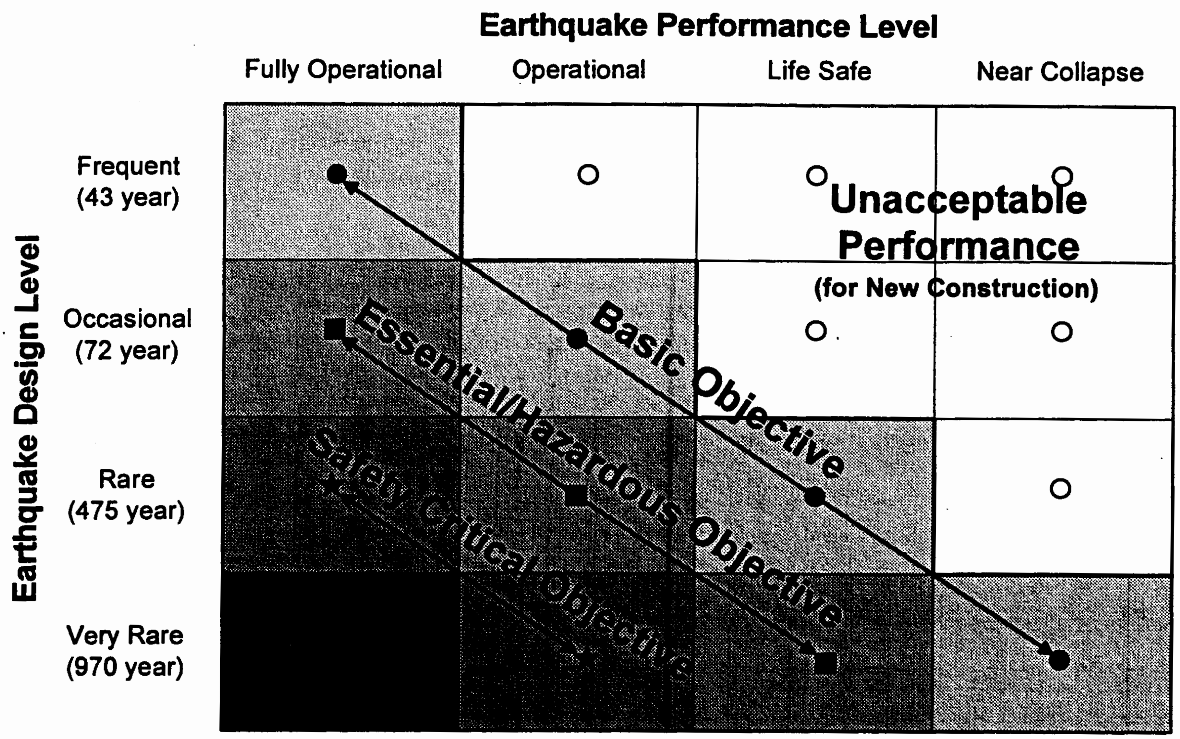

Figure 2-3: Recommended Seismic Performance Objectives for Buildings

System Description	Performance Level				
	10 Fully Operational 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3	2 Collapse 1
Overall building damage	Negligible	Light	Moderate	Severe	Complete
Permissible transient drift	< 0.2%+/-	< 0.5%+/-	< 1.5%+/-	< 2.5%+/-	> 2.5%+/-
Permissible permanent drift	Negligible.	Negligible.	< 0.5%+/-	< 2.5%+/-	> 2.5%+/-
Vertical load carrying element damage	Negligible.	Negligible.	Light to moderate, but substantial capacity remains to carry gravity loads.	Moderate to heavy, but elements continue to support gravity loads.	Partial to total loss of gravity load support.
Lateral Load Carrying Element damage	Negligible - generally elastic response; no significant loss of strength or stiffness.	Light - nearly elastic response; original strength and stiffness substantially retained. Minor cracking/yielding of structural elements; repair implemented at convenience.	Moderate - reduced residual strength and stiffness but lateral system remains functional.	Negligible residual strength and stiffness. No story collapse mechanisms but large permanent drifts. Secondary structural elements may completely fail.	Partial or total collapse. Primary elements may require demolition.
Damage to architectural systems	Negligible damage to cladding, glazing, partitions, ceilings, finishes, etc. Isolated elements may require repair at users convenience.	Light to moderate damage to architectural systems. Essential and select protected items undamaged. Hazardous materials contained.	Moderate to severe damage to architectural systems, but large falling hazards not created. Major spills of hazardous materials contained.	Severe damage to architectural systems. Some elements may dislodge and fall.	Highly dangerous falling hazards. Destruction of components.
Egress systems	Not impaired.	No major obstructions in exit corridors. Elevators can be restarted perhaps following minor servicing.	No major obstructions in exit corridors. Elevators may be out of service for an extended period.	Egress may be obstructed.	Egress may be highly or completely obstructed.

Table 2-1a: General Damage Descriptions by Performance Levels and Systems

System Description	Performance Level				
	Fully Operational	Operational	Life Safe	Near Collapse	Collapse
Mechanical/Electrical/ Plumbing/Utility Systems	Functional.	Equipment essential to function and fire/life safety systems operate. Other systems may require repair. Temporary utility service provided as required..	Some equipment dislodged or overturned. Many systems not functional. Piping, conduit ruptured.	Severe damage and permanent disruption of systems.	Partial or total destruction of systems. Permanent disruption of systems.
Damage to contents	Some light damage to contents may occur. Hazardous materials secured and undamaged.	Light to moderate damage. Critical contents and hazardous materials secured.	Moderate to severe damage to contents. Major spills of hazardous materials contained.	Severe damage to contents. Hazardous materials may not be contained.	Partial or total loss of contents.
Repair	Not required.	At owners/tenants convenience.	Possible - building may be closed.	Probably not practical.	Not possible.
Effect on occupancy	No effect.	Continuous occupancy possible.	Short term to indefinite loss of use.	Potential permanent loss of use.	Permanent loss of use.

Table 2-1b: General Damage Descriptions by Performance Levels and Systems

Elements	Type	Performance Level			
		10 Fully Functional 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Concrete frames	Primary	Negligible.	Minor hairline cracking (0.02"); limited yielding possible at a few locations; no crushing (strains below 0.003).	Extensive damage to beams; Spalling of cover and shear cracking (< 1/8") for ductile columns. Minor spalling in non-ductile columns. Joints cracked < 1/8" width.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some non-ductile columns. Severe damage in short columns.
	Secondary	Negligible.	Same as primary.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some non-ductile columns. Severe damage in short columns.	Extensive spalling in columns (possible shortening) and beams. Severe joint damage. Some reinforcing buckled.
Steel moment frames	Primary	Negligible.	Minor local yielding at a few places. No observable fractures. Minor buckling or observable permanent distortion of members.	Hinges form; local buckling of some beam elements; severe joint distortion. Isolated connection failures. A few elements may experience fracture.	Extensive distortion of beams and column panels. Many fractures at connections.
	Secondary	Negligible.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.	Extensive distortion of beams and column panels. Many fractures at connections.	Extensive distortion of beams and column panels. Many fractures at connections.
Braced steel frames	Primary	Negligible.	Minor yielding or buckling of braces. No out-of-plane distortions.	Many braces yield or buckle but do not totally fail; Many connections may fail.	Extensive yielding and buckling of braces. Many braces and their connections may fail.
	Secondary	Negligible.	Same as primary.	Same as primary.	Same as primary.

Table 2-2a: Performance Levels and Permissible Structural Damage - Vertical Elements

Elements	Type	Performance Level			
		10 Fully Functional 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Concrete shear walls	Primary	Negligible.	Minor hairline cracking (0.02") of walls. Coupling beams experience cracking < 1/8" width.	Some boundary element distress including limited bar buckling; Some sliding at joints; damage around openings; some crushing and flexural cracking; Coupling beams - extensive shear and flexural cracks; some crushing, but concrete generally remains in place	Major flexural and shear cracks and voids; sliding at joints; extensive crushing and buckling of rebar; failure around openings; severe boundary element damage; Coupling beams shattered, virtually disintegrated
	Secondary	Negligible.	Minor hairline cracking of walls, some evidence of sliding at construction joints, Coupling beams experience cracks < 1/8" width, minor spalling.	Major flexural and shear cracks; sliding at joints; extensive crushing; failure around openings; severe boundary element damage; Coupling beams shattered, virtually disintegrated.	Panels shattered virtually disintegrated.
Unreinforced masonry infill walls	Primary	Negligible.	Minor cracking (< 1/8") of masonry infills and veneers. Minor spalling in veneers at a few corner openings.	Extensive cracking and some crushing but wall remains in place; no falling units; Extensive crushing and spalling of veneers at corners of openings.	Extensive cracking and crushing; portions of face course shed.
	Secondary	Negligible.	Same as primary.	Same as primary.	Extensive cracking and crushing; portions of face course shed.

Table 2-2b: Performance Levels and Permissible Structural Damage - Vertical Elements

Elements	Type	Performance Level			
		10 Fully Functional 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
URM bearing walls	Primary	Negligible.	Minor cracking (< 1/8") of masonry infills and veneers. Minor spalling in veneers at a few corner openings. No observable out-of-plane offsets.	Extensive cracking, noticeable in-plane offsets of masonry and minor out-of-plane offsets.	Extensive cracking. Face course and veneer may peel off. Noticeable in-plane and out-of-plane offsets.
	Secondary	Negligible.	Same as primary.	Same as primary.	Same as primary.
Reinforced masonry walls	Primary	Negligible.	Minor cracking (< 1/8"). No out-of-plane offsets.	Extensive cracking (< 1/4"), distributed throughout wall. Some isolated crushing.	Crushing; extensive cracking; damage around openings and at corners; some fallen units.
	Secondary	Negligible.	Same as primary.	Crushing; extensive cracking; damage around openings and at corners; some fallen units.	Panels shattered virtually disintegrated.
Wood stud walls	Primary	Negligible.	Distributed minor hairline cracking of gypsum and plaster veneers.	Moderate loosening of connections and minor splitting of members.	Connections loose, nails partially withdrawn, some splitting of members and panel; veneers shear off.
	Secondary	Negligible.	Same as primary.	Connections loose, nails partially withdrawn, some splitting of members and panel.	Sheathing shears off, let-in braces fracture and buckle, framing split and fractured.
Foundations	General	Negligible.	Minor settlement and negligible tilting.	Total settlements < 6 inches and differential settlements < 1/2 inch in 30 feet.	Major settlements, and tilting.

Table 2-2c: Performance Levels and Permissible Structural Damage - Vertical Elements

System	Performance Levels			
	10 Fully Functional 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Metal deck diaphragms	Negligible.	Connections between deck units and from deck to framing intact. Minor distortions.	Some localized failure of welded connections of deck to framing and between panels; minor local buckling of deck.	Large distortion with buckling of some units and tearing of many welds and seam attachments.
Wood diaphragms	Negligible.	No observable loosening, withdrawal of fasteners, or splitting of sheathing or framing.	Some splitting at connections, loosening of sheathing, observable withdrawal of fasteners, splitting of framing and sheathing.	Large permanent distortion with partial withdrawal of nails and splitting of elements.
Concrete diaphragms	Negligible.	Distributed hairline cracking and a few minor cracks (< 1/8") of larger size.	Extensive cracking (< 1/4") and local crushing and spalling.	Extensive crushing and observable offset across many cracks.

Table 2-3: Performance Levels and Permissible Structural Damage - Horizontal Elements

Element	Performance Level			
	10 Fully Operational 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Cladding	Negligible damage.	Connections yield; some cracks or bending in cladding.	Severe distortion in connections; distributed cracking, bending, crushing and spalling of cladding elements; some fracturing of cladding; falling of panels prevented.	Severe damage to connections and cladding; some falling of panels.
Glazing	Generally no damage; isolated cracking possible.	Some broken glass; falling hazards avoided.	Extensive broken glass; some falling hazard.	General shattered glass and distorted frames; widespread falling hazards.
Partitions	Negligible damage; some hairline cracks at openings.	Cracking to about 1/16" at openings; crushing and cracking at corners.	Distributed damage; some severe cracking, crushing and wracking in some areas.	Severe wracking and damage in many areas.
Ceilings	Generally negligible damage; isolated suspended panel dislocations or cracks in hard ceilings.	Minor damage; some suspended ceilings disrupted, panels dropped; minor cracking in hard ceilings.	Extensive damage; dropped suspended ceilings, distributed cracking in hard ceilings.	Most ceilings damaged; most suspended ceilings dropped, severe cracking in hard ceilings.
Light fixtures	Negligible damage; pendant fixtures sway.	Minor damage; some pendant lights broken; falling hazards prevented.	Many broken light fixtures; falling hazards generally avoided in heavier fixtures (> 20 lbs +/-).	Extensive damage; falling hazards occur.
Doors	Negligible damage.	Minor damage.	Distributed damage; some racked and jammed doors.	Distributed damage; many racked and jammed doors.
Elevators	Elevators operational with isolated exceptions.	Elevators generally operational; most can be restarted.	Some elevators out of service.	Many elevators out of service.

Table 2-4: Performance Levels and Permissible Damage - Architectural Elements

Element	Performance Level			
	10 Fully Operational 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Mechanical equipment	Negligible damage, all remain in service.	Minor damage. Some units, not essential to function out-of-service.	Many units non-operational; some slide or overturn.	Most units non-operational; many slide or overturn; some pendant units fall.
Ducts	Negligible damage.	Minor damage, but systems remain in service..	Some ducts rupture; some supports fail, but ducts do not fall.	Most systems out of commission; some ducts fall.
Piping	Negligible damage.	Minor damage. Minor leaking may occur.	Some pipes rupture at connections; many supports fail; few fire sprinkler heads fail.	Many pipes rupture; supports fail; some piping systems collapse.
Fire alarm systems	Functional.	Functional.	Not functional.	Not functional.
Emergency lighting systems	Functional.	Functional.	Functional.	Not functional.
Electrical equipment	Negligible damage.	Minor damage; panels restrained, isolated loss of function in secondary systems.	Moderate damage; panels restrained from overturning; some loss of function and service in primary systems.	Extensive damage and loss of service.

Table 2-5: Performance Levels and Permissible Damage - Mechanical/Electrical/Plumbing Systems

Element	Performance Level			
	10 Fully Operational 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Furniture	Negligible effects.	Minor damage; some sliding and overturning.	Extensive damage from sliding, overturning, leaks, falling debris etc.	Extensive damage from sliding, overturning, leaks, falling debris etc.
Office equipment	Negligible effects.	Minor damage; some sliding and overturning.	Extensive damage from sliding, overturning, leaks, falling debris etc.	Extensive damage from sliding, overturning, leaks, falling debris etc.
Computer systems	Operational.	Minor damage; some sliding and overturning. Mostly functional.	Extensive damage from sliding, overturning, leaks, falling debris etc.	Extensive damage from sliding, overturning, leaks, falling debris etc.
File cabinets	Negligible damage.	Minor damage; some sliding and overturning.	Extensive damage from sliding, overturning, leaks, falling debris etc.	Extensive damage from sliding, overturning, leaks, falling debris etc.
Bookshelves	Negligible damage.	Minor damage; some overturning and spilling.	Extensive damage from leaks, falling debris, overturning etc.	Extensive damage form leaks, falling debris, overturning etc.
Storage racks & cabinets	Negligible damage; overturning restrained.	Moderate damage; overturning restrained; some spilling.	Extensive damage from leaks, falling debris, overturning, spilling etc.	Extensive damage form leaks, falling debris, overturning, spilling etc.
Art works, collections	Minor damage; overturning restrained.	Moderate damage; overturning restrained; some falling.	Extensive damage from leaks, falling debris, overturning, spilling etc.	Extensive damage form leaks, falling debris, overturning, spilling etc.
Hazardous materials	Negligible damage; overturning and spillage restrained.	Negligible damage; overturning and spillage restrained.	Minor damage; overturning and spillage generally restrained.	Severe damage; some hazardous materials released.

Table 2-6: Performance Levels and Permissible Damage - Contents

3. DESIGN APPROACHES FOR PERFORMANCE BASED ENGINEERING

3.1 Overview

A. *Selection of Design Approach*

The performance based engineering framework and methodology encompass the full range of engineering activities necessary to create structures with predictable seismic performance. Structural design consists of a sequence of steps within the overall engineering methodology. Before design can begin, a specific design approach must be selected to accomplish the specific performance objectives. There are several existing seismic design approaches which can be modified and others in development that can be applied to performance based design. The generalized methodology can be tailored to each of these approaches to enable the design of structures to meet any selected performance criteria.

Performance based design consists of the conceptual, preliminary and final design processes, as outlined in Chapter 2. It includes the selection of the building configuration, structural system, member sizing and detailing and, to varying degrees, the nonstructural components. The design process is complimented by the acceptability analysis. The design approach and the acceptability analysis methods are selected largely independently, but are combined within the context of performance based engineering to provide the methods to achieve the performance objectives.

Several possible design approaches are summarized in this chapter including the Comprehensive Design, Displacement, Energy, General Force/Strength, Simplified Force/Strength and Prescriptive Approaches. More detailed discussions of the approaches can be found in the Appendices to this report and in the documents referenced. These approaches vary in complexity and applicability. The Comprehensive, Displacement, Energy and General Force/Strength Approaches are the more rigorous and exhaustive approaches for application to all types of projects including the most complex and irregular structures. The Simplified Force/Strength and Prescriptive Approaches are simpler, more familiar methods with more limited application. The last two are each modifications of current seismic design practice. These possible design approaches do not comprise an exhaustive list, but represent some of the more promising general design approaches.

Each of the design approaches developed should be adapted to reflect the capacity design philosophy. Capacity design principles come into play particularly at the conceptual design and final design stages, as discussed in Chapter 2.

The selection of an appropriate design approach for a given project depends on a number of factors including the performance objectives and the type and complexity of structural system to be used. In general, simpler buildings with more modest performance criteria can be designed using simpler approaches. More complex structures with more demanding performance objectives will generally require more rigorous design approaches. The relationship of design approach to performance objectives, structural systems and acceptability checks is discussed further in the following sections.

B. *Performance Objectives and Selection of Design Approaches*

The selection of a design approach must be consistent with the performance objective chosen for a project. If the performance criteria demands a multi-level design and tight control of seismic response, the design approach must be capable of meeting that demand.

The Prescriptive and the Simplified Force/Strength Approaches are the simplest of those outlined in this chapter and for many buildings, will be capable of meeting the Basic Objective, and for some buildings, the Essential/Hazardous Objective.

For other more complex buildings and more demanding performance objectives such as the Essential/Hazardous, Safety Critical and Enhanced Objectives, the more sophisticated design approaches including the General Force/Strength, the Comprehensive Design, Energy and Displacement Approaches may need to be used. The possible applicability of the various design approaches to performance objectives is illustrated in Figure 3-1.

C. *Structural Systems and Selection of Design Approaches*

The selection of a design approach must also be consistent with the type and complexity of structural system. The Prescriptive Approach will likely only apply to simple, regular structures constructed of light weight framing systems such as wood frame and steel stud frame, similar to existing code's conventional construction provisions. Complex, irregular structural systems may require more rigorous design approaches such as the Comprehensive or the General Force/Strength Approaches.

D. *Design Approaches and Acceptability Analysis Procedures*

Each of the design approaches must include acceptability analyses to verify that the acceptably criteria are met and that the design as measured by critical response parameters is meeting the performance objectives. The Prescriptive Design Approach is an exception in that the response parameters are not checked for acceptability. The prescriptive requirements themselves must be checked for acceptability at the time the requirements are developed, but the individual prescriptively designed project is not checked other than for code compliance by the building official.

Other design approaches must include formal acceptability checks for the critical response parameters. These acceptability analyses will typically include elastic stress and deformation checks at the Fully Operational performance level and may include inelastic checks considering inelastic behavior at other performance levels. Simpler design approaches, such as the Simplified Force/Strength Approach, may include elastic checks only, with conservative extrapolations to estimate the inelastic response. Acceptability analysis methods are discussed in more detail in Chapter 4.

3.2 Comprehensive Design Approach

A. *Overview*

The Comprehensive Design Approach is, as the name implies, the most comprehensive seismic design approach of those outlined in this report. It is a probabilistic limit state design approach in which optimization of life cycle costs is considered. It is developed from the concept that all buildings as a minimum should meet the Basic Objective and beyond that, the ideal seismic design is based on the least total cost over the life of the facility, including both initial costs and seismic damage repair costs. It therefore considers in the design, and strives to minimize, the damage consequences of each level of earthquake ground motion. It considers each of the four performance level limit states and each of the four probabilistic levels of ground motion identified in the conceptual framework outlined in Chapter 2.

This approach is based on the comprehensive analysis of the building system. It takes into account the simultaneous demands for strength and deformation and their combined effect on the energy demand and capacity for the entire building system, including structural and nonstructural components.

Although the Comprehensive Design Approach can be used for all types of buildings and for any performance objective, its primary use will likely be to calibrate simpler more practical design methods. In practice, it may also be used for special cases involving irregular, complex or safety critical structures.

B. *Methodology*

The Comprehensive Design methodology parallels the general performance based engineering methodology. It includes detailed study at each step in the flow chart shown in Figure 3-2 and highlighted below:

> <u>Selection of Performance Criteria:</u> Comprehensive design can be used with any performance objective though it will most likely be considered only with Essential/Hazardous, Safety Critical or Enhanced Objectives.
>
> <u>Site Suitability Analysis and Design Ground Motions:</u> Complete site analysis is conducted to identify site hazards and suitability for the proposed project. Time histories, elastic response spectra and inelastic response spectra and cumulative damage factors are determined, considering energy content of ground motion which includes the effects of duration.
>
> <u>Conceptual Design and Acceptability Check:</u> The selection of structural systems, structural layout and configuration, and selection and layout of nonstructural systems is performed with careful consideration of the seismic

design guidelines listed in Section 2.4. Particular consideration is given to energy concepts and the energy balance equation:

$$E_{input}=E_{elastic} + E_{dissipated}=(E_{kinetic} + E_{strain})_{elastic} + (E_{damping} + E_{plastic})_{dissipated}$$

Acceptability of the conceptual design is rechecked for technical soundness and economic feasibility, preferably by independent peer review.

Preliminary Design and Acceptability Check: Preliminary design is based on providing required stiffness. Design procedure includes:

1. Estimation of stiffness and period, T_1, required to control damage.

2. Preliminary sizing based on required stiffness

3. Seismic force estimation using inelastic spectra and the estimated T_1 for a two level design (typically for Fully Operational and Life Safe criteria), accounting for all critical load combinations.

Acceptability of critical response parameters is checked using elastic dynamic, linear and nonlinear pushover or nonlinear time history analyses, as appropriate. Parameter checks include deformation, velocity, total acceleration strength, ductility ratio and energy dissipation capacity.

Preliminary Design for Nonstructural Components: Nonstructural components are designed to be either coupled or uncoupled from the structural response (isolated or integral with the structural frame) and are accounted for in the structural analysis model.

Final Design and Acceptability Check: Final member sizing and detailing are designed, preferably using energy balance concepts. Final detailing of nonstructural components and connections is designed. Acceptability is checked using best available 3-D nonlinear analysis techniques. Independent peer review of design is preferred.

Quality Assurance: Quality assurance includes peer review and plan checking of design documents, construction observation by the design professional, testing and special inspection, and construction quality control by a resident engineer where applicable.

Building Maintenance: Structural system is maintained to guard against deterioration and alteration.

C. *Advantages of Approach*

The Comprehensive Design Approach details a thorough procedure to account for all significant issues in the seismic design problem. Reference documents for the approach contain a wealth of reference information regarding issues affecting the seismic design problem. The energy balance concept allows latitude in the design solution which enables economic optimization in the design and detailing. The approach may be highly effective in meeting performance objectives. The approach has application to irregular, unusual and important structures and as a calibration tool for testing simpler design approaches.

D. *Disadvantages of Approach*

The comprehensive design approach is not practical to use in its entirety for typical building design due to the scope and complexity of information to be gathered and the analyses to be performed.

E. *Research Needs*

Research needs for the Comprehensive Approach cover the gamut of research needs for performance based design, as summarized below and discussed in greater detail in Appendix B. Research needs include:

- Development of more reliable seismic hazard data including: source maps, microzonation maps, seismic hazard probability maps, time history prediction, damage potential estimation, and adjustment of SDOF spectra for MDOF effects.
- Identification of response parameters critical to seismic performance and qualification of limiting values.
- Development of more reliable load combination factors and methods.
- Development of more reliable methods to account for effects of torsion, higher modes, ductility demand concentration in weak stories, and over strength.
- Development of practical techniques for considering energy concepts and energy balancing.
- Development of capacity design and detailing procedures for various materials and systems. Development of reliable ductile "fuse" details for all materials.
- Development of improved acceptability analysis procedures to reliably predict the response of a structural design.
- Education of the practitioners, academicians, building officials and public regarding the goals, ways and means of performance based seismic engineering.

F. *References*

Bertero, R.B., Bertero, V.V., *Tall Reinforced Concrete Buildings: Conceptual Earthquake-Resistant Design Methodology*, Earthquake Engineering Research Center Report No. EERC-92/16, University of California, Berkeley, CA, 1992.

3.3 Displacement Based Design Approach

A. *Overview*

The Displacement Based Design Approach uses displacement rather than force as the starting point for seismic design, with the presumption that control of displacement, or drift, is the key to controlling the performance of the structure.

In traditional force based seismic design, the structure is designed elastically for a reduced acceleration response and associated forces. Displacement or drift is then checked as part of the design acceptability check. In the displacement based approach, the design process is inverted by designing the building for displacement control and then checking it for forces. The approach is briefly summarized in this section and presented in greater detail in Appendix C.

When dealing with an existing building, the structure for the starting point is known and the method differs from conventional design only in that the performance measure is deformation rather than strength, and deficient conditions are improved by adding stiffness instead of strength. For new buildings, the process of using displacement criteria as a basis for apportioning the structure to define the initial design is new and not familiar to most engineers. One method for approaching new construction with a displacement based criterion is reported by Kowalsky, Priestley and MacRae in 1994 (Kowalsky-1994). This research report focuses on single degree of freedom concrete bridge structures. Follow-up work is in progress on multi-degree-of-freedom building structures. The approach is very promising for performance based engineering application considering the critical role that displacement plays in seismic performance and the reported success of the initial SDOF procedure. An alternate approach more specifically tailored to building structures is also presented in Appendix C.

As adapted to performance based engineering, the Displacement Based Design Approach begins with establishment of the target ultimate, yield and service level displacements or drifts, consistent with the selected performance objectives. The target displacements should be adjusted to a ratio consistent with the ductility capacity of the anticipated structure. Typically, a substitute elastic structure can be used to model the effective stiffness of the inelastic structure at the ultimate displacement. An appropriately damped elastic displacement response spectrum (DRS) will be used with the substitute structure to design the real structure. Other spectrum, with adjusted damping, will be used with the real structure in its elastic range to design for the frequent earthquake as appropriate for the performance objective selected.

B. *Methodology*

The overall methodology for the Displacement Based Design Approach parallels the generalized methodology outlined in Chapter 2; however, the actual design procedure is very different than that for force based approaches. The Displacement Based Design Approach can be summarized as follows (see Figure 3-3):

Selection of Performance Criteria: The Displacement Based Design Approach can be used with any performance objective.

Site Suitability & Ground Motion Analysis: A complete site and seismic hazard analysis is done. Elastic displacement response spectra (DRS) are generated for ground motions to be considered.

Conceptual Design: Structural and nonstructural systems are selected and structure is configured with a ductile fuse system using capacity design. Initial parameters including target displacements or drifts are selected based on the performance objective and damping relationships for the structural system are established. The predicted displacements are checked against the appropriate displacement response spectrum.

Preliminary Design: Member sizes are derived from target displacements using a substitute elastic structure to model the inelastic response of the system:

1. Determine effective stiffness for the substitute structure.

2. Determine design forces based on calculated deformations.

3. Design members based on yield forces and vertical loads.

4. Recheck assumed elastic and ultimate displacements and ductility demand; re-iterate design if necessary.

Acceptability Checks: Design procedure self-checks displacement, elastic stresses and ductility ratios. Additional acceptability check may be required for other performance levels and specific hazard levels.

Final Design and Acceptability Check: Detail structure in accordance with capacity design principles. Perform acceptability check to validate design.

Quality Assurance and Building Maintenance: Same as for generalized methodology.

C. *Advantages of Approach*

The Displacement Based Design Approach affords several advantages over force based approaches, due to its direct focus on the inelastic displacement response of the structure:

1. Displacement (drift), which intuitively is a better indicator of structural damage and thus performance than is force, is the basis of the design.

2. Nonstructural and structural damage can be largely controlled by controlling displacement (drift). The limiting values of displacement associated with the selected performance levels can be the target displacements for the design.

3. Plastic deformation and resulting structural damage can be better controlled.

4. Ductility demand is directly estimated, giving better basis for structural detailing.

5. P-Δ effects can be calculated directly.

6. Drift-related pounding between adjacent structures can be better controlled.

7. The Displacement Based Design Approach provides a unified, rational and straightforward approach to seismic design with direct insight and control over the inelastic response and the ductility demand.

D. *Disadvantages of the Approach*

The primary disadvantage of the approach at present is that it has not yet been developed and validated for multi-degree-of-freedom systems. It also relies heavily on establishment of realistic effective damping relationships. It requires additional research in order to be applied.

E. *Research Needs*

Specific research needs to develop displacement based design include the following:

- Procedures for applying approach to MDOF systems.
- Methods for applying approach to concrete shear wall, wood frame, and other structural systems.
- Calibration studies applying the approach to real buildings for which earthquake response records are available.
- Development of realistic effective damping and effective period relationships for MDOF systems and various structural systems and structural/nonstructural combinations.

F. *References*

Kowalsky M.J., Priestley M. J. N., and MacRae G.A., *Displacement-Based Design*, Structural Systems Research Project Report No. SSRP - 94/16, University of California, San Diego, 1994.

Qi, X., and Moehle, J.P., *Displacement Design Approach for Reinforced Concrete Structures Subjected to Earthquakes*, Report No. EERC 91/02, University of California, Berkeley, 1991.

3.4 Energy Based Design Approach

A. *Overview*

The Energy Based Design Approach is based on the premise that damage is directly related to total input energy. It is expected that the potential for an earthquake ground motion to damage a specific structure and its contents is closely associated with the energy input to that structure and its energy dissipative capacities.

The energy input is a function of the ground motion's effective velocity and duration and the interaction between the ground motion and the structure. Velocity and duration are not accounted for in the acceleration response spectra commonly used in design. The energy approach directly accounts for duration effects and the energy content of the ground motion at different structural periods as well as for the degradation of the structural system through its cyclic response.

The Energy Based Design Approach begins with estimation of the energy input, or demand, of an expected design earthquake at various frequencies and then the structure is designed to provide an energy absorption and dissipation capacity greater than the expected demand. Such an approach was proposed by Housner in 1956 and has gained increasing attention in recent years. In order to utilize an energy approach, it is necessary to develop methods for estimating the energy demand of design earthquakes, the distribution of energy demands to individual structural components as well as methods for quantifying the energy dissipation capacity of the structural elements undergoing cyclic hysteretic response at different performance levels. A more detailed discussion is presented in Appendix D.

The general energy equation for design is given by:

$$E_{input}=E_{elastic}+E_{dissipated}=(E_{kinetic}+E_{strain})+(E_{damping}+E_{hysteretic})$$

The absolute input energy is the work done by the force applied to the structure at the foundation, acting through the foundation displacement. The energy capacity of the structure is represented by the elastic energy capacity plus the energy dissipation capacity associated with damping and the cyclic plastic hysteretic response.

In regions of strong ground motion potential, it is generally uneconomical to design structures to absorb more than a small fraction of the input energy through elastic response, therefore, seismic design must provide energy dissipation through plastic hysteretic behavior and damping behavior. The plastic hysteretic behavior is designed into the structure through the use of ductile elements, for which the energy dissipation capacities (hysteretic response histories) must be established. To the extent that ductility capacities are limited and hysteretic behavior entails degradation and damage to the structural system, the energy approach points toward the beneficial use of energy dissipation devices as an attractive design strategy for some structures.

B. *Methodology*

A conceptual energy based design methodology can be outlined as follows (see Figure 3-4):

Selection of Performance Criteria: The Energy Approach can be used with any performance objective.

Site Suitability Analysis and Design Ground Motions: Site suitability is analyzed according to Section 2.3. Elastic design spectra are developed for serviceability design. For other performance designs involving inelastic response, time histories are developed for use in tracking the energy delivered to each component for comparison to energy capacities.

Conceptual Design and Acceptability Check: The conceptual design follows the guidelines outlined in Section 2.4. The energy capacities can be qualitatively budgeted between the elastic, hysteretic and damping systems. Systems are then selected and configured so that the energy dissipation due to plastic deformation and damping capacities can balance the inelastic energy demand.

Preliminary Design and Acceptability Check: The structure is proportioned so that the elastic capacities meet the Fully Operational level demands in a manner consistent with capacities that can be developed in the final design and detailing. The drift response, strength demand and energy demand are checked for consistency with the performance objective acceptability criteria. Acceptability analysis procedures include elastic analysis for Fully Operational designs (elastic response) and 3-D nonlinear time history analysis for other performance designs (inelastic response).

Final Design and Acceptability Check: The ductile elements are detailed so that the hysteretic energy capacity exceeds the hysteretic energy demand. Energy dissipation damping devices, if used, are detailed so that the capacity exceeds the damping demand.

Quality Assurance and Building Maintenance: Same as general methodology, Chapter 2. Particular attention is required to the need for long-term maintenance of energy dissipation devices, if used.

C. *Advantages of Approach*

An energy approach can take direct account of the damage potential, including spectral energy content and duration, of the expected ground motions. It can account for the degradation in the hysteretic behavior of elements and it provides the design professional with a good conceptual understanding of the inelastic demands on the structure. It also allows for the use of energy dissipation devices and base isolation techniques which are promising technologies for performance based seismic design and damage control.

D. *Disadvantages of Approach*

The energy approach has not been fully developed at this time and its practicality is unknown. Additional research is necessary into both the energy demand and particularly the energy capacity sides of the equation, as outlined below. Practical and effective design procedures need to be developed.

E. *Research Needs*

The research needs for an energy approach parallel those for the Comprehensive Design Approach, with special research needs for the following areas:

- Methods of predicting energy demands of earthquake ground motions on structures and energy demand distribution to the structural elements.
- Development of reliable detailing methods to create elements with reliable energy capacities and predictable hysteretic energy dissipation behavior.

F. *References*

Blume, John A., *Elements of a Dynamic-Inelastic Design Code*, Proceedings of the Fifth World Conference on Earthquake Engineering, Rome, Italy, 1973.

3.5 General Force/Strength Approach

A. *Overview*

The force/strength approaches include the most common seismic design approach in use today. This approach is used in the Uniform Building Code, NEHRP and other current model codes. Force/strength approaches are based on determining a minimum lateral base shear force, distributing it over the height of the building either by a static or a dynamic procedure, designing members with sufficient strength to resist the resulting member forces, and detailing key components with sufficient ductility.

Current code methods typically utilize an elastic response spectrum to describe the seismic hazard. Forces derived based on the elastic spectrum are reduced with a "R" factor to account for the ability of structure to exhibit inelastic behavior. The structural design is then performed at working stress (or yield) levels based on these reduced seismic forces combined with all other loading conditions. Current code methods require calculations for only a single earthquake level (10% exceedance in 50 years). This strength design, once combined with the system limitations, detailing requirements and quality assurance standards also required by the current codes, lead to their stated performance objective; a performance similar to the standard objective defined in Chapter 2. So far, the code approach has been reasonably effective in achieving its performance goals for well designed, regular structures with complete and redundant lateral force resisting systems.

The General Force/Strength Approach can be expanded to accomplish the goals of performance based engineering by modifying the current code approaches to address the full scope of issues in the conceptual framework. Such modifications to adapt the approach should include the following:

- Explicit integration of capacity design concepts.
- Improvement in the ground motion characterizations to reflect the four design events identified in Section 2.2 of this report in an appropriate form for design.
- Introduction of multi-level designs as needed to address the full range of performance objectives.
- Inclusion of inelastic design methods, with use of nonlinear time histories, inelastic response spectra, where inelastic response is anticipated (typically only the serviceability design will be elastic).
- Establishment of acceptability criteria and inclusion of acceptability analysis in the design process, for both the elastic and the inelastic response.
- Inclusion of design for nonstructural components in the scope of design.
- Improvement of quality control in the design and construction.
- Improvement of the structural maintenance procedures over the life of the structure.

A more detailed discussion of the General Force/Strength Approach and the modifications to the approach can be found in Appendix E.

B. *Methodology*

A methodology for the improved General Force/Strength Approach is illustrated in Figure 3.5. It can be compared to the generalized performance based engineering methodology discussed in Chapter 2 as follows:

<u>Selection of Performance Objectives:</u> The General Force/Strength Approach can be used with any selection of performance objectives. However, because it incorporates elastic analysis procedures for doing acceptability checks, the results must be calibrated to be quite conservative and applicable to only a subset of building types. Otherwise, its reliability may be limited for performance objectives with large inelastic demands and for structures with concentrated inelastic behavior.

<u>Site Suitability Analysis and Design Ground Motions:</u> Perform complete site analysis and characterize ground motion for each performance goal using elastic and inelastic spectra.

<u>Conceptual Design:</u> The General Force/Strength Approach can be used with any framing system adaptable to the guidelines for appropriate seismic design in Section 2.4. Capacity design principles should be followed to control inelastic response with a system of ductile "fuses".

Preliminary Design and Acceptability Check: Consider all performance goals, e.g. Fully Operational and life safe, using the revised equivalent lateral force method and design for strength.

Final Design and Acceptability Check: Complete the design and detailing and check all design objectives using appropriate acceptability methods. Detailing should follow capacity design principles.

Design Review, Quality Assurance and Building Maintenance: Same as generalized methodology.

C. *Advantages of Approach*

The primary advantage of the General Force/Strength Approach is its familiarity to the practicing engineer and its ease of application. With the modifications and enhancements outlined above, the approach can be successfully adapted to performance based engineering. The incorporation of capacity design principals improves the clarity of the concept and the reliability of detailing. The incorporation of simple inelastic acceptability analysis procedures will improve understanding and control of the inelastic response of the structure.

D. *Disadvantages of Approach*

The existing force/strength methods currently in use at times oversimplify the seismic design from a conceptual stand point and over complicate the component detailing. The current methods are based on linear elastic behavior and account for inelastic behavior through the use of simplistic "R" values which depend only on the structural type. Existing methods also address only one performance level directly while claiming to achieve a full set of performance objectives.

The modified and extended general force/strength procedure must be developed and expanded to these limitations.

E. *Research Needs*

The research needs for the General Force/Strength Approach are the same as for the Comprehensive Approach outlined in Section 3.2. Of particular importance to the General Force/Strength Approach is research into the following:

- Development of reliable elastic and inelastic strength and displacement response spectra.
- Refinement of the "R" factor to account for over strength, force level, energy dissipation, and yielding sequence effects.
- Study of effects of torsion and of ductility demand concentrations due to structural irregularities.

3.6 Simplified Force/Strength Approach

A. *Overview*

A Force/Strength Approach similar to current code approaches can be developed for performance based engineering design of certain types of relativity simple buildings on favorable building sites. Such a simplified approach should rely on simplified numerical analysis, backed up by strict code limitations as to its applicability. Restrictions will be set regarding the structural configuration and proportions, structural system, gravity load to seismic load ratios, detailing, nonstructural systems, foundation type and site conditions. The approach will be applicable for all performance objectives for buildings meeting its criteria. Even with the enumerated restrictions, it is anticipated that a simplified approach can be applied to a wide range of typical buildings.

Similar simplified approaches have proven effective as in the case of the city of San Juan, Argentina. San Juan was devastated by an earthquake in 1944 and was then rebuilt to a simplified code. It survived a similar earthquake in 1977 with no significant damage.

B. *Methodology*

The Simplified Force/Strength Approach is illustrated in the flow chart in Figure 3-6 and the details of the method are developed in Appendix F. The approach should be an enhancement of current code methods. Compared to the general force/strength approach presented in Section 3.5, the primary enhancements should include the following:

1. The Preliminary and Final designs should target the Life Safe objective only, using linear elastic analysis methods. Application of the method will be restricted to buildings and sites where the life-safety criteria governs the designs and where the elastic response can be extrapolated to give a good indication of the inelastic response. Methods of extrapolating from the elastic to the inelastic should be developed and incorporated in the simple procedures.

2. Acceptability checks as defined for the preliminary and final design will not be required.

3. The enhanced numerical design procedure should be based on the force/ strength approach of the current UBC or NEHRP provisions, using strength design rather than allowable stress design and using improved "R" factors as discussed in Section 3.5.

C. *Advantages of Approach*

The chief advantage of the approach is its familiarity and relative ease of use. The approach is very similar to current practice, with modest enhancements, and therefore can easily be put into practice. It will provide several improvements over current practice.

Those improvements include:

- Explicit adoption of capacity design principles.
- Improved guidelines to conceptual design and final detailing.
- More realistic and reliable strength and displacement response spectra.
- Better guidelines to realistic load combinations.
- Improved quality control requirements for construction.

The Simplified Force/Strength Approach can lead quickly to more effective performance based engineering by improving current practices.

D. *Disadvantages of Approach*

The Simplified Force/Strength Approach does not apply to all buildings or to all performance objectives. Its use can lead to unnecessary conservatism where more sophisticated approaches are appropriate. Its development out of the current code provisions may result in an overlay of additional specific requirements without clarification of design concepts and the design process.

E. *Research Needs*

The research needs are essentially the same as for the General Force/Strength Approach, however, because of the simplifications in the design and acceptability check procedures, there is particular need for studies and development of the following:

- Limitations in applicability based on building types, configurations and site conditions.
- Reliable elastic and inelastic response spectra for strength and displacement.
- Simple methods to include duration effects for ground motions.
- Capacity design detailing guidelines to accommodate inelastic behavior.

3.7 Prescriptive Design Approach

A. *Overview*

A large number of smaller residential and commercial buildings are currently constructed using prescriptive code approaches, such as the conventional construction code provisions, rather than through engineering analysis and design. In recent years, these prescriptive provisions have been applied to larger and more complex structures and have met with mixed results in subsequent earthquakes. The continued use of the prescriptive provisions points to the need to develop an improved prescriptive approach consistent with limited performance based engineering objectives.

The proposed Prescriptive Design Approach should be the simplest performance based approach to apply but will bring with it the most severe limitations on the design. The prescriptive concept is to develop a set of specific detailing requirements that can be used, without an engineer, to design simple buildings to meet the Basic performance objective. The approach should be restricted to small, light weight, regular buildings and will outline clear limitations to selection of structural systems, layout and configuration, detailing and quality control. It should provide illustrated guidelines for sizing and spacing of lateral load resisting elements, and should provide pre-approved details for shear-resisting elements, hold down anchorages, and drag and collector connections to assure a complete load path to the foundation. It will necessarily rely on redundant systems and conservative requirements to compensate for lack of engineering design. It should depend heavily on quality assurance provided by the building official in plan review and construction inspection.

Modified prescriptive approaches can be developed for seismic strengthening of simple buildings of various types and for anchorage of nonstructural components.

B. *Methodology*

The Prescriptive Design Approach methodology will be much simplified compared to the general performance based engineering methodology, as illustrated in Figure 3-7 and as highlighted below:

> <u>Performance Objectives:</u> Only the Basic Objective can be selected.
>
> <u>Site and Seismic Hazard Analysis:</u> Only regional and local zoning regulations and seismic zonation are considered.
>
> <u>Design:</u> Prescriptive requirements and details are incorporated into layout and design.
>
> <u>Acceptability Checks and Construction Quality Control:</u> Building official checks plans and inspects construction for compliance with prescriptive requirements.
>
> <u>Building Maintenance:</u> Building official checks alterations and remodels.

C. *Advantages of Approach*

The Prescriptive Design Approach will provide simple guidelines for developing seismic resistance in small, simple buildings without direct engineering input and without specialized knowledge. It will provide reasonable seismic resistance to the large number of non-engineered buildings. The guidelines can be clearly presented and illustrated to facilitate effective design and plan checking and to improve reliability of the approach.

E. *Disadvantages of Approach*

Disadvantages of the approach include its narrow scope, the difficulty in preparing effective guidelines, and the heavy reliance on the building official as the only professional involved for acceptability checking and construction quality control. The approach is limited to a narrow range of simple buildings. It will be difficult to write simple, clear guidelines and limitations to cover the myriad of building configurations, irregularities, offsets and discontinuities. Effectiveness of the approach will rely heavily on the building official to interpret and enforce the requirements. Without specific design and professional involvement, construction quality control will ride heavily on the diligence of the building official.

F. *Research Needs*

Research and dynamic testing is needed to determine load/deformation behavior and performance characteristics of assemblies and components to be specified in prescriptive requirements, including:

- Wood and metal stud framing assemblies including shear walls, shear walls with openings, and multi-story shear walls.
- Component testing of sill anchorage, hold downs, sheathing types, shear connectors, new materials and components.
- Components as built in the field and as effected by shrinkage, aging and other field conditions.
- Review of performance of similar systems in past earthquakes.

The previously described Comprehensive Design Approach applied to representative structures can be used to determine appropriate minimum prescriptive detailing requirements.

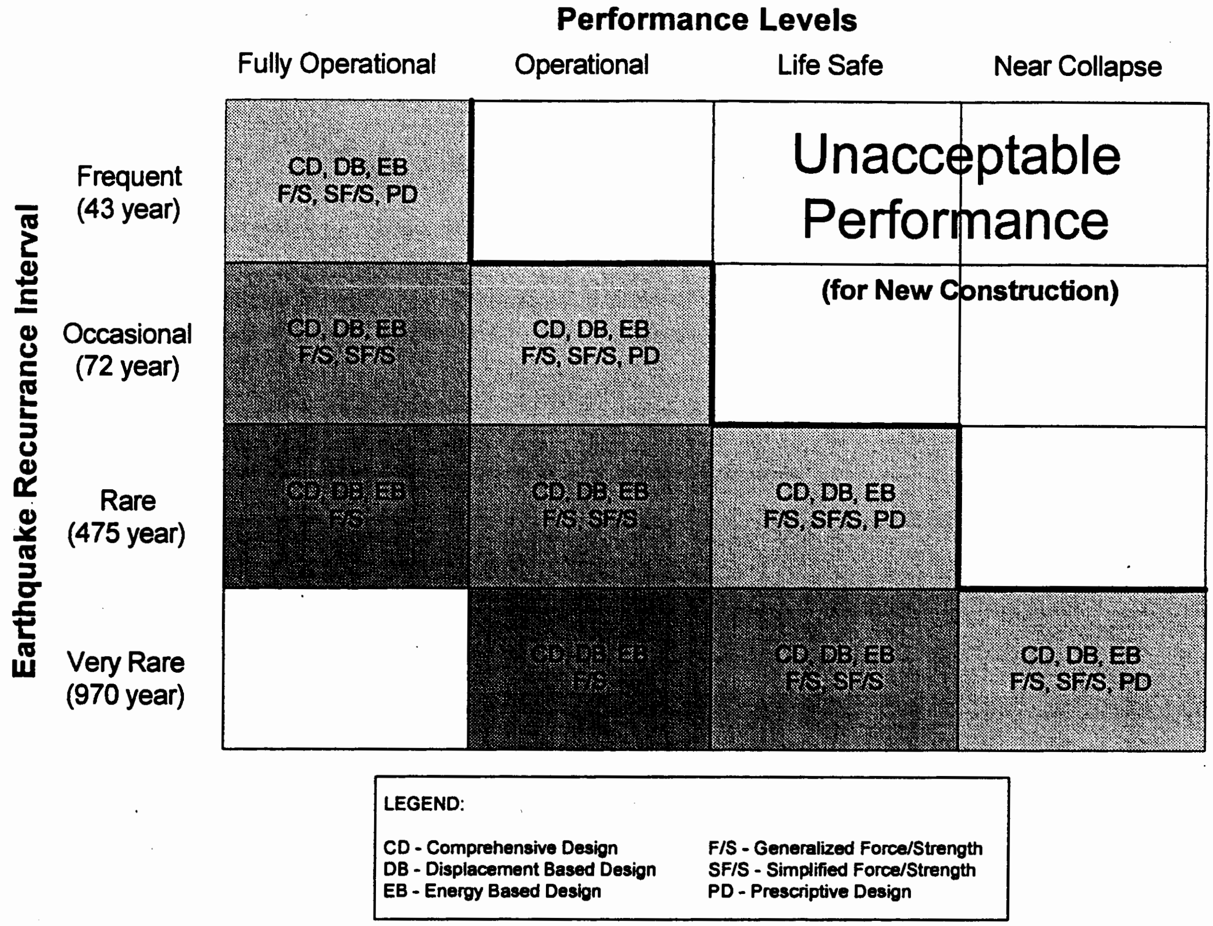

Figure 3-1: Performance Objectives with Design Approaches

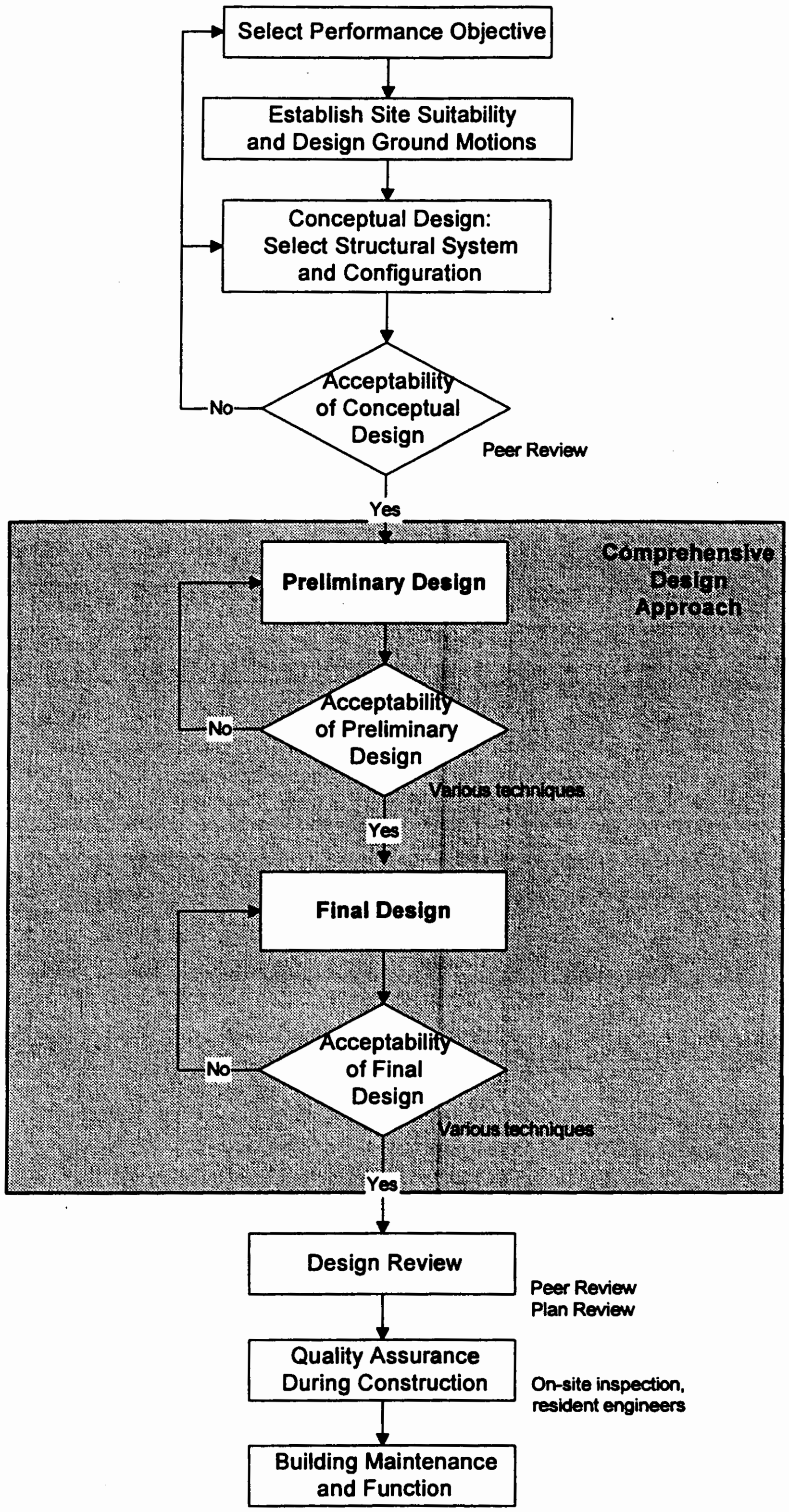

Figure 3-2: Methodology for Comprehensive Design Approach

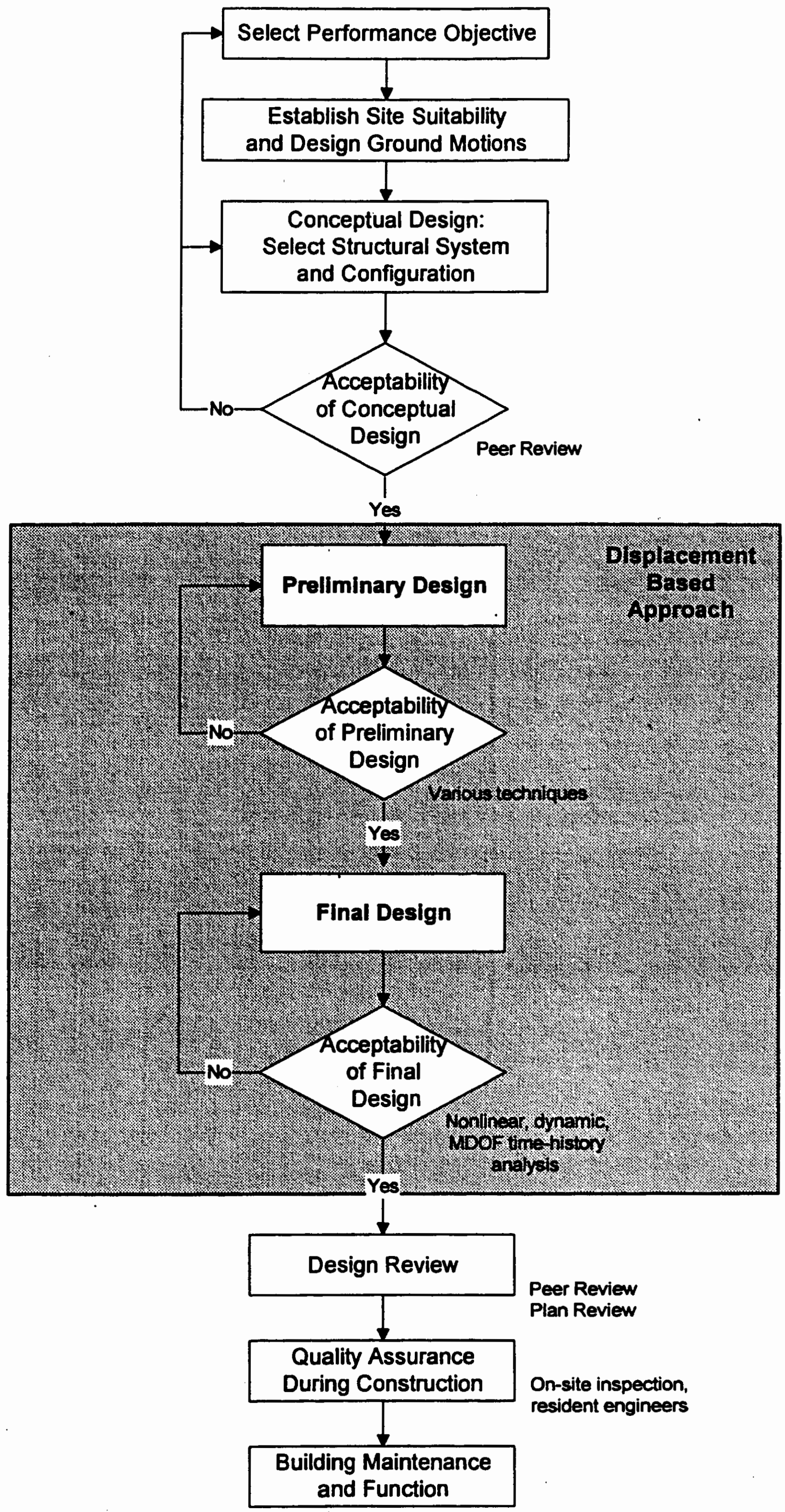

Figure 3-3: Methodology for Displacement Based Design Approach

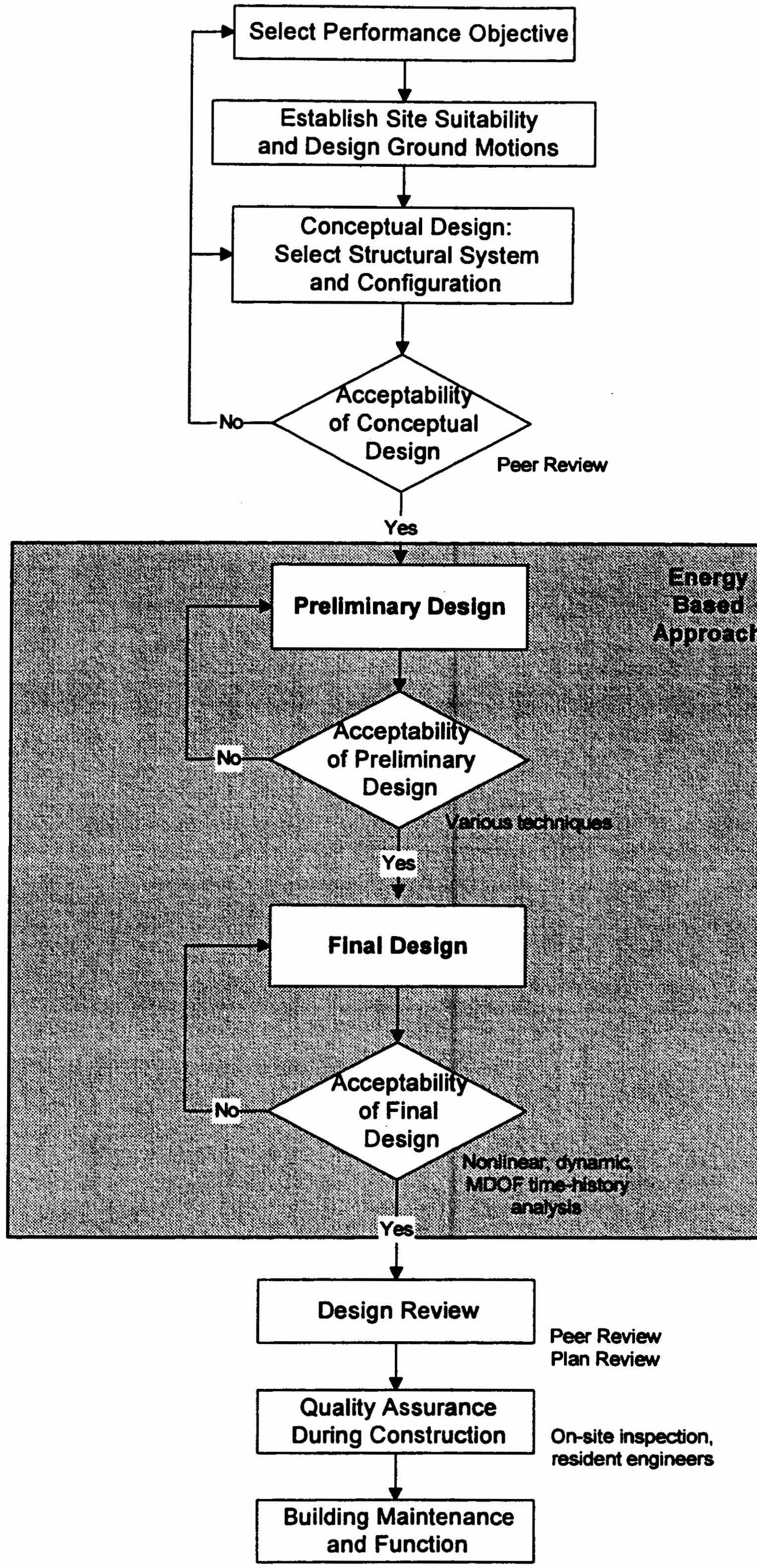

Figure 3-4: Methodology for Energy Based Design Approach

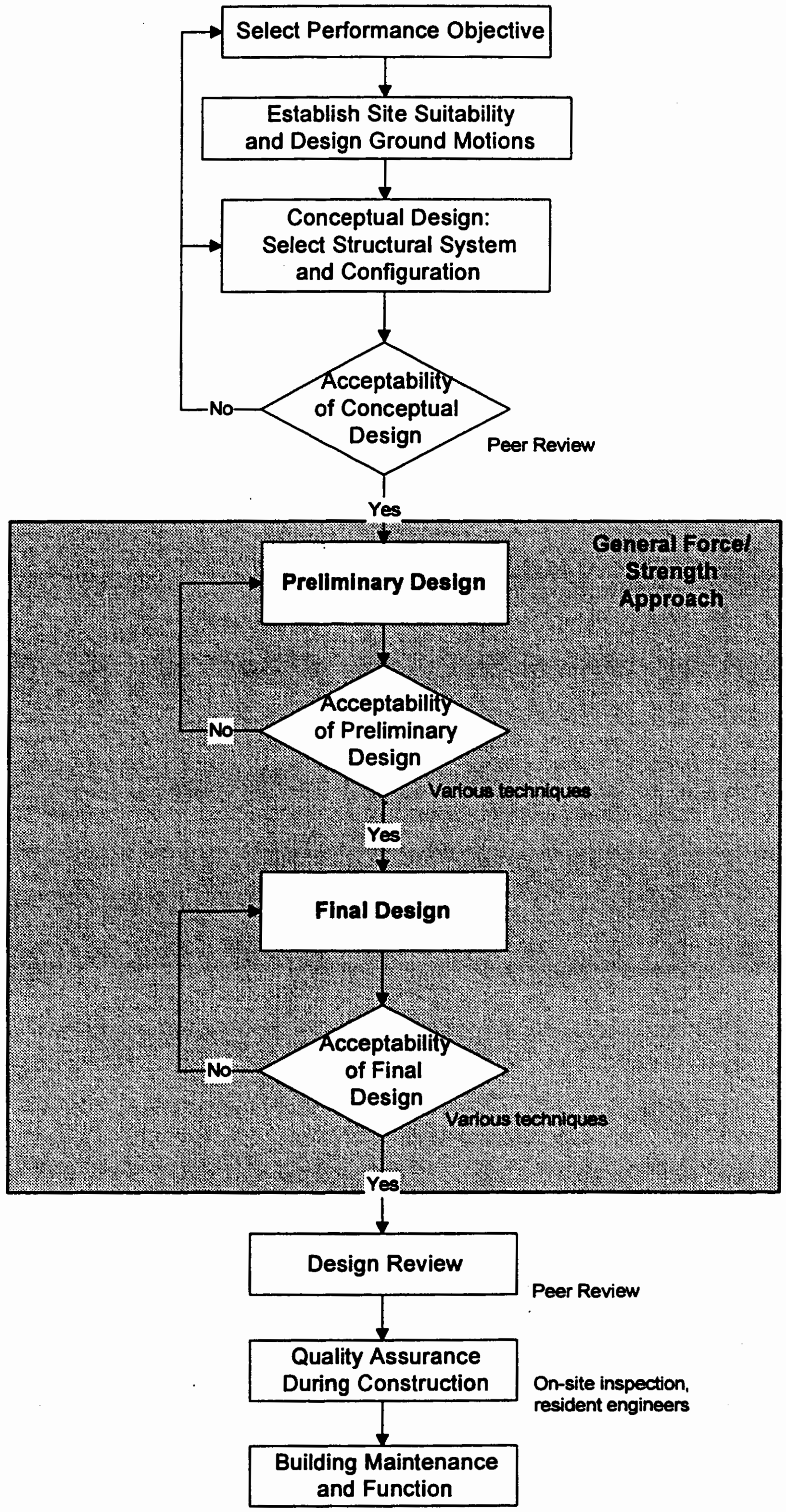

Figure 3-5: Methodology for Generalized Force/Strength Approach

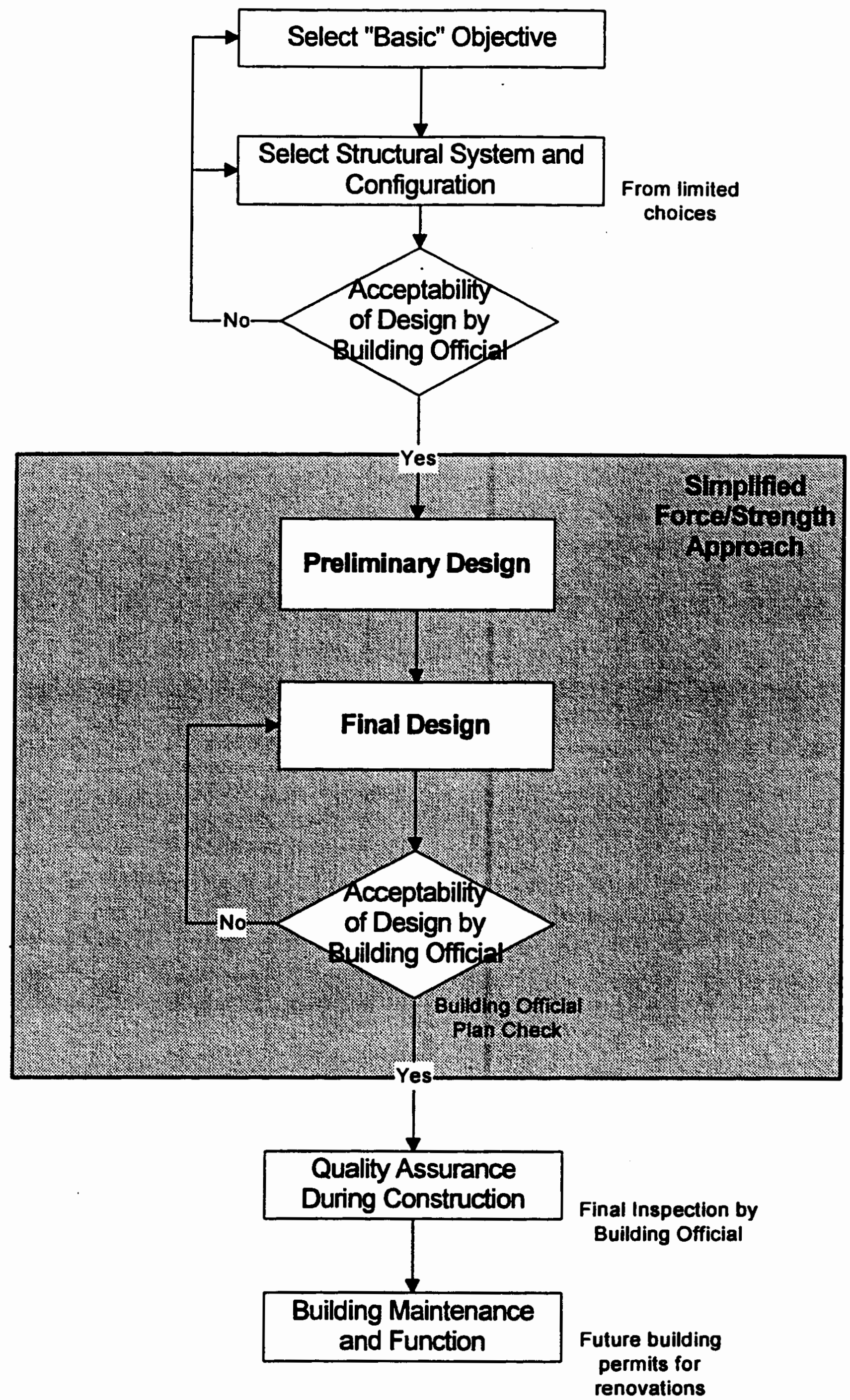

Figure 3-6: Methodology for Simplified Force/Strength Approach

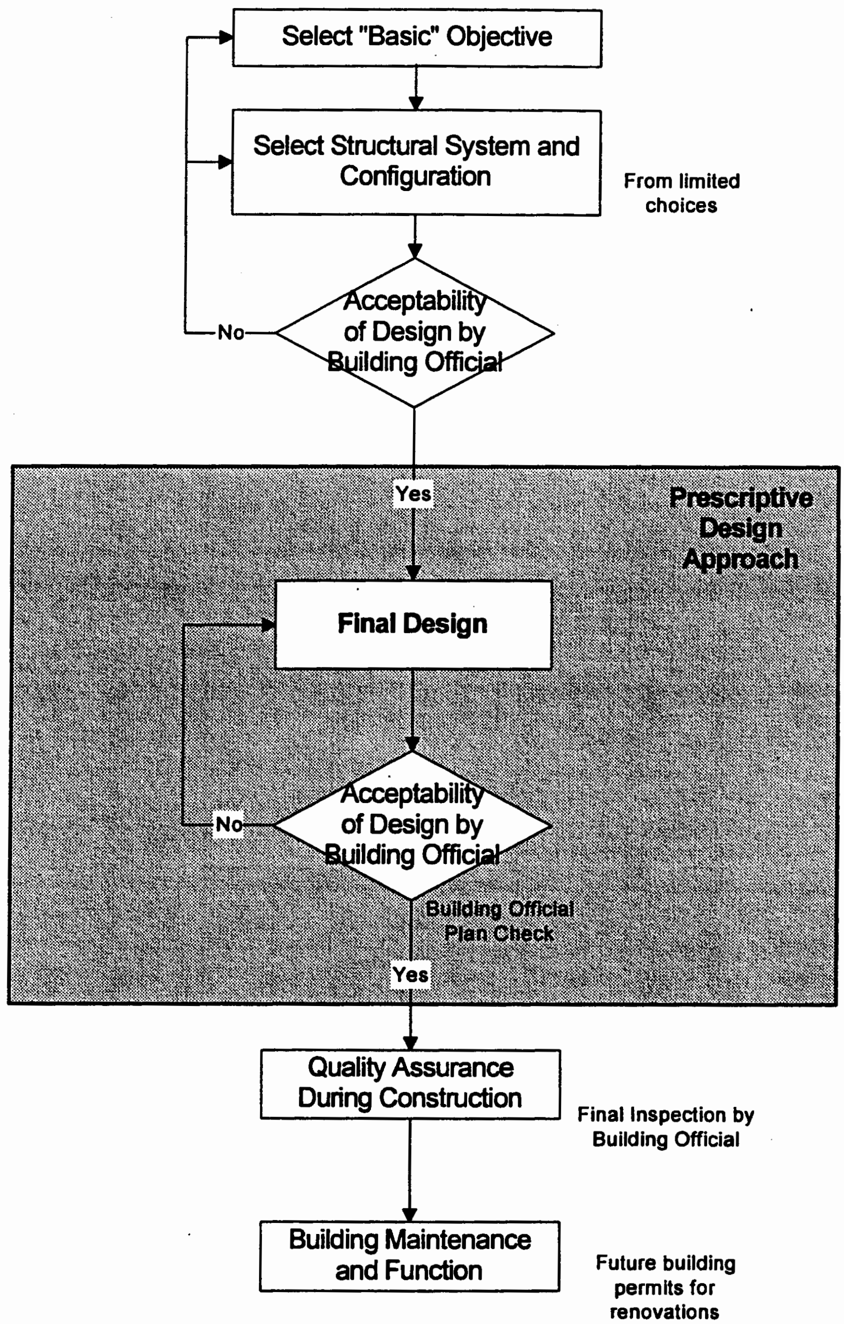

Figure 3-7: Methodology for Prescriptive Design Approach

4. ACCEPTABILITY ANALYSIS PROCEDURES FOR PERFORMANCE BASED DESIGN

4.1 Overview

In the Performance Based Engineering methodology, the key step to verifying the seismic design is the acceptability analysis. The design procedure typically endeavors to provide a strength, displacement, or energy capacity at one or more design ground motions. The acceptability analysis is used to check the adequacy of the design. To verify that the design objectives and acceptability criteria are being met, the structural design is analyzed to compare the critical response parameters with the limiting values of those parameters associated with the selected performance levels.

The limiting values of the various structural response parameters make up the acceptability criteria for the design. The response parameters are measures of the structural response that can be correlated with damage levels and, thus, with the performance objectives. The most critical of the parameters can include drift, displacement, stress and strain ratios, ductility demand, energy demand and acceleration. Limiting values for some parameters have been suggested in this report. Additional study and testing in this area is required to establish appropriate parameters for various components and limiting values for each performance level for use in design analysis.

Several acceptability analysis procedures applicable to performance based engineering are summarized in this chapter. These include Elastic Analysis procedures, the Component Based Elastic Analysis procedure, the Capacity Spectrum Method, Pushover Analysis Methods, Dynamic Nonlinear Time History Analysis and the Drift Demand Spectrum Method. Each of these procedures can be used with the possible design approaches presented in Chapter 3, and each should be used with an applicable structural system. However, the design approach, structural system and selected performance objectives may favor one procedure over another. For example, the prescriptive approach does not require any of these analyses, whereas selection of the Safety Critical performance objective may necessitate the use of dynamic nonlinear time history analysis procedures.

The proposed procedures are summarized in the following sections. Additional discussion is included in the Appendix and reference documents are listed for a few procedures. It will be a task of future guideline development efforts to develop and improve these and alternate analysis procedures.

4.2 Elastic Analysis Procedures

A. *Overview*

Most current seismic code provisions and seismic design practices are based on elastic analysis procedures. These procedures include the static and dynamic analysis procedures used in the equivalent lateral force analysis, response spectra analysis, modal analysis and elastic time history analysis. These methods are well documented in seismic engineering literature and are generally well understood and extensively used.

Generally, elastic analysis procedures can give very accurate information regarding the elastic response of the structure, assuming the modeling is appropriate and the ground motion is properly represented. For the Fully Operational, performance level, which is generally based on elastic response, the elastic procedures provide appropriate analysis.

When significant inelastic response is permitted to occur at other performance levels, the elastic analysis procedures must be used with caution. Current practice uses various 'R' factors and other factors to extrapolate from an idealized elastic response to the expected inelastic response. This approach has been fairly successful at meeting Near Collapse and Life Safe objectives. Well conceived structures with properly detailed ductile elements can achieve basic stability objectives, within acceptable levels of reliability, using elastic analysis. However, to improve the reliability of designs and to achieve intermediate performance objectives which limit damage, better analytical tools are needed. The elastic procedures do not directly analyze the inelastic structural response or measure inelastic response parameters to sufficiently evaluate design acceptability.

B. *Advantages of Procedure*

1. Elastic analysis procedures are familiar and easy to use.

2. Procedures are effective for Fully Operational Performance Objective and, with load factors, are somewhat effective for other levels of design.

3. 3-D dynamic analysis provides some insight into torsional response and response of irregular structures.

C. *Disadvantages of Procedure*

1. Use of elastic procedures, particularly dynamic procedures, can obscure understanding of the inelastic nature of seismic response and can lead to false confidence in the numerical analysis.

2. Elastic modal superposition and time history analysis procedures increasingly lose validity with increasing inelastic response and structural irregularity.

3. Use of factors to extrapolate from the elastic response introduces sufficient uncertainty so as to limit the reliability of performance based design.

D. *Research Needs*

- Development of improved analysis methods and factors to extrapolate from the elastic response to the inelastic response.
- Development of appropriate limitations to the application of elastic analysis in performance based design.

E. *References*

SEAOC, *Recommended Lateral Force Requirements and Commentary*, 1990.

4.3 Component Based Elastic Analysis Procedure

A. *Overview*

Component Based Elastic Analysis Procedures are extensions of the linear analysis procedures which form the basis of current codes. They can be applied as a measure of ductility demand on a component-by-component basis. These procedures have been adopted by several recent seismic design standards including the Tri-Service Manual for Essential Buildings, and the Department of Energy Design and Evaluation Guidelines for Moderate and High Hazard facilities. A procedure of this type will also be adopted by the ATC-33 project "Guidelines and Commentary for Seismic Rehabilitation of Buildings," currently under development.

The basic approach is to perform an elastic analysis of the structure. Ground motion input may be represented either by an inelastic response spectrum, or by a standard elastic response spectrum factored to account for the effects of inelastic behavior. Based on analytical research, it is postulated that the overall structural displacements predicted by this analysis are representative of the actual displacements expected in the real structure when responding to the design earthquake. It is recognized that due to the ability of well detailed structures to exhibit ductile behavior, the stresses on individual elements predicted by the elastic analysis are larger than will really be experienced. Demand to capacity ratios, sometimes called inelastic demand ratios (IDRs), are calculated as the ratio of the strength demand predicted by the elastic analysis to the yield capacity of the individual elements. These ratios are taken as a measure of the ductility demand produced by the design earthquake on the various elements of the structure. Permissible values of the IDRs are specified based on the quality of detailing provided, the importance of the individual element to overall structural stability and the ability of the element to withstand large inelastic demands, as substantiated by laboratory test data.

B. *Advantages of Procedure*

The Component Based Elastic Analysis Procedures are very easy to use and provides some insight into the inelastic demands on a structure, particularly if overall inelastic demands are moderate and are well distributed throughout the structure. If realistic limiting values for the IDRs are established, the method can be both easy to use and effective.

C. *Disadvantages of Procedure*

The Component Based Elastic Analysis Procedures may not provide a good indication of the real distribution of inelastic demands, particularly if a structure has non-uniform strength and stiffness distributions, or if it has a tendency to develop inelastic soft stories of torsional irregularities. There is no direct way to relate IDR values to inelastic deformation demands, and consequently, behavior of the individual structural elements.

D. *Research Needs*

Further study is required in the following areas:

- Establishing realistic limiting values for IDRs, and to correlate the IDR limits with the performance levels identified in this document.

- Define the conditions of structural regularity, loads and response for which this approach is valid.

4.4 Capacity Spectrum Method

A. *Overview*

The Capacity Spectrum Method compares the inelastic capacity of a structure to the response spectrum demand for the governing earthquake ground motion. The inelastic response of a structure is characterized by a sequential softening of the structure. As its stiffness degrades, its fundamental period lengthens in response to a monotonically increasing lateral load and the strength demand on the structure decreases, as indicated by the downward slope of the response spectrum for longer period systems. As long as the structure remains stable, once the increasing inelastic capacity exceeds the decreasing inelastic demand, as illustrated by graphical solution, the structural design is considered to be acceptable.

The 1985 Tri-Service Manual outlines a capacity spectrum method in which a capacity spectrum curve can be plotted and directly compared to a response spectrum demand curve. The capacity spectrum is a load deformation, pushover curve plotted in terms of acceleration and fundamental period. A structure responds elastically, up to a yield point, with a fixed fundamental period. Then as it responds inelasticity to greater accelerations, the structure softens and the period lengthens, forming the capacity spectrum curve.

As an acceptability check procedure, the constructed capacity spectrum curve is compared to a demand curve for the very rare earthquake for a 10% to 20% damped structure. If the capacity curve crosses the demand curve, the design is judged acceptable. If the curves do not cross, then the structure is unacceptable from a life-safety and collapse standpoint.

B. *Advantages of Procedure*

As compared to elastic methods, the Capacity Spectrum Method provides improved insight into the inelastic response of the structure. It estimates the displacement the structure will undergo for various levels of earthquake ground motion causing inelastic response.

C. *Disadvantages of Procedure*

The method is more cumbersome to use than the elastic analysis methods. The theoretical foundations of the method are open to question.

The method uses the elastic displacement response spectrum adjusted for higher effective damping to approximate the true inelastic response. The method may be unconservative for impulse type ground motions. The method cannot account for cyclic degradation of higher mode effects.

D. *Research Needs*

Further investigation into the following areas is necessary to improve the procedure:

- Reliability of representing inelastic response spectrum by using the elastic response spectrum with increased damping.
- Methods of applying the capacity spectrum procedure to each of the four performance levels, rather than just the Life Safe and Near Collapse Levels as is presently formulated.
- Methods of accounting for impulsive types of load earthquake ground motion.
- Methods for accounting for higher mode effects and cyclic degradation.

E. *References*

Freeman, S.A., Nicoletti, J.P., Matsumura, George, *Seismic Design Guidelines for Essential Buildings*, Proceedings, Eighth World Conference on Earthquake Engineering, San Francisco, 1984.

Mahaney, J.A., Paret, T.F., Kehoe, B.E. and Freeman, S.A., "The Capacity Spectrum Method for Evaluating Structural Response During the Loma Prieta Earthquake," 1993 National Earthquake Conference, Memphis, TN, 1993.

4.5 Pushover Analysis Methods

A. *Overview*

Pushover analyses are useful techniques to study the nonlinear response of a structure as various structural elements yield sequentially under incrementally increasing loads. A popular form is the static pushover analysis using incremental static loading in a non-varying (e.g., triangular) distribution which monitors the behavior of the structure at increasing displacement. There are several significant limitations to this method. The method does not account for loading redistribution due to progressive yielding at weak stories, nor higher modes or three-dimensional effects.

The acceptance criterion is based on the premise that the ultimate (elastic plus inelastic) displacements of a structure subjected to spectral demands can be related to the displacement obtained if the structure were to remain elastic. The elements first approaching yield are released in the model and the analysis is rerun with incremental loading. Stresses are summed for successive iterations until progressive yielding results in structural instability. The ductility demands are monitored in the yielding elements and checked against rotational or shear strain limits. If the structure reaches its target displacement without first becoming unstable or exceeding its ductility capacity, it is acceptable. If not, it is redesigned.

An enhanced procedure, which attempts to account for higher mode effects, uses a loading distribution is determined by spectral analysis. Separate spectral analyses are done at various "softened" states of the structure.

The procedure is illustrated in Appendix G of this report.

B. *Advantages of Procedure*

1. Where preliminary or final design is performed by spectral response, the same mathematical model and design spectrum can be used for the acceptability analysis.

2. The procedure uses concepts familiar to most engineers: spectral response and elastic analysis. It also enables the engineer to visualize the sequential yielding and provides insight for design modifications.

3. The allowable capacities or limit states for the various structural elements can be reduced or modified to reflect the effect of earthquake duration (i.e., number and amplitude of hysteretic cycles).

4. Although the procedure lacks the mathematical rigor of an inelastic time history analysis, it avoids some of the sensitivity of such an analysis to the time history or complex cyclic behavior assumed for the key components.

C. *Disadvantages of Procedure*

1. Since the limit states for seismic loading are dependent on the available capacity under various load combinations, considerable "bookkeeping" is required to identify the post-yield limits for the structural elements.

2. The available data on allowable limit states for various structural elements and connections is very limited and is generally based on laboratory testing that may not be representative of the most severe earthquake response.

D. *Research Needs*

- Research data on the limit states for various structural elements and connections. The data should reflect the effects of various types of earthquake loadings and durations.
- Development of computer programs to assist in monitoring the iterative responses and identifying limit states.
- Development of improved nonlinear pushover analysis methods approach, with particular study regarding target displacements and incremental displacement methods.

E. *References*

Newmark, N.M., and Hall, W.V., "Earthquake Spectra and Design", Monograph series by Earthquake Engineering Research Institute, Berkeley, CA, 1981.

Krawinker, H., "Static Pushover Analysis", *Proceedings: SEAONC 1994 Fall Seminar on the Developing Art of Seismic Engineering*, San Francisco, CA, 1994.

Lawson, R.S., Vance, V. and Krawinkler, H., *Nonlinear Static Push-Over Analysis - Why, When , and How?*, Proceedings of the 5th U.S. Conference in Earthquake Engineering, Chicago, Vol. 1, p. 283-292, 1994.

4.6 Dynamic Nonlinear Time History Analysis

A. *Overview*

The Dynamic Nonlinear Time History Analysis procedure is the most sophisticated analysis technique currently available for modeling an entire building system and investigating its dynamic response. The procedure involves a step-by-step solution through the time domain for the dynamic response of a structure to a selected ground motion time history. The structural model must reflect the elastic and post elastic behavior of the materials and should include significant nonstructural component participation and soil-structure interaction. The analysis provides very detailed information on the structural response, accounting for irregularities, nonlinear behavior and modal interactions but the results must be used advisedly due to the many uncertainties inherent in the modeling as well as the ground motion representations.

The analysis procedure requires the use of sophisticated 3-D nonlinear computer analysis programs, which are only now becoming readily available. Its primary use has been in research applications and for verification of simpler analysis techniques. It can be applied to any structure that can be adequately modeled, however, at the present time its use may be practical only for important and simple structures. It may eventually be very effective as an acceptability analysis tool for checking response parameters against limiting values for the performance objectives. A more comprehensive discussion of both linear and nonlinear dynamic time history analysis is included in Appendix H.

B. *Advantages of Approach*

Dynamic Nonlinear Time History Analysis is the most powerful tool currently available for checking the response parameters and the acceptability of a design. It can be used to check all of the key response parameters discussed in Section 2.6.

C. *Disadvantages of Approach*

The procedure is relatively cumbersome and expensive too use and requires very comprehensive and realistic modeling to be accurate. Its power and numerical precision may give a false sense of confidence in the analysis results. Small changes in modeling assumptions or assumed ground motion time history input can drastically effect results. In general, this procedure has not been able to predict the damage observed in past earthquakes.

D. *Research Needs*

- Improved modeling techniques for nonlinear behavior, for participation of nonstructural components and for soil-structure interaction.
- Improved representations of design ground motions.

4.7 Drift Demand Spectrum Analysis

A. *Overview*

The near-field ground motion effects in Northridge and other recent earthquakes have demonstrated that structural demands can be significantly different in the near field than in more remote areas. While the response spectrum for the near-field in Northridge is generally similar in shape to the standard spectrum, some of the time histories differ significantly; the near-field time histories are characterized by potentially damaging, distinct long amplitude velocity pulses and high free field displacements. The effect on structures may be very high inter-story drift demand, often at the lower stories. The overall amplitude of the structural response is fairly well predicted by the elastic response spectra, but local story drift may be much higher than predicted by conventional techniques (on the order of 5% story drift for low damped structures in Northridge). It has been suggested that this near-field phenomenon may be able to be represented by a new measure, the drift demand spectrum (Iwan-1994). With this method, the story drift is plotted as a function of the height or number of stories for different levels of damping based on parametric analysis studies.

The drift demand spectrum can be used to determine damping requirements, drift demand and building separation requirements for standard structures or drift capacity requirements for base isolators in isolated structures.

B. *Advantages of Procedure*

The drift demand spectrum represents special near-field ground motion demands and provides a method for designing and checking structures for near field effects. It specifically enables a check of drift and related ductility demand parameters. It may be simple to use in combination with traditional response spectra and associated analysis techniques.

C. *Disadvantages of Procedure*

The approach is in its initial development stages and requires additional research to be substantiated and improved. To date, there is no suggestion of how it can be calibrated to fit into the performance based engineering procedures.

D. *References*

Iwan, W.D., *Near-field Considerations in Specification of Seismic Design Motions for Structures*, Proceedings of the 10th European Conference on Earthquake Engineering, Vienna, Austria, 1994.

5. CONCLUSIONS AND RECOMMENDATIONS

The Performance Based Engineering (PBE) methodology reported herein proceeds from the recognition that a need exists to reformulate seismic design guidelines so as to enable achievement of predictable seismic performance and to restate the guidelines so that the fundamental seismic design concepts and processes are transparently clear.

It is the conclusion of this report that the Performance Based Engineering development should proceed from the following premises:

1. PBE can be developed within the conceptual framework articulated in this report to enable the achievement of predictable seismic performance while maintaining clarity of the seismic design concepts and processes.

2. PBE must address the full range of issues in the framework, not just seismic design of the structure but particularly seismic design of the nonstructural components and quality assurance in construction, as well as monitoring the maintenance and occupancy of the building during its life.

3. The capacity design philosophy should be adopted as an underlying principle of good seismic design.

4. PBE must directly address the inelastic response of structures to accomplish its objectives.

5. PBE must consider more than one level of earthquake ground motion in design, in order to accomplish higher performance objectives.

6. PBE must permit a range of design approaches to address the range of design problems and objectives.

7. PBE must identify the sources of uncertainty in the seismic design process and quantify the risk and reliability in the PBE design.

Further, in order to implement PBE, it is the recommendation of this report that the following steps be taken toward the development of Performance Based Engineering guidelines and codes:

1. The next phase of the PBE development initiative should proceed by using the conceptual framework presented in this report to develop performance based seismic engineering guidelines.

2. The design approaches and acceptability procedures proposed herein, and promising alternate methods, should be developed and, in some cases, refined.

3. The capacity design procedures for various systems and materials should be developed.

4. The Displacement Based Design Approach should be further researched and developed.

5. Design method based on energy concepts should be researched and developed.

6. Use of passive energy dissipation devices in structural systems should be developed and encouraged.

7. Guideline writers and code developers should hold as a guiding principle that seismic design concepts and procedures must be kept clear and understandable to be effective.

8. Directed research and development should be encouraged into the issues identified in the "Research Needs" sections of this report.

9. The education process for engineers, architects, academics, building officials, the construction industry and the public should begin forthwith.

10. The transition process of adapting current practice and current codes to PBE by incremental introduction of PBE concepts and procedures should begin immediately.

If this document contributes to the advancement of seismic engineering practice and accelerates the process of development of effective Performance Based Seismic Engineering, then its purposes will be served.

APPENDICES

APPENDIX A

CRITICAL REVIEW OF AVAILABLE DESIGN PHILOSOPHIES AND APPROACHES

by

Vitelmo V. Bertero

A.1 Available Design Philosophies and Approaches

The following main design philosophies and design approaches have been suggested and used in practice.

Linear Elastic Design Philosophy. The two following approaches, which have been developed and used in present codes and therefore in practice, are based on the assumption that the structure will behave in its linear elastic range.

- *Allowable-Stress or Working Stress or Service Stress Design Approach.* According to this approach, the structure and its members are designed to support (resist) the working or service loads without exceeding certain specified allowable stresses.
- *Strength Design Approach.* Although this approach is based on the assumption of linear elastic behavior of the structure, the structural members are designed on the basis of their critical section yielding strength capacity when subjected to factored service or working loads (excitations).

Limit Design or Plastic Design or Collapse Design Philosophy. According to this philosophy, the structure is designed on the basis of its collapse strength against properly factored service or working loads (excitations).

- *Rigid-Plastic Design Approach.* This approach is based on the assumption that the structure's members have sufficient ductility to allow the development of a collapse mechanism ignoring the elastic deformations.
- *Elastic-Plastic Design Approach.* In this approach, the elastic and plastic deformations are considered in the redistribution of the internal forces due to the plastic deformations.

Serviceability Limit States Philosophy. This philosophy considers the performance of the structure under the normal service or working excitations as in the case of allowable stress approach, but is also concerned with the other requirements besides the stresses for ensuring a satisfactory use of the building according to its occupancy, including the possible consequences of excessive deformations and/or deformation rates and vibrations on the persons occupying the building, and on the nonstructural components (such as cracking, slipping etc.).

Note that the limit state defines a deformation condition of the structure at which it or any part of it ceases to perform its intended function. Thus, limit states are usually called limits of usefulness.

Strength Limit States Philosophy. The design based on this philosophy will consider the safety or load-carrying capacity of the building when it is subjected to the critical combinations of the factored service loads, including not only the plastic strength of the members but also the effects of normal classic fatigue, low cyclic fatigue, incremental collapse, fracture etc. The LRFD approach for steel structures has been developed based on this philosophy.

Comprehensive Design Philosophy. In Reference 1, in a discussion of the problem of strength and deformation capacities of buildings under extreme environments, it is pointed out that the possible occurrence of a severe event, such as an earthquake, wind, flood or fire poses special problems when designing new buildings or evaluating the adequacy of existing buildings. There is the understandable desire to construct a building that can withstand the worst event without damage, but the combination of the expense that this would entail and the low probability of such an event suggests less rigid performance criteria. The question is whether is it necessary only to prevent collapse and subsequent loss of life, or whether the expense of damage should be limited as well. A solution is offered by the philosophy of comprehensive design.

In Ref. 2, Sawyer [1964] discusses a *comprehensive design procedure that correlates the resistance of a structure at various failure stages (limit states or limits of usefulness) to the probability that possible disturbances can reach the intensity required to induce such failure stages, so that the total cost (including the first cost and the expected losses from all the limit stages) is minimized.* As illustrated in Fig. A-1, under increasing loads structures generally fail at successively more severe failure stages with increasingly less probability that the load will reach the required intensity levels. The relationship shown in Fig. A-1 gives possible structure failure states versus a monotonically increasing pseudo-static load for a typical statically indeterminate reinforced concrete building [2].

Owing to the variability of loss for a given load (or the variability of load for a given loss), this relationship represents the mean values of the random variables involved. The full distribution, as shown in Fig. A-2, can sometimes involve large variances [3]. This should be clearly understood by the analysts and designers in order that they will not put all their trust in numerical results obtained via just one deterministic analysis, no matter how sophisticated a computer program is used.

In comprehensive design, identification of potential failure modes requires the prediction of the mechanical behavior of the entire building system at each significant level of critical combinations of all possible excitations to which the building may be subjected. For severe environmental conditions, current design methods require that the strength and deformation capacity of a building be established.

The major threat to life and loss of property under natural hazards stems from existing buildings. To resolve the problem of hazard abatement, existing building must be continuously assessed for danger under extreme environmental conditions, necessitating comprehensive analyses of the buildings to predict their strength and deformation capacities.

From the above brief discussion of the above existing design philosophies, it appears that for the purpose of EQ-RD and particularly for achieving efficient performance-based EQ-RD, the comprehensive design philosophy is the most attractive one. As is shown in the next sections, not only does the worldwide-accepted general philosophy of EQ-RD agree completely with the comprehensive EQ-RD philosophy, but also it covers the more detailed performance design objectives that have been included in the definition of all performance-based design that has been adopted by the Vision 2000 Committee.

A.2 General Philosophy of EQ-RD

The general philosophy of EQ-RD for nonessential facilities has been well established and accepted worldwide, and it proposes to prevent:

1. structural and nonstructural damage in frequent, minor earthquake ground shaking;
2. structural damage and minimization of nonstructural damage during occasional, moderate earthquake ground shaking; and
3. collapse or serious damage in rare major earthquake ground-shaking.

This general philosophy qualitatively agrees with the comprehensive design concept or philosophy but, as is discussed below, present practical applications of this general philosophy fall short of realizing its objectives, mainly because it does not define specifically (quantitatively) the earthquake ground-shaking (EQGMs) or the degree of damage that has to be prevented. To remedy this, the Vision 2000 has adopted the following more detailed and specific definition of what is understood for performance-based design.

A.3 Definition of Performance Based Design

"Performance based seismic design consists of the selection of appropriate systems, layout, proportioning and detailing of a structure and its nonstructural components and contents such that at specified levels of ground motion and with defined levels of reliability, the structure would not be damaged beyond certain limiting states."

At any particular earthquake demand level, a given structure will respond within a particular damage state. An infinite spectrum of limiting damage states, ranging from no damage to complete collapse, exists. For purposes of performance based design, four specific limiting damage states, or *performance levels*, are defined. Performance levels are defined based on consequences to building owners, occupants and other interest groups, of the building reaching that state.

The performance level is by itself independent of the seismic hazard; however, when coupled with a specific ground motion criterion, it becomes a *performance design objective*. Typically, a project should be designed for a spectrum of seismic design objectives, ranging from no damage for earthquake ground-shaking which is likely to affect the building relatively frequently to avoiding collapse for infrequent extreme events. The Vision 2000 Committee has defined four performance levels: *Fully Operational, Operational, Life Safe, and Near Collapse*. Furthermore, the committee has defined as a minimum *three standard design objectives*.

From the above definitions and discussion, it is clear that earthquake-resistant design must involve consideration of serviceability and strength limit states and should include the cost of losses. Furthermore, in view of the uncertainties involved in defining the levels of the EQGMs as well as predicting the real mechanical behavior, it will be necessary to use a probabilistic approach in such a type of design. Thus, it becomes clear that among the different existing design philosophies the ideal one for performance-based design is the comprehensive design philosophy.

A.4 Current Seismic EQ-RD Approaches: A Critical Review from the Point of View of the Accepted EQ-RD Philosophy on which Performance Based EQ-RD Approaches should be Based

When Vision 2000 began to develop the work plan to produce design and construction standards that will yield buildings with predictable EQ performance, one of the first questions and problems that needed to be tackled was whether it will be possible for performance-based EQ-RD to be achieved using the present seismic codes' EQ-RD approaches. To answer this question, it is necessary to review first what is needed to be able to produce EQ-RD and construction of predictable performance.

Information Needed for Conducting Performance-Based EQ-RD of Buildings. As briefly discussed above in Sections A.1 and A.2, the ideal design philosophy for conducting a performance-based design seems to be the comprehensive design philosophy. According to this philosophy, *the ideal design is that which results in minimum total cost of the facility, which includes not only the cost of construction but the cost of all possible losses at all the possible limit states that the facility can reach or be subjected to during its service life and the repair and/or upgrading work, as well as the cost of its demolition*. Thus, it is clear that the comprehensive design philosophy and consequently the performance-based design is based on the use of the so-called limit state design philosophy, but goes beyond this philosophy. The comprehensive design philosophy that recognizes the uncertainties involved in defining each of the different excitations to which a facility can be subjected, and particularly their critical combinations (i.e., the design excitations) at each of the different limit states as well as the uncertainties in defining the engineering parameters controlling the mechanical behavior of the facility, is a *Probabilistically Formulated Limit State Design Philosophy*.

To summarize: it can be stated that to conduct performance-based design, we need to apply the comprehensive design philosophy, which is a probabilistically formulated Limit State Design philosophy, and the use of this philosophy requires the following information:

- The different sources of excitations (loads) to which the facility to be designed can be subjected during its service life.
- Definition of the limit states (performance levels) that need to be considered.
- The variation in the intensity of each of the excitations that can act on the facility during its service life and the probability that the combinations of these excitations can reach the required intensity to induce each of the limit states (failure stages) that need to be considered.
- The types of failures (limit states) of the different components, structural and nonstructural, of the entire facility system, associated with the types of excitation and the increasingly small probability that the excitations will reach the intensity levels required to induce such failures.
- The costs of the losses and repairs associated with each of the different limit states (failure stages) that need to be considered.

Thus, as is the case with any type of engineering design, the most important information for performance-based design is that concerning the sources of excitations, their variation in intensity and their corresponding probability of reaching the intensity required to induce any of the limit states that have to be considered. Once this information is available, the owner, together with the designer has to decide on the performance levels (limit states) that should be considered in the design together with the recurrence periods over which such levels are reached in accordance with the controlling excitations at these levels. For example, the owner may desire a design and construction that will perform as stated in the table in Fig. A-3.

According to the expected intensity and duration of the EQGM excitations and the combination of other significant potential seismic hazards and loading conditions in the owner-desired recurrence period, the designer has to analyze if it will be economically feasible to design for such requirements and to offer alternative recurrence periods for the different limit states. Assume that a compromise is reached on the recurrence period of the limit states indicated in Fig. A-3. Once this has been reached, the design criteria in the form of performance-based design objectives has been established, the next step is to conduct the necessary analysis and preliminary design to comply with the established design criteria.

The necessary analysis and preliminary designs are commonly based on idealized mechanical behavior under simplified excitations because it is not usually possible to consider actual behavior and the true history of disturbances. Sources, treatment and effects of the different types of excitations are summarized in Fig. A-4. Structures are usually subjected to unpredictable fluctuations in the magnitude, direction and/or position of each of the individual excitations that may act on them during their service life, and the extreme values between which each of these excitations will oscillate are the only characteristics that can be estimated accurately.

These types of action are classified in Fig. A-4 as generalized or variable-repeated excitations.

The types of failures associated with variable-repeated excitations are classified as long-endurance fatigue, low-cycle fatigue, and incremental collapse. Long-endurance fatigue is only critical for very special structures. Regarding low-cycle fatigue, review of results from experimental data shows that the real danger associated with repeated reversed inelastic actions (alternating excitations) is not only material fracture, as is usually reported in the literature, but deterioration of stiffness, particularly in reinforced concrete and masonry. Incremental collapse is related to the progressive development of excessive deflections which occur under the cyclic applications of different combinations of peak actions. Because deterioration of stiffness can lead to an undesirable increase in deformations, the effects of alternating excitations cannot be treated independently as is usually done, from those caused by excitation patterns leading to incremental deformations.

Failure prediction is clearly essential in designing against extreme environments and requires knowledge of the strength at different levels of structural deformation. The discussion above points out the difficulty in predicting strength and deformation capacities and the need for a probabilistic approach, or, at least, for considering the range of probable mechanical behavior and possible excitations.

Based on the possible different types (dynamic characteristics) of EQGMs, such as impulsive or harmonic, and on the relative intensity of the other types of excitations, such as the different types of gravity loads that can be acting simultaneously with the expected future EQGMs, Reference 4 gives a detailed discussion of what is needed regarding the EQGMs, as well as the prediction of the mechanical behavior (dynamic response) up to each of the limit states involved in the desired performance design objectives of a performance-based EQ-RD. This is done considering the different types of failures illustrated in Fig. A-4. From this information, it is clear that except for the rare case that it is desired to keep the entire building serviceable under even the maximum credible EQGM that can occur during the service life of the building in which case linear elastic analysis can be used in all the other cases, particularly in cases in which the performance levels (limit states) of life safety and/or impending collapse (collapse prevention or stability) are permitted, estimation of the damage potential of the EQGMs and preliminary design and analysis of the entire building demands the use of approaches and procedures based on nonlinear dynamic analyses. Regarding the expected EQGMs, the essential information needed is: *the time history of the expected EQGMs at the different recurrence periods of the performance levels that should be considered.* Note that, because of the uncertainties in predicting such EQGMs, it is necessary to specify for each recurrence period a suite of EQGM time histories. With this information, engineers can compute the specific detailed information needed to conduct the preliminary EQ-RD and the needed acceptability analyses. As discussed in detail in Reference 4, the specific information to be obtained from the processing of the time history of the EQGMs at each of the recurrence periods that need to be considered are the following spectra.

The Smoothed Inelastic Design Response Spectra (SIDRS) for:

- strength,
- total acceleration,
- velocity,
- displacement,
- energy input, and
- energy dissipation

corresponding to the predicted or established suite of EQGMs. These spectra have to be computed considering the different levels of ductility ratio, μ, that can be accepted according to the desired performance (damage) at the recurrence period under consideration. These spectra should include as a particular case the Smoothed Linear Elastic Design Response Spectra (SLEDRS) which is for μ=1.

How these spectra are used in a comprehensive EQ-RD approach to performance-based EQ-RD is summarized in Appendix B.

Critical Review of Current Code EQ-RD Approaches. As discussed in detail in Reference 4, the current code EQ-RD approaches for most buildings are based on the use of a strength (base shear) SLEDRS for just one performance level, the Life Safety level corresponding to a return period of 475 years. Although technically it may be justified and it is understandably desirable to design and construct a building that can resist such a severe level of EQGM or even the worst expected event, i.e., the maximum credible EQGM (the one possessing the highest damage potential to the building) without any damage, the combination of the low probability that such an event will occur and the high cost of construction suggests less rigid performance design objectives or design criteria. Taking advantage of the dissipation of energy that takes place in ductile structures, the codes have introduced an EQ-RD approach that reduces the demanded linear elastic strength (base shear) through a reduction factor called the response modification factor, R, by the NEHRP recommendations, or the structural system factor, R_w, by the SEAOC blue book. To keep code design procedure as simple as possible using only linear elastic methods of analysis (which allows use of the principle of superposition), the codes base their design on either the allowable or service stress approach (UBC), or the first significant yielding of the most stressed section (NEHRP). There is no doubt that this is the simplest approach, except for the case of buildings where prescriptive EQ-RD can be used. However, as shown in Reference 4, blind use of the current linear elastic code approaches, the so-called static equivalent lateral force (ELF) and even the linear dynamic response spectrum, which are based on the use of R factors whose values depend only on the type of structural system, independent of the period of the structure and of the type (dynamic characteristics) of the EQGMs and of the relative importance of the other types of excitations (loads) that can act simultaneously with the effects of the EQGMs, can result in designs that can have quite different performance. As shown in Reference 4, the current ELF approach cannot lead to design with desirable performance in the following cases.

- The critical EQGMs are of an impulsive type and consist of just one important relatively long duration pulse, say, $t_d/T > 0.5$ for the case of a nearly rectangular pulse, where T_d is the duration of the pulse and T is the fundamental period of the structure.
- The critical EQGMs consist of just several (two or more) pulses of relatively long duration which can lead to a failure due to incremental collapse load.
- The critical EQGMs are of harmonic type, with a predominant period, T_g, larger than the T of the structure. In this case, the larger the T_g is with respect to the T, the larger will be the actual inelastic deformation when compared with the deformation that will be estimated on the basis of just a linear elastic behavior.
- The critical EQGMs are of harmonic type and the effects of gravity forces are relatively important when compared with the effects of the EQGMs. In this case, the ELF, if the nonlinear pushover approach is used as well (as is discussed in Reference 4) cannot give any idea that a moving plastic hinge creating a plastic region around the midspan of the beams of a moment resisting frame (MRF) can be developed. Furthermore, and perhaps one of the main weaknesses of the code ELF approach using the values of R currently specified in the code is that these values of R are related to the first significant yielding of the structure without considering the actual overstrength of the entire structure, which varies significantly with the variation in the relative importance of the other loads that can act simultaneously with the EQGMs and with the degree of redundancy in the lateral-resistant system.

Another main weakness of the ELF approach, as well as any other of the approaches of current code EQ-RD approaches, is that they are based on just one performance design objective: the life safety. Although the code requires that the deformations (lateral interstory drift indices) resulting from elastic analysis under the design lateral forces be checked against the specified values of what are considered the maximum allowable lateral drift values under working load conditions, and that these deformations, modified by a factor that the NEHRP recommendations called the deflection amplification factor, C_d, and by a factor $3(R_w/8)$ in the 1994 UBC code regulations have to be equal to or smaller than specified maximum acceptable values, it has to be noted that: first, the deformations obtained from the elastic analysis under the reduced forces for the life safety EQGMs with a return period of 475 years do not in general represent the deformations that can be expected under the service EQGMs, i.e., the EQGMs that can occur with a lower return period; and secondly that these elastic deformations amplified by the specified or recommended deflection amplification factor in general do not result in a reliable prediction of the actual inelastic deformation that will occur under the critical EQGMs with a return period of 475 years.

In judging the different code EQ-RD approaches that are based on designing for strength (base shear) as well as for those new approaches that have been suggested as promising for performance-based EQ-RD and are also based on just using strength as the main design criterion or parameter, the following should be kept in mind: first, that the defined performance levels are based on different degrees of acceptable damage and damage is more a consequence of deformation than strength, and secondly that for the performance levels accepting damage through the yielding of the structure as a mechanism, the use of the yielding strength as a design parameter is completely insensitive to the amount of deformation, i.e., to the amount of damage and therefore cannot be used alone to conduct the required performance-based EQ-RD.

From the above discussion, it is obvious that the current code ELF EQ-RD approach, which is based on the use of specified linear elastic strength spectrum and of specified values for the R factor cannot result in general in design of building with predictable performance. Because this ELF approach is the most used approach for EQ-RD of buildings, it would be highly desirable first to investigate the kind of buildings to which it can be applied to achieve predictable performance of a building. Secondly, to investigate what simple modifications can be introduced to extend the applications of this ELF approach to the performance-based EQ-RD of other types of buildings. There is no doubt that this is one of the most challenging works for the practical implementation of performance-based EQ-RD. Thus, attempts are made in Appendix E to develop the needed modifications to the existing ELF approach so it can be used in performance-based EQ-RD for a larger number of building types.

A.4 References

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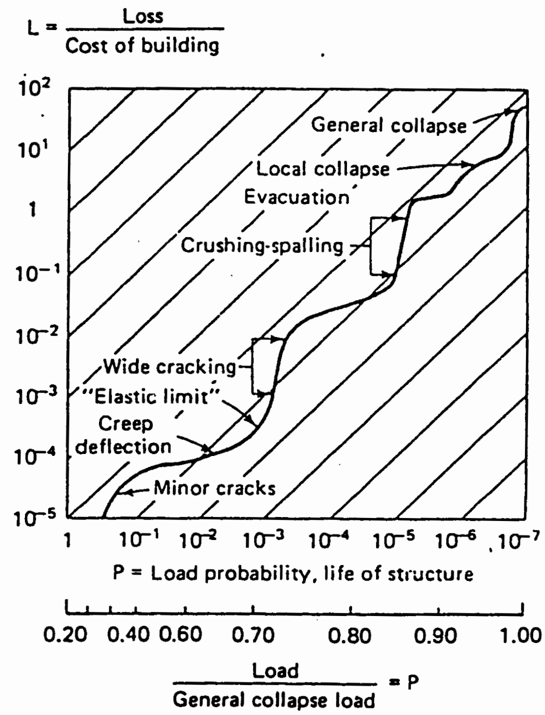

Figure A-1: Losses versus the probabilities of occurrence of increasing loads during the service life of an RC structure [Sawyer 1964]

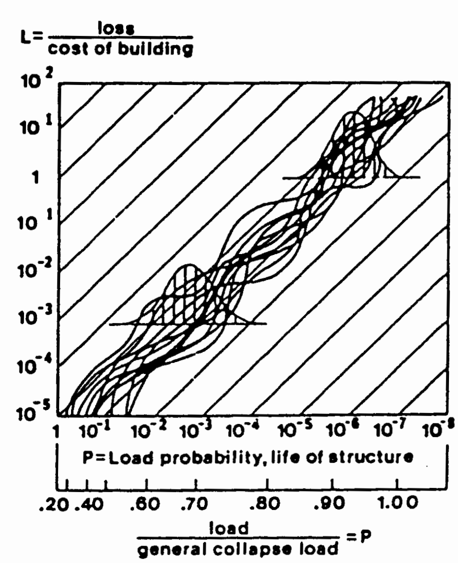

Figure A-2: Distribution of losses versus load probabilities during the service life of an RC structure [Sawyer 1964].

Performance Levels	No Damage	Damage Control			
		No struct. damage, minor nonstruct. damage	Minor struct. damage, moderate nonstruct. damage	Life safety and economic repairability	Life safety but no economic repairability
Limit States	Service	Continuous Operation	Immediate Occupancy	Life Safety	Impending Collapse
Owner desired recurrence period*	10 years	30 years	50 years	450 years	900 years
Compromise recurrence period	8 years	20 years	40 years	450 years	700 years

* note that the values of recurrence periods given herein are arbitrarily selected to illustrate the procedure

Figure A-3: Initial selection of performance levels

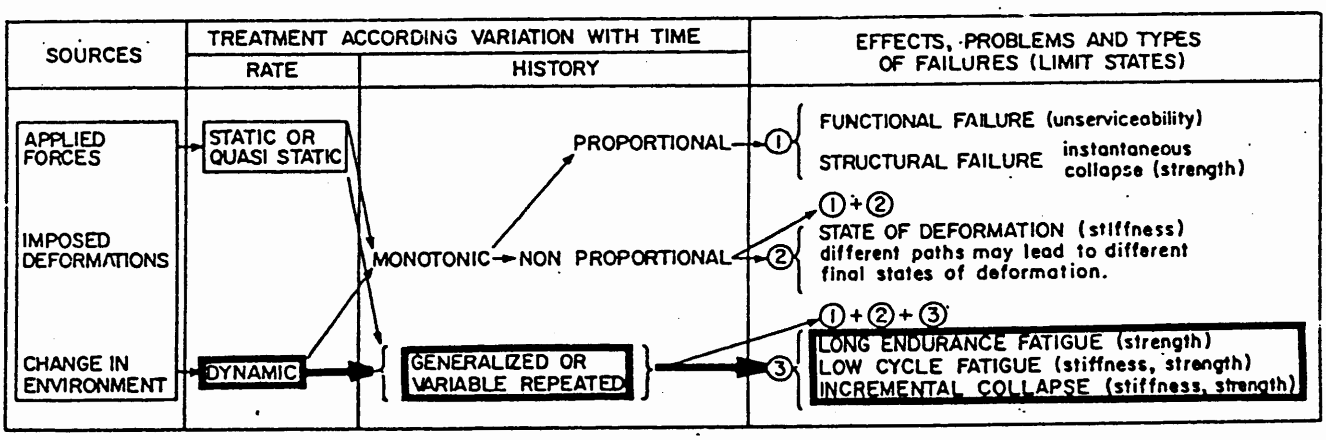

Figure A-4: Sources, treatment and effects of excitations on structures [Bertero, 1971]

APPENDIX B

COMPREHENSIVE EARTHQUAKE-RESISTANT DESIGN (EQ-RD) APPROACH

by

Vitelmo V. Bertero

B.1 Overview of Comprehensive Design Concept

A. *Introductory Remarks*

The comprehensive approach for performance-based EQ-RD forms part of a proposed new conceptual code format. The new format aims at developing regulations that cover in a rational and transparent way all that a seismic code should regulate to ensure the desired building performance when significant EQGMs occur. The conceptual methodology for this proposed comprehensive approach is in compliance with the worldwide-accepted EQ-RD philosophy for standard occupancy structures and with the definition of performance-based design, because it considers all the minimum performance design objectives. It considers also the minimum requirements for special (emergency response) and hazardous facilities as well as for special occupancy structures. This conceptual approach is based on the use of energy concepts and on fundamental principles of structural dynamics and comprehensive design, and it considers the realistic mechanical behavior of the entire building system. It takes into account from the start of the design process the simultaneous demands for strength, deformation (including torsional effects) and their combined effects on the demand and supplied energy capacities of the entire building system. This comprehensive approach recognizes that to have a good performance of a building when subjected to EQGMs, a good design is necessary but not sufficient. What is needed is a good Earthquake-Resistant Construction (EQ-RC) at the moment that the EQGM occurs.

B. *Design aspects considered in the formulation of conceptual methodology for comprehensive EQ-RD of a structure*

The proposed methodology for EQ-RD of new structures covers the following aspects:

- conceptual establishment of the design criteria (performance design objectives) according to the desired function (occupancy) of the facility;
- conceptual establishment of the design EQGMs, as well as any other potential hazard to be considered in the design, according to the selected site;
- formulation of the design methodology;

- conceptual overall design of the entire facility system;
- conceptual numerical preliminary design of the facility system;
- analysis of probable performance of preliminary design when subjected to the critical hazards (EQGMs) corresponding to each of the desired performance level;
- final design (detailing); and Comprehensive EQ-RD: Iterative Procedure
- acceptability of final design.

Furthermore, to ensure that the building is built as designed and then properly maintained during its service life, the proposed comprehensive approach also covers the following aspects:

- quality control, and
- monitoring of maintenance and function (occupancy).

C. *Application and Advantages*

Although the proposed approach can be applied for all kinds of structures, its main use will be for carrying out studies to develop simplified practical methods of performance-based EQ-RD of certain types of facilities such as low-rise buildings having relatively regular structural configurations and systems and constructed on normal sites. This comprehensive methodology should be used to calibrate simpler practical performance-based EQ-RD approaches. Therefore, in practice, the application of this comprehensive approach will be limited to structures located on abnormal sites and/or complex structures such as buildings with irregular configuration, structural layout and/or structural systems; and/or very tall slender buildings located on normal or abnormal sites. The detailed flow chart in Fig. B-1 illustrates the comprehensive EQ-RD approach. The importance of this detailed chart is that it can be used for the development of any other simplified performance-based EQ-RD approach.

B.2 Site Suitability Analysis

As indicated in the flow chart for EQ-RD and EQ-RC (Fig. B-1) the first step in the comprehensive EQ-RD approach is to conduct a *suitability analysis of the selected building site*. To conduct such analysis it is necessary to obtain the following information and make the following identifications.

- *Site location and site conditions (soil profile and topography)*. For the selected building site, besides the normal types of loads (excitations), possible sources of EQs and any other extreme environment should be identified.

The soil profile and topography should be identified and the influence of these site conditions on the seismic activity (particularly on the time history of the EQGMs) at the site should be assessed.

- *Identification of the seismic activity at the region and site of the facility: locations of the EQ sources, and EQ mechanisms.* In order to assess the seismic activity at the site, all possible sources of EQ hazards should be identified, with the recognition that the distance from the site of these sources, which can induce significant seismic activity at the site, varies according to the EQ mechanism, regional and local geology and site conditions. Reliable mapping of possible sources of EQGMs (such as active faults) is needed.

- *Information of sources of potential seismic hazards at the site and their damage potential.* The sources of potential seismic hazards should be evaluated according to the site conditions and the built environment around the site. There is a need for more reliable microzonation of the area of the site that is at present available.

- *Identification of the seismic hazards that may control EQ-RD at the different desired levels of building performance: EQGMs, soil failures (liquefaction, landslides, etc.), tsunamis, floods, fires and other hazards that may control the design.* While usually the EQ-RD of buildings aims only at resistance against the vibratory response to the EQGMs, in many cases other sources of damage may control the design. It has to be recognized that the seismic hazard controlling the EQ-RD for serviceability may be quite different from that controlling the design for safety. For example, liquefaction and landslides may not be a problem for frequent minor EQGMs, but they might control the design for major EQGMs (i.e., safety). Reliable maps identifying the probabilities of different hazards are needed.

- *Site suitability analysis.* Based on the information obtained regarding the seismic hazards that may control the EQ-RD of the building, a decision has to be made regarding the suitability of the selected site.

B.3 Selection of Performance Levels and Performance Design Objectives

As illustrated in the flow chart of Fig. B-1, a suitability analysis of the selected site should be conducted according to the desired occupancy and function of the building. If the given site is suitable, the next step in the design process is to decide about the performance design objectives. To do this, the designer should discuss with the client the severity of the expected potential sources of seismic hazard (levels of EQGMs as well as any other type of seismic hazards) and their corresponding frequency (return period) as well as the number of discrete performance levels that should be considered.

A. *Performance levels*

According to the adopted definition of performance-based design, the following minimum performance levels should be considered initially:

- Fully Operational
- Operational
- Life Safety
- Near Collapse

As discussed in Appendix A and illustrated in Fig. A-3, after the owner expresses the desired return or recurrence period of each of the limit states associated with these performance levels, compromise should be reached about such selection depending on the severity of the expected EQGMs at such recurrence periods and the cost of designing for such levels and the probable losses during the life of the structure. Once an agreement has been reached on the performance levels and their respective recurrence period, the designer has to establish the corresponding minimum performance design objectives.

B. *Minimum performance design objectives*

The designer should explain to the owner that the following are the minimum design objectives, as well as the initial cost of construction that will demand such design and the probable cost of the losses if the limit states associated with the selected performance levels are reached.

- Resist minor EQGMs, which can occur frequently, without damage (service or fully operational limit state).

- Resist moderate EQGMs, which can occur occasionally with controlled structural and nonstructural damage (amount of damage depends on function of facility) (functional or operational limit state).

- Resist expected major EQGMs, which can occur rarely, with controlled damage that cannot endanger the life safety of its occupants or those of adjacent facilities (safety limit state).

- Resist extreme EQGMs, which can occur very rarely but are probable, with damage up to impending collapse but without collapsing and endangering the lives of its occupants or inducing damage to surrounding facilities (impending collapse limit state).

Note that the owner might elect to associate the desired performance level (degree of damage) with more severe EQGMs than those corresponding to the minimum levels, as is illustrated in the matrix of Fig. B-2.

C. *Acceptable performance design objectives*

See matrix on acceptable design objectives (Fig. B-2), where the return periods, T_r, for EQGM level, have been arbitrarily selected for the San Francisco Bay Area. It has to be clearly noted that the dynamic characteristics of the EQGMs and the values of their return periods for the different levels of EQGMs will vary according to the seismic activity in the area of the site of building.

Performance design objectives on which preliminary and final designs are based. Although the ideal would be to carry out the numerical EQ-RD considering simultaneously the demands imposed by all the performance design objectives in most of the cases this would not be necessary because there will be only a few of these objectives that will dominate the design. In any case, it is highly recommended that the preliminary EQ-RD to be conducted considering at least two of these objectives which usually will be those corresponding to the following performance levels: service (or functional) and life safety (or impending collapse).

For the final EQ-RD, if the design has been based on the four minimum performance design, compliance with the requirements dictated by the four limit states under the corresponding four levels of EQGMs associated with these four design objectives (service, functional, life safety and impending collapse) should be checked.

B.4 Conceptual Overall Seismic Design of the Entire Building System

Conceptual overall seismic design is the avoidance or minimization of problems created by effects of seismic excitations, using understanding of behavior rather than numerical computations. Conceptual overall design of the facility system involves not only the choice of overall shape and size of the building, but also the selection of the structural layout, the structural system, the structural material, the type of nonstructural components (particularly those that could become unintentional structural components), and the foundation system. Both the architect and the engineer have to understand how design decisions regarding building layout may have serious seismic effects on the structure. The inertial forces depend on the mass (amount and distribution), the damping, and the structural characteristics (stiffness, yielding strength, maximum strength and energy absorption and energy dissipation capacities). Although there is not universal ideal building and structural configuration, certain basic principles of EQ-RD can be used as guidelines to select adequate building and structural configuration, as well as efficient structural foundation types and systems, nonstructural components and their respective materials.

A. *Guidelines for overall seismic design of entire building system*

- Building and structure should be light weight (avoid unnecessary masses); if mass is needed, such as the use of the so-called architectural components, use it to improve EQ response.

- Building and structure should be simple, symmetrical and regular in plan and height to prevent significant torsional forces; avoid a large height/width ratio.

- The vertical elements of the building should be properly tied together at the foundation and at the top of their openings along the different stories, as well as to the horizontal elements (diaphragms) at each floor level and roof.

- Structure should have the largest possible redundancy against translational and torsional response. Redundancy and ductility are the blessings of EQ-RD that permit the overcoming of the many uncertainties in numerical EQ-RD.

- Structure should have sufficient initial stiffness to avoid significant damage of nonstructural components and contents under minor and moderate earthquake shaking. The stiffer the structure, the less sensitive it will be to effects of interacting nonstructural components. However, for buildings with nonstructural components, equipment and contents whose damages are sensitive to total acceleration, the stiffness should be limited by the acceptable damage to these components and contents.

- The structure should have sufficient toughness (energy dissipation capacity) with stability of strength and stiffness under repeated reversals. Furthermore, the tougher the structure is the less sensitive it will be to effects of sudden failure of interacting nonstructural components.

- Structure should have a uniform and continuous distribution of strength, stiffness, ductility and energy dissipation capacity throughout.

- Structure should have relatively shorter spans than buildings that do not need to be designed to be EQ-resistant, and the use of long cantilevers should be avoided.

- Structure should have the largest possible number of structural defense lines, i.e., it should be composed of different ductile structural subsystems that interact or are interconnected by very ductile and tough structural elements (structural fuses) whose inelastic behavior would permit the structure to find its way out from a critical stage of dynamic response.

- Structure should be designed and detailed so that inelastic deformations can be constrained to develop in desired regions.

- Structure should be provided with balanced strength and stiffness between members, connections and supports.

- Strength and stiffness of the entire building should be compatible with strength and stiffness of soil foundation.

- Selection of the structural system to be used in the preliminary EQ-RD and the final sizing and detailing of the structural members should be based on (or consider) application of energy concepts through the use of the energy balance equation, which can be written as:

$$E_I = E_E + E_D \tag{1a}$$

$$E_I = E_K + E_S + E_{H\xi} + E_{H\mu} \tag{1b}$$

where E_I is the energy input at the foundation of the building due to the EQGMs, E_E is the stored elastic energy, E_D is the dissipated energy, E_K is the kinetic energy, E_S is the strain energy, $E_{H\xi}$ is the energy dissipated through hysteretic damping and $E_{H\mu}$ is the energy dissipated through hysteretic plastic deformation.

B. *Importance of applying energy concepts on EQ-RD*

Comparing the energy balance equation (1) to the following equation used in designs and called the *Design Equation,*

$$\text{DEMANDS} \leq \text{SUPPLIES} \tag{2}$$

it becomes clear that E_I represents the *demands*, and the summation of $E_E + E_D$ represents the *supplies*. Equation (1a) points out clearly to the designer that to obtain an efficient seismic design, the first step is a good estimate of E_I for the critical EQGM. Then the designer has to analyze whether it is possible technically and/or economically to balance this demand with only the elastic behavior of the structure to be designed, i.e., with just E_E, or whether it is convenient to attempt to reduce E_E by dissipating as much as possible of the effects of E_I, i.e., using E_D. As revealed in Eqs. (1a) and (1b), there are three ways to increase E_D: one is to increase the linear viscous damping, $E_{H\xi}$; another is to increase the plastic hysteretic energy, $E_{H\mu}$; and the third is a combination of increasing $E_{H\xi}$ and $E_{H\mu}$. At present, it is common practice to just try to increase $E_{H\mu}$ as much as possible through inelastic (plastic) behavior, i.e., through the use of deformation ductility ratio, , which implies damage of structural members throughout the structure. Only recently has it been recognized that it is possible to increase $E_{H\mu}$ significantly and control damage through the use of *energy dissipation devices* inserted at properly determined locations throughout the structure. Increasing E_D by increasing $E_{H\xi}$ rather than $E_{H\mu}$ has the great advantage that it can provide control of the structure's behavior throughout all of its limit states, i.e., for impending collapse, safety, functional and service performance levels. Increasing $E_{H\mu}$ by just increasing μ will not improve behavior at the service limit state.

If it is technically or economically impossible to balance the required E_I by E_E alone or through $E_E + E_D$, the designer has the option of attempting to decrease the E_I to the structure. This can be done by *seismic base isolation techniques*. A combination of controlling (decreasing) E_I by seismic base isolation techniques and increasing E_D using energy dissipation devices is a very promising strategy for achieving efficient

EQ-RD and EQ-RC of new building structures, as well as seismic upgrading of some existing buildings.

C. *Acceptability checks of conceptual overall design*

As indicated in the flow chart of Fig. B-1, once the overall design has been conceived and formulated, it is necessary to check if the overall configuration and the structural layout are acceptable and that the structural system [superstructure and foundation (considering the mechanical characteristics of the soil under and surrounding the selected type of foundation)], the proposed structural materials and the nonstructural components are technically sound according to the desired performance levels and also economically feasible to construct and maintain them according to the desired risk protection during the service life of the building. For important buildings, it is highly desirable to use pier review to conduct such acceptability checks.

B.5 Methodology for Comprehensive Numerical EQ-Resistant Preliminary Design

In accordance with the comprehensive design philosophy, in the developed comprehensive EQ-RD design approach to attain an economically and technically efficient design, an iterative procedure that starts with an efficient preliminary EQ-RD is recommended. The flow chart of Fig. B-1 and. in more detail, that of Fig. B-3, illustrates the comprehensive numerical design iterative procedure.

As is shown in Fig. B-3, the preliminary EQ-RD is divided into two main phases:

- establishment of the design EQs or EQGMs, and
- numerical preliminary design procedure

A. *First phase: establishment of the design EQs*

As indicated in Figs. B-3 and B-4, to establish design EQs for the desired minimum different performance design objectives [at least two: service (or functional) and safety (or impending collapse)], it is recommended to:

- first, acquire necessary data, including the time histories of the EQGMs (recorded at the site or at other stations with similar site conditions) with different levels of damage potential and return periods (recurrence intervals), T_r;

- second, to select the T_r for the desired performance design objectives (limit states), which should be at least service (or functional) and safety (or impending collapse); and

- third, to process the data, i.e., the EQGMs, to facilitate reliable selection of the design EQs.

B. *Processing of the data*

In this key step, the available data about probable future EQGMs at the site are processed to facilitate reliable selection of the design EQs. Conceptually, a design EQ should be *the critical EQGM* for the limit state under consideration, i.e., the EQGM that drives the structure to its critical (maximum) response for the failure state under study. However, the application of this simple concept in practice meets with serious difficulties. The problems involved in this step, and their solutions, can be summarized as follows.

Given the time histories of probable EQGMs for at least two of the four performance design objectives, say service and life safety limit states, the following are required:

- *For service limit state:* to obtain Smoothed Linear Elastic Design Response Spectra (SLEDRS) for strength (C_s) and displacement (S_d) for different damping coefficients (ξ). For the design of nonstructural components and contents, it is necessary to add the SLEDRS for velocity (S_v) and total acceleration (S_a) as illustrated in Fig. B-5.
- *For safety limit state:* to obtain the SLEDRS and the Smoothed Inelastic Design Response Spectra (SIDRS) for the above design parameters (C_s, S_d, S_v and S_a) as well as the parameters needed for evaluation of the cumulative damage caused by cyclic load reversals considering different acceptable values of the displacement ductility ratio, μ, and ξ. The cumulative damage can be judged through the use of the spectra for the factor γ and proposed damage indices or damage functions. The factor γ is related to the maximum acceptable ductility ratio, μ, with the dissipated energy, $E_{H\mu}$ and the desired damage index or damage function.

C. *Second phase: numerical preliminary design procedure*

This second phase of the proposed conceptual methodology is devoted to the preliminary design (sizing and detailing of the members and their connections and supports) of the entire building system against the critical combinations of the established design EQs with other excitations that can act on the building according to its location and site. As illustrated in Fig. B-3, in order to arrive at the desired final design it is necessary to start with a preliminary numerical design procedure, whose main objective is a design that is as close as possible to the desired final design. As illustrated in Fig. B-3, the preliminary design phase consists of three main groups of steps: (i) preliminary analysis, (ii) preliminary design, and (iii) analysis of preliminary design.

i. *Preliminary analysis*: The objective of this first group of steps is to establish the design criteria by selecting the values of the damage indices (or damage functions) for each of the desired performance levels considered in this preliminary design. The criteria should be based on the results of the processing of the available data for the EQGMs associated with the considered performance levels. To achieve this, it is necessary to:

- *estimate the needed period, T_1, of the building.* The minimum stiffness (or maximum fundamental period, T_1) required to control damage [maximum deformation (displacement and total or tangential interstory drift) or deformation rate (velocity and/or total acceleration)] are estimated based on the use of the single-degree-of-freedom system (SDOFS) spectra developed in the processing of the EQGMs expected at the site, but considering from the beginning through adequate factors (parameters) that:

- the real building structure is a multi-degree-of-freedom system (MDOFS), and therefore the response can be affected by higher modes (factor β_2);

- there can be important torsional effects, and these effects can be different for the different limit states (performance levels) (factors β_1, β_3 and β_4);

- it is necessary to take into account that the acceptable value of is affected by the expected energy dissipation due to plastic deformations, $E_{H\mu}$ (factors β and γ); and

- the final designed and constructed building will offer some overstrength, as well as some overstiffness (at least before it is subjected to at least a moderate EQGM) (factor R_{OVS}).

The flow chart of Fig. B-5 shows the steps involved in the preliminary analysis for estimating the design seismic forces.

ii. *Preliminary design: design for lateral stiffness.* As shown in Fig. B-5, the preliminary sizing of the members is carried out in the preliminary analysis step, and is done to obtain an entire building system that has the stiffness or fundamental period, T_1, required to control damage. This involves an iterative procedure: after sizing the members (including foundation) to the provided required lateral stiffness, the resulting structural system is analyzed to compute its eigenvalues, and the resulting value of T_1 is checked against the required $T_{1\,max}$.

iii. *Preliminary design: design for lateral strength.* Once the preliminary sizing of the structure satisfies the demanded minimum stiffness, it is possible to estimate the seismic design forces through the use of the strength (C_s) SIDR spectra for the different limit states under consideration. It should be noted that although the spectra used are for SDOFS, the values of C_s obtained from these spectra are modified by factors that consider the effects of the higher mode of the MDOFS (β_2); the effects of torsion (β_1, β_3 and β_4); acceptable damage [(through the use of damage indices that consider the effects of μ, $E_{H\mu}$ and β)]; and expected overstrength (R_{OVS}). The overstrength depends on the design methodology that is used (allowable stress, ultimate strength or limit plastic design) and the redundancy of the EQ-resistant system, and also significant overstrength can be obtained) because usually most structural elements are oversized and constructed with materials having yielding strength and maximum strength higher than the nominal values used in the design.

The preliminary design for strength can be carried out using different methods:

- designing for just one limit state, the one that appears to control the design, and then checking for the others
- designing simultaneously for the demands of all the main limit states

It is recommended to design simultaneously for at least two limit states: fully operational/service (or functional) and safety (or impending collapse). The numerical design can be carried out using methods based on optimization theory and using practical requirements as constraints. To conduct such optimal design, it is necessary to estimate the critical combinations of the seismic forces with the other loads (excitations) that can affect the behavior at each of the limit states (damage levels) controlling the design.

- <u>Load Combinations</u>. Usually, the following sources of loading (excitations) are considered:

  - gravity loads (dead, D, and live, L loads);
  - snow, S;
  - wind, W;
  - seismic, E;
  - rainwater or ice, R', exclusive of ponding conditions; and
  - self-straining forces, T.

- <u>Load Factors</u>. In current building codes, the values for each of the loads and their respective load factors vary according to the different ways that these loads are defined, as well as according to the limit state (performance level and corresponding level of excitation) for which the building is designed, and the structural material that is used. To have a consistent methodology for combining the loads for the design of buildings with different materials, it is recommended that the basic combinations recommended by the recent revision of ANSI/ASCE 7-88, i.e., ASCE 7-93, be used.

- <u>Combining Loads Using Allowable Stress Design.</u> *Basic Combinations.* All loads shall be considered to act in the following combinations, whichever produced the most unfavorable effect in the building, foundation or structural member being considered. The most unfavorable effect may occur when one or more of the contributing loads are not acting.

1. D
2. D + L + (L_r or S or R')
3. D + (W or E)

   where L_r is the roof live load.

The most unfavorable effects from both wind and earthquake loads shall be considered, where appropriate, but they need not be assumed to act simultaneously.

Load Combination Factor. For the load combinations given above, the total of the combined load effects may be multiplied by the following load combination factors:

1. 0.75 for combinations including, in addition to D:
   L + (L_r or S or R') + (W or E)
   L + (L_r or S or R') + T
   (W or E) + T

2. 0.66 for combinations including, in addition to D:
   L + (L_r or S or R') + (W or E) + T

- Combining Loads Using Strength Design. *Basic Combinations*. Structures, components and foundations shall be designed so that their design strength exceeds the effects of the factored loads in the following combinations:

  1. 1.4D
  2. 1.2D + 1.6L + 0.5(Lr or S or R')
  3. 1.2D + 1.6(Lr or S or R') + (0.5L or 0.8W)
  4. 1.2D + 1.3W + 0.5L + 0.5(L_r or S or R')
  5. 1.2D + 1.0E + 0.5L + 0.2S
  6. 0.9D - 1.3W or + 1.0E

  Exception: The load factor on L in combinations 3, 4 and 5 shall equal 1.0 for garages, areas occupied as places of public assembly and all areas where the live load is greater than 100 lb/ft^2 (pounds-force per square foot).

Each relevant strength limit state shall be considered. The most unfavorable effect may occur when one or more of the contributing loads are not acting. The most unfavorable effects from both wind and earthquake loads shall be considered, where appropriate, but they need not be assumed to act simultaneously.

- Specific Definition of the Earthquake Load Effect, E. The value of E to be used for each of the design objectives (limit states) in this comprehensive approach should be the one associated with the base shear, V, and the corresponding inertial forces resulting from the use of each of the Smoothed Design Response Spectra (SDRS) that has been selected. Then for each of the four main limit states the E would be the effect directly produced by:

- $V_{service} \rightarrow E_{ser}$
- $V_{functional} \rightarrow E_{fun}$
- $V_{safety} \rightarrow E_{saf}$
- $V_{impending\ collapse} \rightarrow E_{imp.coll.}$

While E_{ser} should be used in the basic combination of factored loads when allowable (working or service) stress design is used, the E_{fun}, E_{saf}, and $E_{imp.coll.}$ should be used with basic combinations of factored loads when strength design methods are used.

- It is also recommended that the earthquake load effect, E, be determined using the approach recommended by the 1994 NEHRP, which includes the effects due to gravity loads and horizontal and vertical components of earthquake-induced forces.

The importance of proper (reliable) estimation of, first, all the possible sources of excitations to which the entire building system can be subjected during its service life, and secondly, all the different probably critical combinations of the effects of these excitations that can act simultaneously cannot be overemphasized. This has been clearly demonstrated and illustrated by Bertero and Tezán in their analysis of the effects of the relative values of the gravity loads with respect to the effects of the EQGMs on, first, the design of the structure, and then on its behavior under different limit states. There is an urgent need to investigate not only the reliability of the present recommended basic load combinations but also of the values of the load factors that are currently specified. It has to be noted that, as discussed above, the specified values for the load factors are for just the cases that either allowable stress design or strength design methods are used *without any considerations that in the performance-based EQRD the strength design method is used for different limit states which involves EQGMs of increasing damage potential at a decreasing probability of occurrence.* Therefore, it is doubtful that just the above specified set of load factors can be reliably used for different limit states (performance levels) such as functional (continuous operation) with EQGMs that may occur each 30 years, and impending collapse with EQGMs whose return periods is 950 years.

B.6 Preliminary EQ-RD of Nonstructural Components Supported by the Building Structure

In this comprehensive design approach, the preliminary EQ-RD of the nonstructural components such as the *architectural components (cladding, parapets, partitions, stairs,* *doors and windows, tailings and furnishings) supported by the building structure, the equipment mounted in the building (electrical and mechanical) and the plumbing components*, can be conducted following one of the two following philosophies or strategies:

1. the nonstructural elements are designed and constructed so that their seismic response is uncoupled from the response of the primary structure;

2. the nonstructural elements are designed and constructed so that their responses are coupled with the behavior of the primary structure.

A. *Architectural components*

If the above first strategy is used, a sufficient gap should be left between these components and the primary structure so that they will not interact. In case of partitions, the gap is made along the top of the partitions and up the sides. The architectural elements will need restraints at the top against overturning by out-of-plane forces, which for each of the performance levels (limit states) considered in the design criteria have to be determined based on the expected floor response spectrum of the primary structure.

In the second strategy, which appears to be the more rational strategy for the architectural elements that are permanent, these elements are considered as part of the entire structural system and are included from the start of the design in the modeling of the entire structural system, so that the deformations and forces for which they have to be designed at each of the performance levels are obtained directly from analysis of the developed 3D model.

B. *Preliminary design of equipment mounted in buildings*

The reliable prediction of the vibrational forces induced by the operation of equipment mounted in a building and its design to resist the effects of EQGMs and have an acceptable performance at each of the established different levels of these EQGMs is a complex dynamic problem which cannot be solved reliably with equivalent static methods alone.

The EQ-RD of the equipment and its mounting at each of the performance levels considered in the selected design criteria takes into account the interaction between the dynamic characteristics of the buildings and the equipment. It also considers the type and level of service (performance) expected from the functionality of such equipment and therefore its mounting are designed to remain elastic and functional, i.e., to more stringent EQ criteria than the design of the building housing it, which could be designed taking advantage of its ductility. This seems also to be the philosophy involved in the seismic provisions for equipment in the new 1994 UBC and 1994 NEHRP. These new provisions could be used for preliminary design of the nonstructural components whose performance could then be checked through the performance analysis.

B.7 Analysis of the Performance of the Preliminary EQ-RD: Acceptability Checks

As indicated in the flow chart of Fig. B-1, to determine if the preliminarily designed building will have an acceptable performance according to the desired performance levels in the adopted design criteria, the following different methods of structural and stress/strain analyses are recommended:

- a check of the stress ratios, using the results of the 3D linear elastic dynamic analysis;
- the use of static linear and nonlinear analyses (pushover analyses); and
- the use of nonlinear dynamic time-history analyses of realistic MDOFS (finite element models) or reduced MDOFS (stick models).

The advantages and limitations of each of these methods of analysis are discussed, and emphasis is put on the need to judge the reliability of the results obtained from the use of each of these methods considering the uncertainties involved in the: establishment of the EQGMs; computation of the SDOFS spectra; critical load combinations; factors used to account for the effects of MDOFS, torsion, damage due to global, story and local μ and $E_{H\mu}$, and overstrength; modeling of the real building system (particularly the effects of foundation, nonstructural components and contents); estimation of the demands; and estimation of the supplies (strength, deformation and energy capacities). It is recommended that for irregular or complex or important buildings, the acceptability checks be conducted with a peer review.

A. *Acceptability Checks: Acceptability Criteria*

The criteria are based on comparisons between the results obtained from the above analyses of the preliminary EQ-RD considering each of the levels of performance that need to be considered according to the adopted design criteria, and the values of the response parameters controlling the damage at each of the performance levels of the structure as well as nonstructural components and contents. These parameters are:

- strength (allowable or service, first significant yielding, maximum and ultimate);
- deformation [lateral and vertical displacements, lateral IDI (total and/or tangential) and rotation]; and
- energy dissipation due to plastic deformation (overall, story and local).

At each of the main desired performance levels (limit states), it is necessary to check the values of the estimated demanded strength, deformation, rate of deformations, μ and $E_{H\mu}$ against the acceptable values that have been established in the adopted design criteria, which themselves depend on what controls the behavior of the structural elements and of the so-called nonstructural components and contents.

As indicated below, usually the criteria used are based on comparing values of the global, story and/or local story response with the acceptable values established for such responses. Because the judgment of performance is based on the degree of damage, and damage is more a consequence of deformations than strength, emphasis is placed on using deformations as the response parameter. In establishing these values, it should be kept in mind that what controls the global and story responses is the local behavior of the critical regions of the members, and this varies significantly with the structural system and detailing of these regions, as well as with the types of loads and time history of these loads. The importance of attaining not only reliable estimation of the local demands on the critical regions of the structural members but also of the provided capacity and of what can be considered as acceptable local performance cannot be overemphasized.

B. *Acceptable Values for Deformation*

- *Displacements: interstory drift index.* Acceptable values of IDI at *service level* are dictated by the nonstructural components and contents, and vary from 0.002 to 0.005, depending on the type of nonstructural components and contents. For *safety limit state,* the acceptable IDI depends on the nonstructural components and the function (occupancy) of the buildings, it varies between 0.01 and 0.02. For *impending collapse limit state,* the acceptable IDI depends on the structural system and can vary between 0.02 and 0.04.

- *Rotations.* The maximum plastic rotation at the critical regions (plastic hinges) of the structural elements at *safety limit state* varies according to the structural material used from 0.01 to 0.03 radians, and at impending collapse state can vary from 0.02 to 0.05 radians depending on the structural system and material.

C. *Acceptable Values for the Rate of Deformation: Velocity, Acceleration and Jerk*

Although at present reliable procedures and criteria for limiting building motions do not exist, some values for maximum jerk and total acceleration have been suggested to avoid human discomfort during service EQGMs with $T_r \leq 10$ years. For these cases, the total floor acceleration should not exceed 0.10g. Duration of severe motion plays an important role in the acceptable values.

For safety limit states, the total floor acceleration should be limited to 1.5g to avoid failure and replacement and/or repair of nonstructural elements and contents. This and even more strict requirements will accelerate the interest in the use of damping devices to decrease demands in intensity as well as duration of strong motion of buildings.

D. *Acceptable Values for Strength*

- *For acceptability under service (minor) EQGMs,* the stresses due to critical load combinations should be equal to or smaller than specified allowable stresses for the structural material. By using proper load factors for the different loads, the computed demands can be compared directly to acceptable yielding stresses of the structural material. [There is a need to improve control of the mechanical characteristics of each of the materials (stiffness; yielding stress; strain hardening; maximum strength; ductility, i.e., maximum deformation and ultimate usable strength -- toughness and low-cyclic fatigue life) as well as to improve the specifications of their values for structural values].

- *For acceptability check under moderate, major and extreme EQGMs,* the stresses resulting from the critical combinations of the factored loads are compared to the yielding strengths and/or the maximum strengths (depending on the acceptable ductility ratio,) of the structural materials.

E. *Acceptable Values for Ductility Ratio, μ, and Energy Dissipation Capacity, $E_{H\mu}$*

- *For acceptability check under moderate, major and extreme EQGMs.* The acceptable depends not only on the acceptable maximum values of the μ of the structure determined by monotonically increasing deformations [determined experimentally or analytically based on the local maximum monotonic μ, $(\mu_{mon})_l$], but also according to the desired acceptable damage index (damage function), which includes the effect of the demanded $E_{H\mu}$ on the expected damage of the structure. For reinforced concrete and masonry structures, damage indices suggested by Ang and his associated researchers are used:

$$(DMI)_{PA} = \frac{\delta}{\delta_u} + \beta\frac{\int dE_{H\mu}}{F_y\delta_u} \qquad (3)$$

For steel structures, damage indices based on the Miner index,

$$(DMI)_M = \sum_i \frac{n_i}{N_i} \qquad (4)$$

are used.

The demanded $E_{H\mu}$ at each story and, in the case of SMRF structures, the $E_{H\mu}$ at each of the beam-column connections, as well as any region along the members on which moving plastic hinge regions can be developed, is compared with available experimental information regarding acceptable $E_{H\mu}$.

F. *Acceptable Checks for Performance of Nonstructural Components and Contents*

The performance of preliminarily designed nonstructural components and contents at each of the selected levels of EQGMs is evaluated through the same analyses that are used for the entire structural system since the components and contents are included in the models of the entire structural system.

The acceptability check is based on comparing the results obtained from the above analyses and the values of the response parameters controlling the damage at each of the performance levels included in the adopted design criteria. The response parameters controlling the damage of architectural components are similar to those used for the primary structure, with the exception that for the behavior out of plane of these elements the rate of deformation (total acceleration) could be the controlling parameter.

For equipment and its mounting and components, the maximum deformations and particularly the maximum deformation rates (velocity and total acceleration) could control their performance. The acceptable values for these response parameters need to be investigated thoroughly.

B.8 Final Design and Detailing

Although the ideal comprehensive design procedure should be based on target reliability for the different limit states (performance design objectives) considered in the design criteria, and attempts have been made to use probabilistic approaches in the establishment of design EQGMs and damage functions, and in the analyses of the demands and of the structural capabilities, the currently proposed approach cannot be considered a true reliability-based EQ-RD approach. This is something that has to be developed through intense focused research. Therefore, until this is done, what can be called the "Reliability Analyses of the Final Design" should be conducted using the most reliable available 3D method of analysis for each of the probable critical load combinations corresponding to each of the adopted performance design objectives. Evaluation of the results obtained from these analyses will allow determination of whether it is necessary to modify the sizing of the components of the entire structural system or if it will be sufficient just to improve the detailing of the critical regions of some of these components.

As indicated in Fig. B-1, it is recommended that an independent review be conducted on the Final Design. This should be done particularly in case of irregular, complex, tall or important buildings.

A. *Final detailing of the members of the primary structure*

Although the final detailing can be done using present seismic code provisions (1994 UBC and/or 1994 NEHRP) for the different structural materials (which have been improving significantly *during the last two decades*), the performance of certain building structures during recent moderate to major EQGMs has shown that there is room for improvement. In this comprehensive design approach, it is recommended that the final detailing of the members and their connections and supports be done based on energy concepts. This requires estimation (determination through 3D time-history analysis) of the deformation time history of the critical regions of the structural members when the building is subjected to the critical EQGM at each of the performance levels (limit states). Research is needed on the $E_{H\mu}$ of the members and their connections using realistic time histories of the deformations.

B. *Final detailing of the nonstructural components and their attachments to the primary structure*

Although the 1994 NEHRP recommended two analytical procedures to estimate the seismic forces acting on these components and procedures to evaluate deficiencies in nonstructural components are described in FEMA 178, an analysis of these recommended methods and procedures, as well as from a review of the literature pertinent to this subject, it can be stated that at present:

- There is no reliable "Code of Practice" on detailing of the nonstructural components.
- There is very little literature available giving specific guidance on the detailing for seismic effects of the architectural components and of their attachment to the primary structure.
- There is a need for basic research in this area.

B.9 Quality Assurance During Construction

Quality control of materials and of the construction (workmanship), inspection and supervision are of utmost importance in attaining efficient EQ-RD. Therefore, the new proposed conceptual framework for seismic codes should address specifically what constitutes satisfactory construction quality assurance and how this can be achieved, spelling out the responsibility of each of the key personnel involved in such quality control.

The design professional is responsible for seismic quality assurance during construction. The designer is the professional most familiar with the intent of the design. For prescriptive design without professional involvement, the building official shall be responsible for quality assurance. Specially licensed independent inspectors may be required to assist the building official. Professional responsibility is necessary to obtain structures with predictable seismic performance. The high costs of the Northridge earthquake are directly related to poor construction and poor field inspection.

The type and frequency of the inspections will depend upon the performance level objectives for the structure and the structural system. Uniform regional application of quality standards will improve the quality of construction by allowing the design professional, building official and the construction team to be familiar with the quality requirements. All professionals, building officials, special inspectors and contractors shall demonstrate an adequate understanding of the seismic requirements for their area of work.

- Building Officials

    - Review design documents for conformance.
    - Inspect construction for general conformance.
    - Review special inspection and testing results reports.

- Design Professionals

    - Specify special testing and inspections required beyond those mandated by codes and/or building officials.
    - Periodic inspection of shop fabrication and job site construction for general conformance.
    - Review of special inspection and testing reports.

- Resident Engineers
- Contractors
- Field Inspectors
- Testing Agencies

B.10 Monitoring Building Maintenance and Function

The maintenance of the system for transferring and resisting the induced lateral load, usually called "lateral load path" is essential. Deterioration and/or removal of structural elements will affect the lateral capacity of a structure. All changes to a structure shall be reviewed for seismic effects by the building official. Field investigations and existing drawings, when available, shall be used to determine the effect of changes on the lateral resisting elements. The review by a qualified independent professional may be required.

The structure's function may affect the lateral capacity. Stored contents may increase the lateral forces. Unsecured contents present a falling hazard as well as hazards to framed floors, columns and walls. Provisions may be required to protect important structural elements. For structures with damage control requirements, periodic seismic review of the contents may be necessary.

B.11 Commentary on Effectiveness of the Comprehensive EQ-RD Approach

A. *General comments*

This new proposed approach for performance-based EQ-RD covers, in a rational and transparent way all the aspects that need to be considered for resulting in designed buildings with predictable performance. *The flow chart of Fig. B-1, summarizing all the steps needed for the developed comprehensive iterative EQ-RD procedure and the analytical methods that can be used, can be used as a guideline for the development of other performance-based EQ-RD simplified approaches as well as adaptation of already existing or proposed approaches.*

The proposed conceptual methodology for the comprehensive numerical EQ-RD, which was developed in accordance with the comprehensive design philosophy, is in compliance with the worldwide-accepted EQ-RD philosophy, and is based on the use of energy concepts and on fundamental principles of structural dynamics and comprehensive design, considering realistic mechanical behavior of the entire building system. It takes into account from the very beginning of the EQ-RD procedure (i.e., from the preliminary design) the simultaneous demands for strength, deformation, including torsional effects, and their combined effects on the demanded and supplied energy capacities of the entire facility system, as well as on the acceptable damage at the different limit states associated with the desired performance levels.

The main advantage of the proposed conceptual comprehensive methodology for performance-based EQ-RD is that, notwithstanding great uncertainties in some of the concepts involved in its codification, the numerical quantification of these concepts can be improved without changing the format of this codified methodology as new and more reliable data are acquired.

B. *Comments on effectiveness of this proposed comprehensive EQ-RD approach*

The proposed conceptual methodology for conceptual numerical EQ-RD was applied to the design of a 30-story SMRF reinforced concrete building [Bertero V.V. and Bertero R.D. (1993 & 1994)]. From these applications, the following observations can be made regarding the design of the:

<u>Superstructure:</u>

- It is highly effective in meeting the performance objectives for the two main performance levels (service and safety)

- The application of energy concepts through the use of energy equations has the advantage that it guides the designer through the different alternatives at his or her disposal to find a technically and economically efficient design. It encourages and guides the designer in the proper application of recent developments in the use of techniques in base isolation and energy dissipation devices.

<u>Foundation:</u>

- By including the foundation and the soil under and surrounding the foundation in the mathematical modeling of the entire structural system, the EQ-RD of the foundation is very effective in meeting the intent of current seismic code provisions (UBC 1994 and NEHRP 1994) which is ". . . to provide foundations that have the capability to support the loads that may develop during the inelastic response of the structure induced by major EQGMs. The foundation elements themselves should be designed so that significant inelastic behavior does not occur." This is achieved by designing the foundation to respond without significant yielding and deformation forces that can be induced by the superstructure (i.e., designing for the capacity of the superstructure). The possibility of allowing the foundation to dissipate significant amounts of energy input through its controlled inelastic behavior or through the use of special energy dissipation devices located in the foundation and the advantages of doing so need to be investigated.

Nonstructural components and contents:

- By introducing the main architectural elements and equipment in the mathematical model of the entire building system, at least an idea of their performance at the different desired levels can be achieved. To improve the effectiveness of designing these nonstructural components as well as the proper anchorage of the contents of a building, it is necessary to investigate experimentally their mechanical characteristics and their dynamic response.

B.12 Research, Development and Implementation Needs

A. *Research and Development Needs*

To improve application of the recommended performance-based comprehensive EQ-RD, it is necessary first to carry out research on the research done on the quantification of the different concepts involved in this approach, and then to conduct integrated analytical and experimental studies needed to obtain more reliable data regarding those concepts in which the present information available is deficient. Topics that require investigation are grouped according to main steps involved in the EQ-RD procedure.

Establishment of the Design Seismic Hazards and Particularly the Design EQs

There are research needs for:

- Development of more reliable maps of possible sources of EQGMs;
- Development of more reliable seismic microzonation maps;
- Development of more reliable maps for identification of the probabilities of different potential sources of seismic hazards;
- Development of reliable methods of predicting the time histories of EQGMs at the free-field surface of different site conditions considering the linear and nonlinear behavior of soil layers, particularly very soft soils, which under low-intensity EQGMs at the base rock can be amplified and under very intense EQGMs can be attenuated. Special emphasis should be placed on predicting the probable duration of the strong EQGMs and the return period, T_r, of the expected EQGMs at different levels of damage potential.

- Improvement of present methods and development of more reliable methods for identifying the damage potential to structures of recorded or analytically predicted EQGMs. Study of the advantages and disadvantages of the methods based on energy concepts and the method recently proposed by Bazurro and Cornell (1994).

- As most of the advances in the use of energy concepts to investigate the damage potential of the EQGMs to structures have been achieved through analytical studies conducted on SDOFSs, and because real structures, and particularly building structures, are MDOFSs, there is an urgent need to conduct integrated analytical and experimental studies on the validity of applying the results obtained from analysis of SDOFSs to MDOFSs. There is a need to develop reliable methods for modifying the spectra obtained for SDOFSs to take into consideration the effects of MDOFSs. Among these spectra of special importance is how the SDOF spectra for the parameter can be affected by the MDOF response of real structures.

Preliminary analysis and design for stiffness

To improve the quantification of the concepts involved in the preliminary analysis, there is a need to conduct the following studies:

- In order to establish reliable design criteria, it is necessary to define the tolerable structural and nonstructural damage under the different levels of EQGMs that are associated with the selected performance levels. There is a need to investigate the parameters controlling these damages and their reliable quantification, particularly for the nonstructural components and contents.

- Probabilistic studies of the equations proposed for the preliminary analysis should be done. Design factors for the demand and supply variables in these equations should be derived using those probabilistic studies.

- Development of more specific and reliable guidelines for overall conceptual design (i.e., conception of configuration, structural layout and structural system and selection of structural material and types of nonstructural components and their material

- Investigation of the contributing architectural components to the lateral stiffness of the primary structure.

- Development of more efficient and reliable methods for determining preliminary sizes of the structural members for minimum required lateral stiffness.

- Improvement of the probable combinations of the loads that can act simultaneously and the reliability of the load factors associated with such combinations.

- Extensive studies of the factors β_1, β_2, β_3, β_4, and R_{OVS}, which are used to introduce the effects of elastic (β_1) and inelastic (β_3 and β_4) torsion, concentration of IDI and μ demands at one story in the case of MDOFS effects (β_2), damage (β) and overstrength (R_{OVS}) during the preliminary analysis, are needed. These studies must build a database to improve the quantification of the concepts involved in the proposed design methodology.

- In order to consider the torsional accidental eccentricity in a rational way in the design, there is an urgent necessity for statistical studies about the actual eccentricity in existing buildings.

- The influence of higher modes in the distribution of the lateral forces along the height as well as thorough the plan of each story at the different desired performance levels (limit states) need further investigation.

- Improvement of present damage indices or damage functions and a search for new better ones for the different types of structural systems that can be used and for the different structural materials. The needed studies should address first the prediction of the local damage at the critical regions of the members, i.e., local $(\mu)_l$ and $(E_{H\mu})_l$; and then at the story level, $(\mu)_s$ and $(E_{H\mu})_s$; and finally the damage for the entire (global) facility, i.e., $(\mu)_g$ and $(E_{H\mu})_g$.

Preliminary Design for Strength

Studies are needed to improve presently available methods for automatic optimal design satisfying simultaneously the requirements imposed by the different performance levels (limit states) associated with the performance design objectives (design criteria). These studies should cover all types of structural systems and structural materials that could be used in practice. The use of the *Capacity Design Philosophy* to tell the structures what to do (i.e., design methodology based on structural control) should be used in these needed investigations. To accomplish the objectives of these studies there is a need to:

- Improve the quality of the materials used in construction, particularly the control of the main mechanical (dynamic) characteristics of these materials that are needed to determine their strength and stiffness under the different states of strain to which they can be subjected at the different limit states of their behavior, as well as their ductility, toughness and low cyclic fatigue life under the deformation histories to which they can be subjected during their life due to earthquake excitations.

- Conduct studies of the contribution of the nonstructural (architectural) components to the strength capacity of the entire structural system.

Analysis of the Performance of Preliminary EQ-RD (Acceptability Check)

- Investigation of the development of more reliable and efficient: methods for the 3D modeling of entire facility systems (soil-foundation, superstructure, nonstructural components, equipment and contents); and computer programs for the 3D linear and particularly the nonlinear dynamic analyses of realistic 3D models. To facilitate this investigation, it will be necessary to:

    1. use earthquake simulator facilities to perform integrated analytical and experimental studies on the 3D seismic performance of different types of facilities;

    2. properly instrument entire multi-story facility systems having different structural systems and of different structural materials; and

    3. conduct integrated analytical and experimental investigations on the dynamic behavior at different limit states, and particularly on the energy dissipation capacities of the different facility structural components as well as their basic subassemblies when these components and assemblies are subjected to excitations reliably simulating the effects of the response of the facility system to critical ground motions.

Final Design and Detailing

Research and development are needed to improve the design and particularly the detailing of the different:

- structural members and their connections and supports [this will require the integrated analytical and experimental investigations described above under item (3)]. There is an urgent need for the development of a reliable "Code of Practice" on detailing of nonstructural components.

Acceptability of Final Design

Extensive studies are needed, including detailed comparative analysis of the performance of the facilities designed using the proposed comprehensive design with the performance of similar facilities designed by current seismic code design procedures that have performed successfully during EQGMs when subjected to different levels of EQGM damage potential. These comparative analyses should include not only the volume of materials that are required but also the initial cost of their construction and particularly the total cost of the probable seismic risk protection that will be demanded in their service life due to the losses and cost of repair and/or upgrading.

B. *Education needs*

There is an urgent need for starting a massive education program which will cover:

- education of the practitioners (engineers, architects, contractors etc.), government, code officials, etc., regarding the basic concepts involved in the performance-based EQ-RD and EQ-RC, and on application of the performance comprehensive design approach discussed above, as well as the other derived simplified alternative approaches.

- Education of the public about the advantages of the proposed performance-based EQ-RD and EQ-RC, emphasizing the importance of not just looking at the initial cost of the construction, but of the probable total cost of the seismic risk protection during the intended service life of the facility given such a type of design and construction.

C. *Implementation needs*

Researchers, practitioners and code officials should collaborate in formulating seismic code regulations based on the proposed performance-based EQ-RD and EQ-RC, including the different approaches, and the government agencies should strictly implement the practical application of these regulations. This is the only effective way to mitigate the seismic risks.

B.13 References

Bertero, V.V. and Bertero, R.D. (1994), "Formulation and Application of a Conceptual Code Procedure for Earthquake-Resistant Design of Building Structures," *Proceedings of the Fifth US National Conference on Earthquake Engineering,* July 1994, EERI, Chicago, IL, Vol. II pp 391-400.

Bertero, V.V. and Bertero, R.D. (1993), "Formulation of a Conceptual Seismic Code," *Proceedings of the Tom Paulay Symposium,* September 1993, University of California at San Diego, La Jolla, CA, pp 286.

Bazurro and Cornell, A. (1994), "Nonlinear Design for Seismic Hazards," *Journal of the ASCE Structural Division,* Vol. 120 - No. 11 - pp 1320-1365.

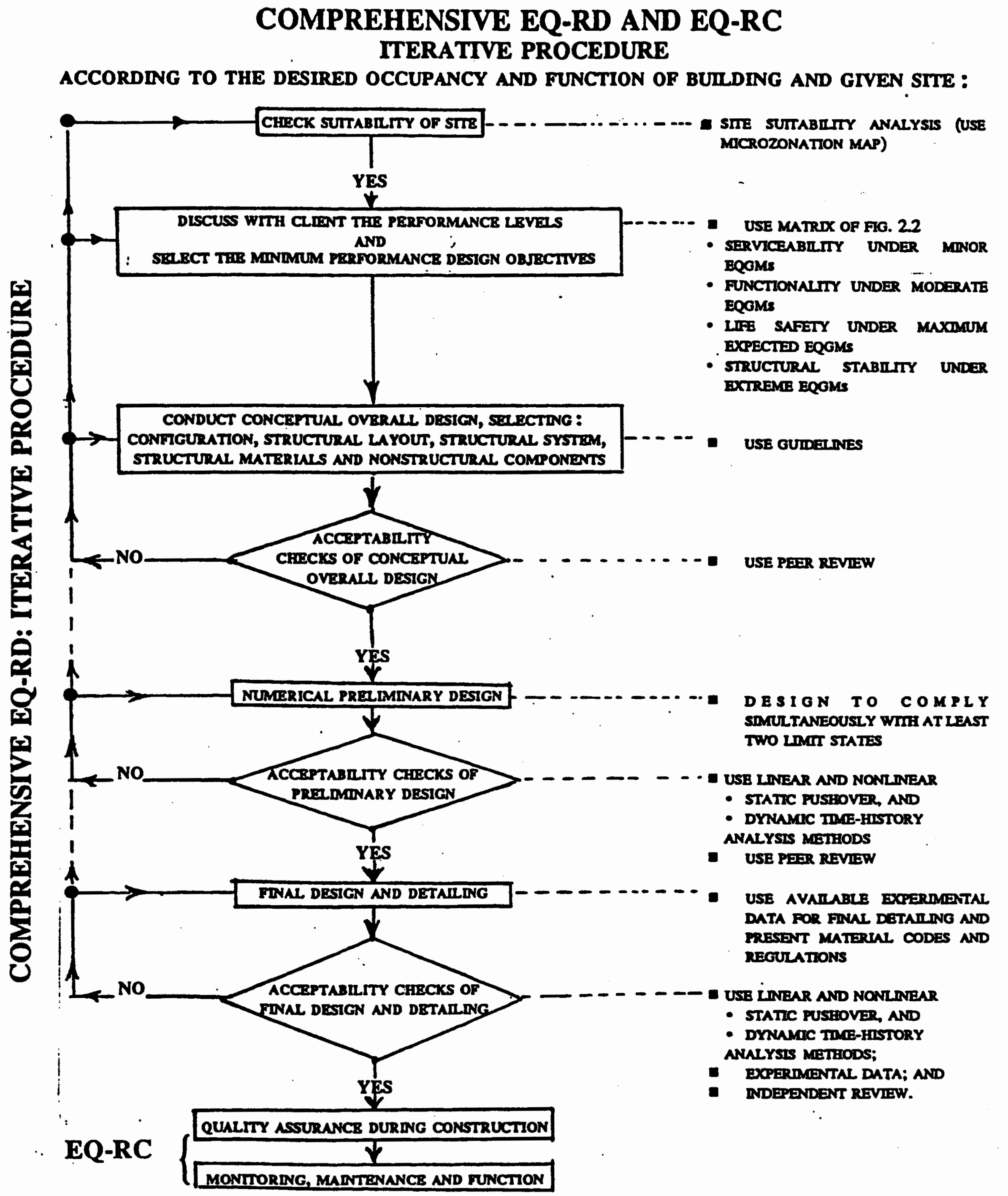

Figure B-1: Flow chart for comprehensive EQ-RD and EQ-RC

PERFORMANCE-BASED EQ-RD
(BASED ON DESIRED PERFORMANCE OF BUILDINGS ACCORDING TO THE DAMAGE POTENTIAL OF EQGMs)

LEVELS OF EQGMs		RELATIVE COSTS OF CONSTRUCTION AND SEISMIC RISK PROTECTION ACCORDING TO THE FOLLOWING DESIRED LEVELS OF PERFORMANCE OF THE BUILDING			
T_r (YEARS)	DAMAGE POTENTIAL	NO DAMAGE	CONTROLLED DAMAGE		
			Controlled Nonstructural Damage	Controlled Nonstructural and Structural Damage	
		SERVICE	FUNCTIONAL: CONTINUOUS OPERATION	LIFE SAFETY	
				Repairable Damage	Impending Collapse (Demolition)
Frequent (5)	Minor				
Occasional (30)	Moderate				
Rare (475)	Major				
Very Rare (950)	Extreme				

UNACCEPTABLE

Figure B-2: Matrix of acceptable performance of design objectives
for the San Francisco Bay Area

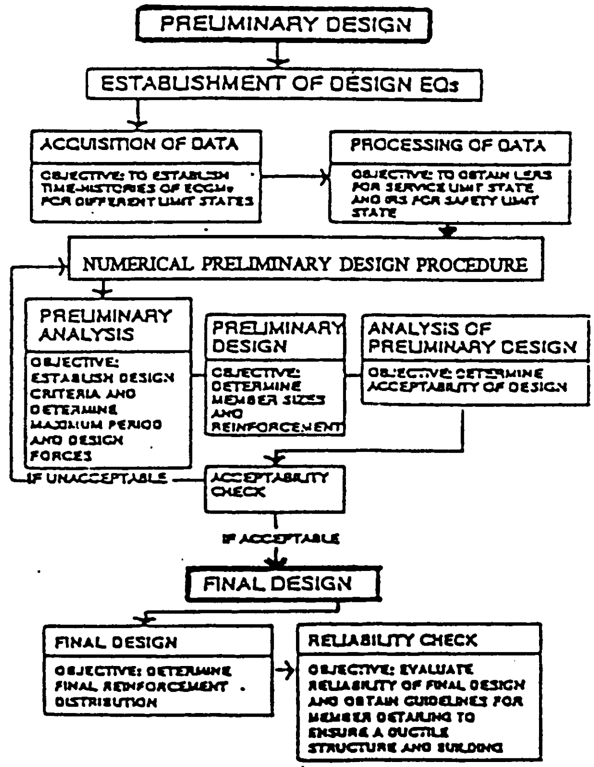

Figure B-3: Flow chart for comprehensive numerical EQ-RD

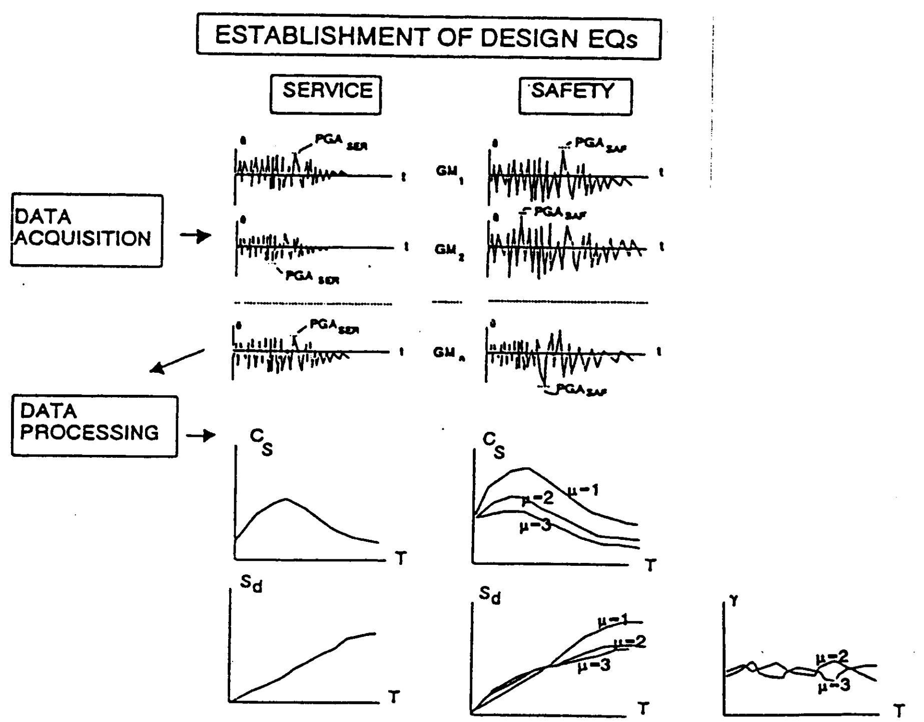

Figure B-4: Steps needed for the establishment of design EQs

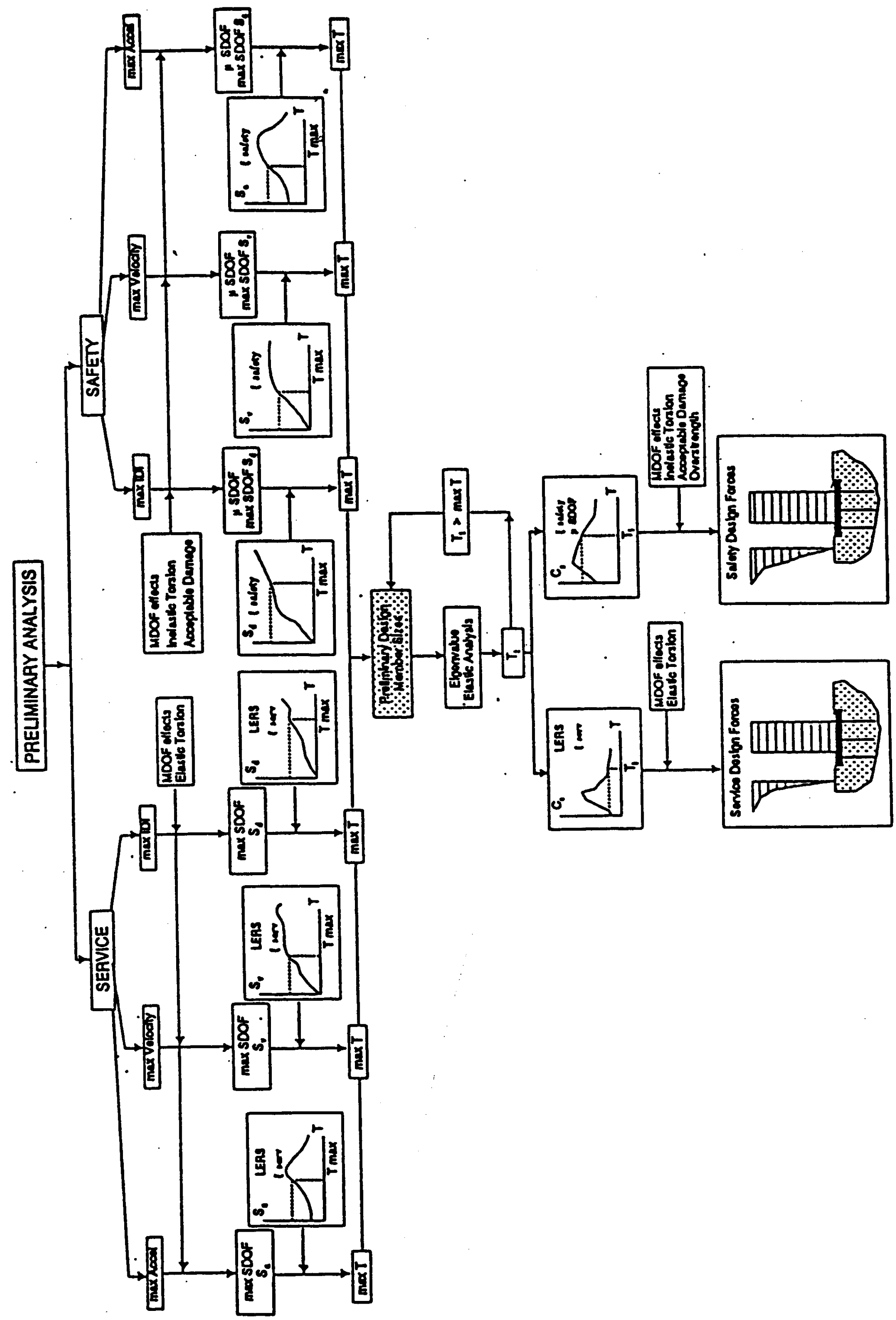

Figure B-5: Flow chart of preliminary analysis

APPENDIX C

DISPLACEMENT BASED DESIGN APPROACH

by

James Hill

1. Overview of Procedure

The performance based seismic design provisions are based upon controlling story drifts to prescribed limits under the design basis earthquake. Since damage can usually be related in some manner to story drift, the approach is well suited to the objectives of performance based seismic design. The approach is applicable to both new and existing buildings, the principal difference being is developing the initial member apportionment. For an existing building the initial sizing is defined, whereas for a new building, a procedure is required to provide the initial member sizes and stiffness. The following discusses a simple approximate method for relating story masses, story stiffness, and story drift, and discusses a deformation design approach currently being proposed for use in the retrofit of nonductile concrete buildings.

2. Initial Sizing and Performance Estimations

While there are several simple and approximate methods for estimating mode shapes, frequencies as a function of mass and story stiffness, the following procedure is useful and can be developed in a spreadsheet format to provide rapid solutions. The method is based upon an assumption of harmonic motion with the peak acceleration related to the peak displacement by the frequency squared. The procedure is to estimate a mode shape, calculate the inertial forces, compute the deflections from the inertial forces and compare the displacements with the original estimate. The calculated mode shape is then used as the new estimate until a convergence is achieved. Consider the following three story example as shown in Figure C-1.

The initial estimate was quite close and in this case the first iteration could be used.

For the assumed and the calculated displacements to be equal the term

$$3.6\, w^2\, a/100 = a \quad \text{or} \quad w^2 = 100/3.72$$

The participation factor is calculated as:

$$PF = \frac{\Sigma y_i * m}{\Sigma y_i * m_i^2}$$

and the estimated building deflections can be determined using a single degree of freedom response spectrum scaled by the participation factor. For a design story mass distribution an estimated required stiffness to achieve a maximum drift can be quickly established by iteration.

3. Dynamic Lateral Analysis Procedure

The following procedure and suggested code language has been developed for nonductile concrete frame and nonductile concrete frame and infill buildings. Similar procedures can be adapted to other building types.

1. **General.** Structures shall be analyzed for seismic forces acting concurrently on the orthogonal axes of the structure. When using modal analysis procedures, the effects of the loading on two orthogonal axes shall be combined using an appropriate statistical method.

2. **Ground Motion.** For Life Safety purposes, the ground motion shall be one having a 10 percent probability of being exceeded in 50 years and may be one of the following:

a. **Normalized Response Spectra.** A normalized response spectrum developed for a damping ratio of 0.05 and an appropriate soil type. The spectrum shall be constructed using average values of spectral amplification.

b. **Site Specific Design Spectra.** A site specific response spectrum based on geological, tectonic, seismologic and soil characteristics associated with the specific site. he spectrum shall be developed for a damping ratio of 0.05.

c. **Time Histories.** A series of recorded time histories scaled to match the building period over a range of periods bracketing that of the buildings from -.5 sec to +1.0 sec. A minimum of three time histories shall be used and the maximum drift obtained shall be used, or seven time histories may be used and the average drift obtained considered as the design response.

3. **Mathematical Model.** The three-dimensional mathematical model of the physical structure shall represent the spatial distribution of mass and stiffness of the structure.

4. **Effective Stiffness.**

a. **General.** The nonlinear force displacement relationships for the structural elements of the building shall be used for calculating the time history response.

EXCEPTIONS: 1. Where it can be shown to be appropriate the nonlinear response may be approximated using the effective stiffness of concrete and masonry elements. The effective stiffness shall be calculated as the secant stiffness of the element or system with due consideration of the effects of tensile cracking and compression strain. The secant stiffness shall be taken from substructure models of the force-displacement relationship of the element or system. The force-displacement relationship shall consider the effects of cracking, propagation of cracking and nonlinear material behavior. The force-displacement analysis shall include the calculation of the displacement at which strength degradation begins.

2. The reduction in stiffness of beams and columns in shear wall or frame buildings may be calculated from the ACI formula: given in section of the UBC

c. **Infills.** The stiffness of an infill shall be determined from a nonlinear analysis of the infill and the confining frame. The effect of the infill on the stiffness of the system shall be determined by differencing the force-displacement relationship of the frame-infill from the frame only system.

d. **Model of Infill for the approximate method.** The mathematical model of an infill frame structure shall include the stiffness effects of the infill as a pair of diagonals in the portions of the frame that are infilled. The diagonals shall be considered as having concrete properties and only axial loads. Their lines of action shall intersect the beam-column joints. The secant stiffness of the force-displacement relationship shall be used to determine the effective area of the diagonals. The effective stiffness of the frame shall be determined as specified above.

4. **Effective Stiffness of Elements and Systems for the approximate method.** The effective stiffness shall be determined by an iterative method. The mathematical model using assumed effective stiffness shall be used to calculate dynamic displacements. The effective stiffness of all concrete and masonry elements shall be modified to represent the secant stiffness obtained from the nonlinear force-displacement analysis of the element or system at the calculated displacement. A re-analysis of the mathematical model shall be made using the adjusted stiffness of the existing and supplemental elements and systems until closure of the iterative process is obtained.

5. **Description of the Approximate Nonlinear Analysis Procedure.**

a. **Response spectrum analysis.** An elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal response are calculated using the ordinates of the appropriate response spectrum curve which corresponds to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

b. **Number of Modes.** The requirements of Section X. that all significant modes be included may be satisfied by demonstrating that for the modes considered, at least 90 percent of the participating mass of the structure id included in the calculation of terseness for each principal horizontal direction.

3. **Combining Modes.** The peak displacements for each shall be combined by recognized methods. Modal interaction effects of three-dimensional models shall be considered when combining modal maximal.

4. **Torsion.** The three-dimensional analysis shall be considered as including all torsional effects including accidental torsional effects.

6. **Material Characteristics.**

a. **General.** The stress-strain characteristics of concrete and masonry materials shall be determined by testing or from published experimental data obtained from similar concrete for masonry. Testing shall be done in displacement control to establish the stress- strain relationship for loading to peak stresses and the post peak stress-strain relationship.

b. **Existing Materials.** Existing materials may be sampled from the structure as cores or prisms. The cutting of cores and prisms shall not significantly reduce the strength of the existing structure.

c. **Unreinforced Masonry.** The stress-strain relationship of the existing unreinforced masonry shall be determined by in-place cyclic testing.

d. **Combination of Concrete and Masonry Materials.** Combinations of masonry and concrete infills shall be assumed to have equal strain. The elastic secant moduli of the masonry and concrete shall be used to determine the effective area of the composite material.

7. **Story Drift Limitation.**

a. **Definition.** Story drift is the displacement of one level relative to the level above or below calculated for the design ground motion

b. **Limitation for Life Safety.** The story drift is limited to that displacement that causes any of the following effects:

(1) Compressive strain of 0.003 in the frame confining infill or in a shearwall.

(2) Compressive strain of 0.004 in a reinforced concrete column.

(3) Peak strain in masonry infills as determined by experimental data or by physical testing.

(4) Displacement that was calculated by the nonlinear analysis as when strength degradation of any element began.

> **EXCEPTION:** Item D may be taken as the displacement that causes a strength degradation in that line of resistance equal to 10 percent of the sum of the strength of the elements in that line of resistance.

8. **Compressive Strain Determination.**

a. **General.** The compressive strain in the columns, shear walls and infills may be determined by the nonlinear analysis or a procedure that assumes plane sections remain plane.

b. **Axial Loading.** The compressive strain shall be determined for combined flexure and axial loading. The flexural moments shall be taken from the response spectrum analysis. The axial loads shall have the following combination of effects:

$$U = 1.05\ D + 0.3\ L + 1.0\ E$$

9. **Shear Strength Limitation for Concrete and Masonry Elements.**

a. **General.** The in-plane shear strength of all columns, piers and shear walls shall equal or exceed the shear design shear or the shear associated with development of nonlinear flexural moments in the ends of columns or piers and at the base of shear walls. No strength-reduction factors shall be used in the determination of strength.

b. **Frame Members Subjected to Combined Bending and Axial Loading.** The shear force shall be the lesser of that determined from using the maximum probable moment strength, Mpr, at the base of the shear wall or the design shear loads. Factored axial load as prescribed and all vertical reinforcement in the shear wall shall be used for calculation of the maximum probable moment.

				scale factor
Mass (m)	1	1	0.5	
Stiffness (k)	100	100	100	
Assumed Displacement (xa)	1	1.6	2.0	
Inertial force (mw2xa)	1.0	1.6	1.0	$w^2 a$
Spring Force (Q)	3.6	2.6	1.0	$w^2 a$
Extension (Q/k)	3.6	2.6	1.0	$\frac{w^2 a}{100}$
Displacment (xd)	3.6	6.2	7.2	$\frac{w^2 a}{100}$
Displacment (xd)	1.0	1.72	2.0	$\frac{3.6 w^2 a}{100}$

Figure C-1: Three-story Example

APPENDIX D

ENERGY BASED DESIGN APPROACH

by

Vitelmo V. Bertero

D.1 Overview of Approach

A. *Introductory Remarks*

The information needed to improve the earthquake-resistant design (EQ-RD) of structures can be grouped under the following three basic elements: *earthquake input, demands on structures* and *supplied capacities of structures*. The use of energy concepts is a promising approach for improving solutions of the problems involved with these three elements.

The design earthquake depends on the design criteria, i.e., *the limit state controlling the design*. Conceptually, the design earthquake should be the ground motion that will drive a structure to its critical response. In practice, the application of this simple concept meets with serious difficulties because, first, there are great uncertainties in predicting the main dynamic characteristics of ground motions that have yet to occur at building sites, and second, even the critical response of a specific structural system will vary according to the various limit states that can control the design.

Until a few years ago, seismic codes specified design earthquakes in terms of a building code zone, a site intensity factor, or a peak site acceleration. Reliance on these indices, however, is generally inadequate, and methods using *Ground Motion Spectra* (GMS) and *Smoothed Elastic Design Response Spectra* (SEDRS) based on *Effective Peak Acceleration* (EPA) have been recommended. While this has been a major improvement conceptually, great uncertainties regarding appropriate values for EPA and GMS, as well as other parameters that have been recommended to improve this situation, persist. It is believed that a promising engineering parameter for improving the selection of proper design earthquakes is the use of the concept of *Energy Input, E_I, and associated parameters*. Traditionally, displacement ductility has been used as a criterion to establish *Inelastic Design Response Spectra (IDRS)* for earthquake-resistant design of buildings. The minimum required *strength* (or capacity for lateral force) of a building is then based on the selected IDRS. As an alternative to this traditional design approach, an energy-based method was proposed by Housner [1]. Although estimates have been made of input energy to Single Degree of Freedom Systems, SDOFS [2], and even to Multi Degree of Freedom Systems, MDOFS (steel structures designed in the 60's for some of the existing recorded ground motions) [3], it is only recently that this approach has gained extensive attention [4]. This design method is based on the premise that the *energy demand* during the Earthquake Ground Motions (EQGMs) generated by an earthquake or an ensemble of earthquakes can be predicted, and that the *energy supply* of a structural element (or a structural system) can be established. *A satisfactory design implies that the energy supply should be larger than the energy demand.*

To develop reliable design methods based on an energy approach, it is necessary to derive the energy equations. Although real structures are usually MDOFS, in order to facilitate the analysis and understanding of the physical meaning of the energy approach, it is convenient first to derive the energy equation for SDOFS and then to derive these equations for MDOFS.

B. *Derivation of Energy Equations: Linear Elastic-Perfectly Plastic SDOFS*

In Ref. 5 is a detailed discussion of the derivation of two basic energy equations, starting directly from Eq. 1 for a given viscous damped SDOFS subjected to an EQGM

$$m\ddot{v} + c\dot{v} + f_s = 0 \qquad (1)$$

where: m = mass; c = viscous damping constant; f_s = restoring force (if k = stiffness, f_s = kv for a linear elastic system); $v_t = v + v_g$ = absolute (or total) displacement of the mass; v = relative displacement of the mass with respect to the ground; and v_g = earthquake ground displacement.

<u>Derivation of "Absolute" Energy Equation</u>. Integrating Eq. 1 with respect to v from the time that the ground motion excitation starts and considering that $v = v_t - v_g$, it can be shown that

$$\frac{m(\dot{v}_t)^2}{2} + \int c\dot{v}dv + \int f_s dv = \int m\ddot{v}_t dv_g \qquad (2)$$

$$E_k + E_\xi + E_a = E_I \qquad (3)$$

"Absolute" Kinetic Energy +	Damping Energy +	Absorbed Energy =	"Absolute" Input Energy

Considering that E_a is composed of the recoverable Elastic Strain Energy, E_s, and of the irrecoverable Plastic Hysteretic Energy, $E_{H\mu}$, Eq. 3 can be rewritten as

$$E_I = E_K + E_s + E_\xi + E_{H\mu} \qquad (4)$$

E_I is defined as the "Absolute Input Energy" because it depends on the absolute acceleration, $\ddot{v}_t$. Physically, it represents the inertial force applied to the structure. This force, which is equal to the restoring force plus the damping force (see Eq. 2 and Fig. D-1) is the same as the total force applied to the structure foundation. Therefore, E_I represents the work done by the total base shear at the foundation on the foundation displacement v_g.

Derivation of "Relative" Energy Equation. Because Eq. 1 can be rewritten as

$$m\ddot{v}+c\dot{v}+f_s=m\ddot{v}_g \tag{5}$$

and the structural system of Fig. D-1(a) can be conveniently treated as the equivalent system in Fig. D-1(b) with a fixed base and subjected to an effective horizontal dynamic force of magnitude, $m\ddot{v}_g$, integrating Eq. 5 with respect to v, the following equation can be obtained:

$$\frac{m(\dot{v}_t)^2}{2}+\int c\dot{v}dv+\int f_s dv=-\int m\ddot{v}_g \tag{6}$$

$$E'_K+E_\xi+E_a=E'_I \tag{7}$$

"Relative" Kinetic Energy + Damping Energy + Absorbed Energy = "Relative" Input Energy

As $E_a = E_s + E_{H\mu}$, Eq. 8 can be rewritten as

$$E_I^{'} = E_K^{'} + E_s + E_\xi + E_{H\mu} \tag{8}$$

The E_I' that is defined as the "Relative" Input Energy represents the work done by the static equivalent external force ($-m\ddot{v}_g$) on the equivalent fixed-base system; that is, it neglects the effect of the rigid body translation of the structure.

Difference Between Input Energies from Different Definitions. Reference 5 discusses in detail the differences between the values of the input energies E_I and E_I'. Although the profiles of the energy time histories calculated by the absolute energy equation (Eq. 2) differ significantly from those calculated by the conventional relative equation (Eq. 6), the maximum values of E_I and E_I' for a constant displacement ductility ratio are very close in the period range of practical interest for buildings, which is 0.3 to 5.0 secs.

Comparison of E_I with the Maximum Input Energy That is Stored in a Linear Elastic SDOFS. For a linear elastic SDOFS, the maximum energy that is stored is given by

$$E_{IS} = \tfrac{1}{2}m(S_{pv})^2 \tag{9}$$

where S_{pv} is the linear elastic spectral pseudo-velocity.

This E_{IS} has been used by some researchers as the energy demand for an inelastic system. In Ref. 5, it is shown that E_{IS} may significantly underestimate the E_I for an inelastic system. In this reference, it is also shown that, except for highly harmonic ground motions (like the one recorded at SCT in Mexico City during the 1985 earthquake), the E_I for a constant ductility ratio can be predicted reliably by the elastic input energy spectra using Iwan's procedure [6], which takes into consideration the effect of increasing damping ratio and natural period.

<u>Input Energy to MDOFS.</u> The E_I for an N-story building can be calculated as follows [5]:

$$E_I = \int(\sum_{i=1}^{N} m_i \ddot{v}_{ti})dv_g \tag{10}$$

D.2 Information Needed to Conduct Reliable Performance Based EQ-RD Using an Energy Approach

It has been pointed out that the first and fundamental step in earthquake-resistant design of structures is the reliable establishment of the design earthquakes. This requires a reliable assessment of the damage potential of all the possible earthquake ground motions that can occur at the site of the structure. An evaluation of the different parameters that have been and are still used is offered in Ref. 7. Currently, for structures that can tolerate a certain degree of damage, the *Safety or Survival-Level Design Earthquake* is defined through *Smoothed Inelastic Design Response Spectra, SIDRS.* Most of the *SIDRS* that are used in practice (seismic codes) have been obtained directly from SEDRS through the use of the *displacement ductility ratio,* μ, or *reduction factors, R.* The validity of such procedures has been questioned, and it is believed that at present such SIDRS can be obtained directly as the mean or the mean plus one standard deviation of the *Inelastic Response Spectra, IRS,* corresponding to all the different time histories of the severe ground motions that can be induced at a given site from earthquakes that can occur at all the possible sources affecting the site.

While the above information *is necessary* to conduct reliable design for safety, i.e., to avoid collapse and/or serious damage that can jeopardize human life, *it is not sufficient.* Although the IRS takes into account the effects of duration of strong motion on the required strength, these spectra do not give an appropriate idea of the amount of energy that the whole facility system will dissipate through hysteretic behavior during the critical earthquake ground motion. They give only the value of the maximum strength associated with a maximum global ductility ratio demand (μ). In other words, *for a given yielding strength the maximum global ductility ratio (μ) demand by itself does not give an appropriate definition of the damage potential of ground motions.* In Ref. 7, it has been shown that E_I is a more reliable parameter than those presently used in assessing damage potential. As is clearly shown by Eqs. 2 and 3, this damage potential parameter depends on the dynamic characteristics of both the shaking of the foundation and the whole building system (soil-foundation-superstructure and the nonstructural components). Now the question is: *Does the use of the SIDRS for a specified global μ and the corresponding E_I of the critical ground motion give sufficient information to conduct a reliable seismic design for safety?*

Although the use of E_I can identify the damage potential of a given ground motion, and therefore permits selection, among all the possible motions at a given site, of the critical one for the response of the structure, it does not provide sufficient information to design for the safety level. From recent studies [8 and 5], it has been shown that the energy dissipation capacity of a structural member, and therefore of a structure, depends upon both the loading and the deformation paths. Although the energy dissipation capacity under monotonic increasing deformation may be considered as a lower limit of energy dissipation capacity under cyclic inelastic deformation, the use of this lower limit could be too conservative for earthquake-resistant design. This is particularly true when the maximum ductility deformation ratio, say μ_{max}, is limited (because of the need to control damage of nonstructural components or other reasons), to values that are low compared to the ductility deformation ratio reached under monotonic loading, μ_{mon}. Thus, efforts should be devoted to determining experimentally the energy dissipation capacity of main structural elements and their basic subassemblies as a function of the maximum deformation ductility that can be tolerated, and the relationship between energy dissipation capacity and the hysteretical loading and/or deformation history.

From the above studies, it has also been concluded that damage criteria based on the simultaneous consideration of E_I and (given by SIDRS) and the $E_{H\mu}$ (including *Accumulative Ductility Ratio*, μ_a, and *Number of Yielding Reversals*, *NYR*) are promising for defining rational earthquake-resistant design procedures. From the above discussion, it is clear that when significant damage can be tolerated, the search for a single parameter to characterize the ground motion or the design earthquake for safety is doomed to fail. The information needed to conduct reliable performance-based EQ-RD using a general energy approach follows. The iterative procedure for EQ-RD is similar to that summarized in the flow chart for the Comprehensive Design Approach shown in Fig. B-1.

1. **Select Minimum Performance Design Objective: Establishment of Design Earthquake Hazards**

   Similar to the Comprehensive Design Approach. The following smoothed design spectra are needed:

   - ***For full operation (serviceability)***: the SLEDRS corresponding to the energy input (E_I), strength (PS_a), displacement (S_d), velocity (S_v), and the total acceleration (S_a).

   - ***For operational, life safety and impending collapse (collapse prevention)***: same as above but considering inelastic behavior, i.e., the SIDRS for different values of μ and ξ, adding the smoothed design spectra for the dissipation of energy due to hysteretic plastic deformation, $E_{H\mu}$, and modifying the SIDRS for strength considering the effects of the $E_{H\mu}$. This modification can be made using one of the following two damage functions or damage indices [9].

I. *The One Seismic Parameter Damage Function*

$$D_{E_{H\mu}} \quad = \quad \frac{\mu_e - 1}{\mu_{e,\mu,\,mon} - 1} = \frac{\mu_e - 1}{\mu_{u,\,mon}} \tag{11}$$

where μ_e = equivalent for the $E_{H\mu}$ and is given by the following expression

$$\mu_e = \frac{E_{H\mu}}{F_y X_y} + 1 \tag{12}$$

where F_y = the yielding strength of the structure and x_y = deformation (displacement) at the yielding of the structure.

II. *The Two Seismic Parameter Damage Functions*. For example, the Park and Ang function, $D_{P.A.}$,

$$D_{P.A.} = \frac{x_{\max}}{x_{u,mon}} + \beta\frac{E_{H\mu}}{F_y x_{u,mon}} = \frac{\mu + \beta(\mu_e - 1)}{\mu_{u,mon}} \tag{13}$$

where for RC structures the parameter β depends on the value of shear and axial forces in the flexural member, as well as on the amount and detailing of the longitudinal and transverse reinforcement [10].

Figure D-2 illustrates the comparison of the resulting values for the collapsing strength (pseudo-acceleration spectra) of an elastic perfectly plastic model characterized by an ultimate monotonic ductility ($\mu_{u,\,mon}$) equal to 4.

From the results presented in Fig. D-2 it becomes clear that while the use of the so-called cinematic may represent satisfactorily the performance of the structure in case of ground motions and building response characterized by just one cycle of large plastic deformation, the use of a damage function based on just the E_H can lead to very conservative values for the required strength in the case of EQGMs characterized by long duration of strong motions like the one recorded at the SCT station during the 1985 Ixtapa, Mexico, earthquake.

2. **Conduct Suitability Analysis of Selected Site.** According to the minimum performance design objectives (established design earthquake hazards) conduct a site suitability analysis (use microzonation maps if available).

3. **Conduct Conceptual Overall Design (or Conception) of the Entire Building System and its Acceptability Checks.** This step is similar to that for Comprehensive Design, i.e., use the guidelines given in Section B.4.

4. **Numerical Preliminary Design**. Ideally, this step should be identical to that recommended for the Comprehensive EQ-RD Approach (Section B.5), i.e., determine first the maximum acceptable fundamental period, T_{max}, required to satisfy the acceptable limits for the deformation, and deformation rates of the building at each of the performance levels or at least at those that appear to control the design. Once the T_{max} has been estimated, the ideal would be to follow the procedure recommended for the Comprehensive EQ-RD Approach, i.e., carry out first a preliminary sizing for stiffness and then complete or revise the preliminary sizing to satisfy the strength requirements. For designers unfamiliar with design for stiffness, an iterative preliminary design procedure based on just strength similar to that recommended for the general force/strength approach discussed in Appendix E can be used. The only difference from the force/strength approach is that rather than determining the required strength using the SIDRS for strength, it is determined using the spectra illustrated in Fig. D-2, corresponding to the desired degree of damage using the damage functions based on the expected $E_{H\mu}$.

   To summarize, the numerical preliminary design in the energy-based EQ-RD is practically identical to that used in the force/strength approach, except that instead of using a $R_{\mu,E_{H\mu}}$ in the evaluation of the R factor this value is already included in the established SIDRS as illustrated in Fig. D-2.

5. **Analysis of the Performance of the Preliminary EQ-RD: Acceptability Checks**. As indicated in flow chart for Comprehensive EQ-RD (Fig. B-1), it is necessary to determine if the preliminarily designed building will perform acceptably according to the desired or selected performance levels under the associated EQGMs established in the adopted design criteria (performance design objectives). If the preliminary sizing has been conducted directly based on strength and not on the lateral stiffness required to comply with the acceptable T_{max}, then the first acceptability check will be to compute the fundamental period of the preliminarily designed structure and check if it is equal to or smaller than the acceptable T_{max}.

   Because this method is based on the consideration of the effects of μ and $E_{H\mu}$ on the damage to the building, it is necessary to conduct nonlinear time-history analyses of the preliminarily designed entire building system. Ideally, 3D dynamic nonlinear time-history analysis should be conducted. If no computer program or facility is available for conducting such 3D analysis, at least a 2D dynamic nonlinear time-history analysis should be conducted to estimate the demanded hysteretic response of the critical regions of the structure (critical regions of beams, columns, walls, braces and of their joints and supports). These nonlinear analyses are required when checking acceptability for the performance levels involving significant damage to the building, particularly for the levels of life safety and impending collapse. For the acceptability checks corresponding to the full operational (serviceability) level linear elastic methods of analysis can be used to check that the deformation, deformation rates and the stress ratios do not exceed the established acceptable values.

6. **Final Design and Detailing and Acceptability Checks.** These are similar to those required for the Comprehensive EQ-RD approach and are discussed in detail in Appendix B.

D.3 Advantages of the General Energy Approach

Equations (4) and (8), which can be rewritten as:

$$E_I = E_E + E_D \tag{14a}$$

$$E_I = E_K + E_S + E_{H\xi} + E_{H\mu} \tag{14b}$$

where E_E can be considered as the stored elastic energy and E_D the dissipated energy. Comparing this equation with the design equation

$$\text{DEMANDS} \leq \text{SUPPLIES} \tag{15}$$

it becomes clear that E_I represents the *demands*, and the summation of $E_E + E_D$ represents the *supplies*. Thus, Eq. (14a) points out clearly to the designer that to obtain an efficient seismic design, the first step is to have a good estimate of the E_I for the critical ground motion. Then the designer has to analyze if it is possible to balance this demand with just the elastic behavior of the structure to be designed, i.e., with just E_E, or will it be convenient to attempt to *dissipate* as much as possible of the E_I using E_D. As revealed by Eq. (14b), there are three ways to increase E_D: one is to increase the linear viscous damping, E_ξ; another is to increase the plastic hysteretic energy, $E_{H\mu}$; and the third is a combination of increasing E_ξ and increasing $E_{H\mu}$. At present it is common practice to try to increase just the E_D as much as possible through inelastic plastic behavior, which implies damage of the structural members. Only recently has it been recognized that it is possible to increase the E_D significantly and to control damage through the use of *energy dissipation devices* inserted at properly determined locations throughout the structure. Increasing E_D by increasing E_ξ rather than E_H has the great advantage that it can provide control of the structure's behavior throughout all the limit states. Increasing E_D by just increasing , i.e., increasing just $E_{H\mu}$, will not improve behavior at the service (full operational) level.

If technically and/or economically it is not possible to balance the required E_I through either E_E alone or $E_E + E_D$, the designer has the option of attempting to control (decrease) the E_I to the structure. This can be done by *base isolation techniques.* A combination of controlling (decreasing) the E_I by base isolation techniques and increasing the E_D by the use of energy dissipation devices is a very promising strategy not only for achieving efficient earthquake-resistant design and construction of new structures, but also for the seismic upgrading of existing hazardous structures.

D.4 Disadvantages of General Energy Approach

The main disadvantages follow.

- To use this energy approach reliably, it is essential to be able to select the critical ground motion (design earthquake) i.e., the ground motion that has the largest damage potential for the structure being designed and thus controls the design. Although many parameters are used to establish design earthquakes, most of them are not reliable for assessing the damage potential of earthquake ground motions. As mentioned before, a promising parameter for assessing the damage potential of these motions is E_I. However, *this parameter alone is not sufficient to evaluate (visualize) the E_D* (particularly $E_{H\mu}$) that has to be supplied to balance the E_I for any specified acceptable damage. Additional information is needed regarding the variations of $E_{H\mu}$ with the characteristics of the hysteretic behavior (strength-deformation) of the entire building system. Most of the information available is for structures having a linear elastic perfectly plastic hysteretic behavior.

- For conducting the design it is necessary to select reliable values of the damage functions. There are not sufficient data available regarding the effects of the hysteretic behavior due to plastic deformations, i.e., the $E_{H\mu}$, on real structures. Thus, there are at present great uncertainties regarding how to establish reliable values for the damage functions, as well as the acceptable values that can be used in the acceptability checks for the different performance levels.

D.5 Research, Development, and Implementation Needs

The research and development needs are that same as those for the Comprehensive Design Approach. It is of paramount importance to obtain reliable experimental data on the $E_{H\mu}$ for the different structural members and their subassemblies when fabricated using the different structural materials and considering all possible loading and/or deformation time histories.

D.6 References

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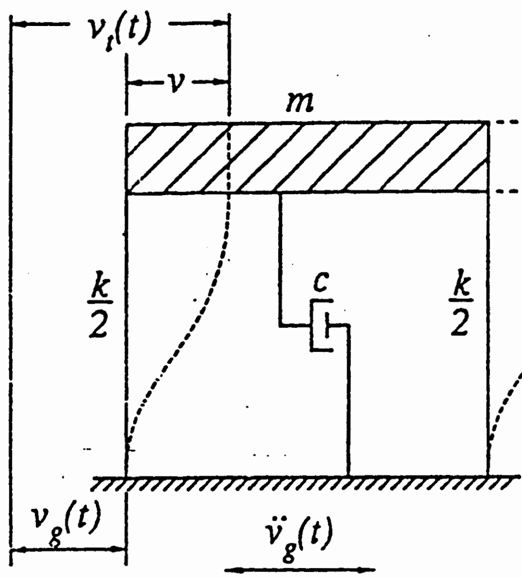
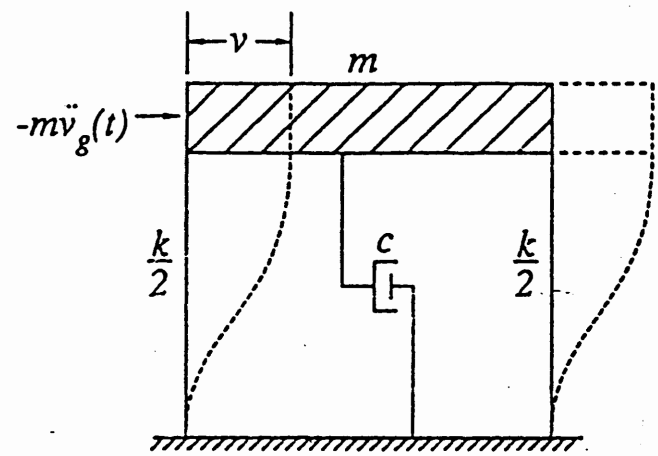


(a) Moving Base System



(b) Equivalent Fixed-Base System

Figure D-1: Mathematical model of a SDOFS subjected to an earthquake ground motion

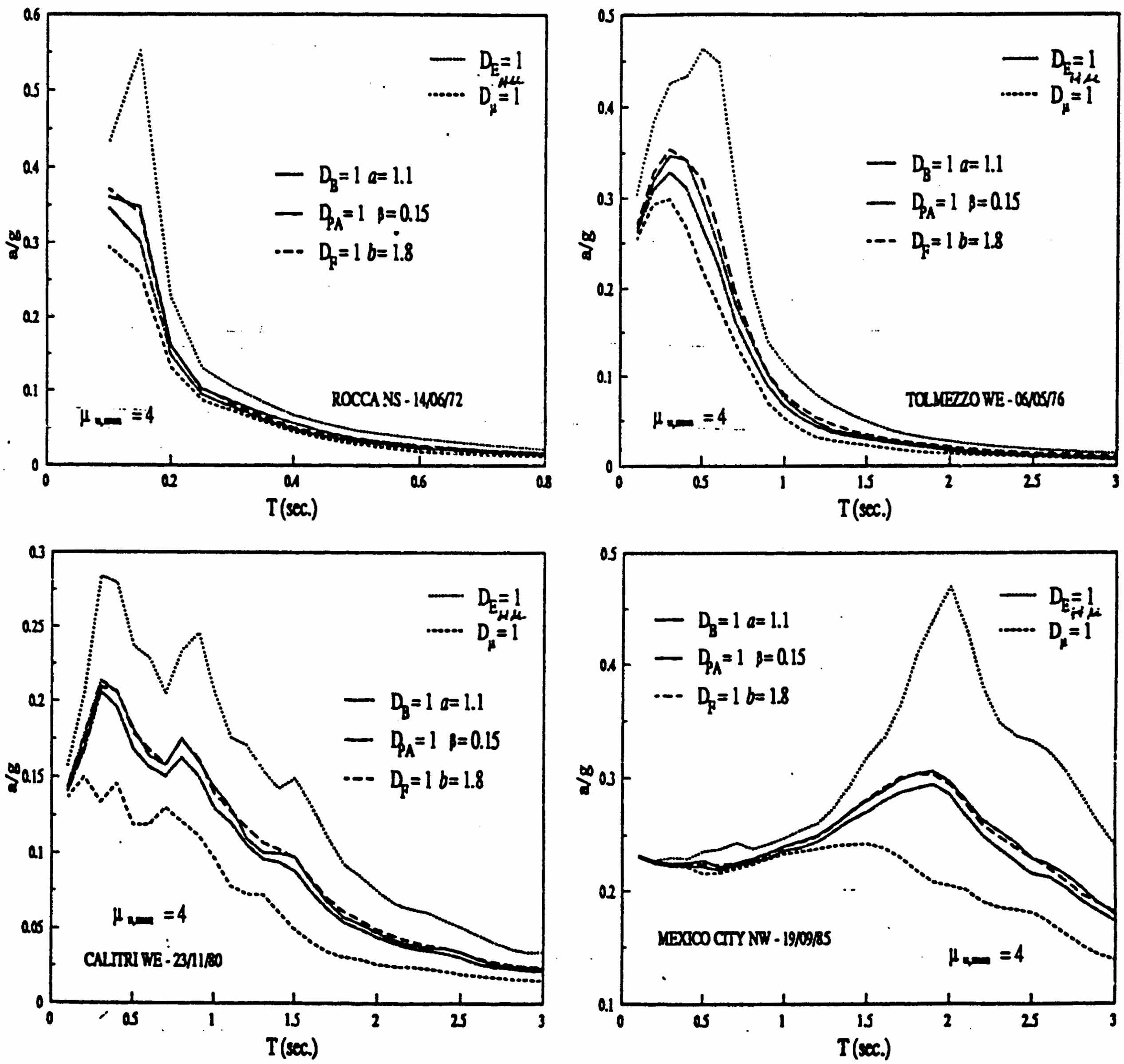


Figure D-2: Comparison between different damage functions with mean values of parameters

APPENDIX E

GENERAL FORCE/STRENGTH DESIGN APPROACH

by

Vitelmo V. Bertero

E.1 Overview of Approach

At present the most commonly used approach for earthquake-resistant design (EQ-RD) of buildings is based on supplying the structure with a minimum required base shear strength, using lateral forces distributed over the height of the building according to a specified or computed pattern. This "force/strength" approach is the approach specified in present seismic regulations of US building codes. Because in this approach all the numerical designs and analyses involved in the overall design procedure of a structure are based on direct use of force, in the available literature this force/strength approach is usually referred to as the *"Lateral Force Approach"* or the *"Equivalent Lateral Force (ELF) Approach."* Depending on how the lateral forces to be used in the numerical designs and analyses of this overall design approach are defined (computed), the following two different basic methods have been developed and used in practice in applying the force/strength approach to buildings:

- Static Equivalent Lateral Force Method; and
- Dynamic Lateral Force Method.

Static Equivalent Lateral Force Method. This is the simplest method of those commonly used for seismic analysis and design of engineered regular low- to medium-rise structures. This static ELF analysis method is fairly easy to understand and implement because it is based on the simple equation, $F = Ma$. For very tall, more complex structures (irregular horizontal or vertical configuration), the use of dynamic analysis methods should be considered.

The static ELF method assumes that the critical dynamic cyclic response to earthquake ground motions of a structure with a reasonably regular configuration can be adequately estimated using static equivalent lateral forces. For a given structural system, the design base shear is estimated based on the assumption that the structure will undergo several cycles of inelastic deformation during major seismic motion. The force (base shear) level is related to the type of structural system and its assumed capacity to undergo these inelastic deformations and dissipate the input energy at its foundation without collapse. The base shear is then distributed over the height of the structure, usually based on an assumed linear mode shape (inverted triangle).

Dynamic Equivalent Lateral Force Method. The basic differences between a static ELF analysis and an ELF dynamic modal analysis are in the calculation of the fundamental vibrational period of the structure, the calculation of forces, and the distribution of the lateral force (base shear) over the height of the structure. For the static ELF, the period T is generally calculated using a semi-empirical formula based on the type of framing and the construction material used. In the dynamic ELF modal analysis, the natural vibrational modes are calculated using principles of mechanics. With ELF, the lateral forces are calculated assuming that they are acting nonconcurrently, i.e., in one direction at a time using two-dimensional (2D) analysis. However, three-dimensional (3D) analysis using a 3D model can also be performed.

With the dynamic ELF modal analysis, the vertical distribution of lateral forces is based on the natural vibrational modes, which are determined from the actual mass and stiffness distributions over the height of the structure.

Dynamic analysis can provide a more accurate representation of the distribution of the lateral forces in a structure. However, the accuracy of the results depend on the realism of the mathematical mode, and thus a certain percentage (90-100) of the ELF base shear is used as a lower bound.

E.2 Advantages of the Force/Strength (ELF) Approach

As already noted above, this is at present the most commonly used approach for EQ-RD of buildings. The two methods used in this approach are based on the use of the force/stress concept rather than the deformation/strain concept, and have been used for decades because they are considered fairly easy to understand because they are consonant with the way the average professional engineer has been educated. The two numerical methodologies currently used in this EQ-RD approach, the static and dynamic, are summarized in Appendix C. It has to be noted that if this approach is used to conduct performance-based EQ-RD, it is necessary to predict the different levels of damage (limit states) associated with the adopted performance levels. Levels of damage are more a consequence of deformations, elongations or distortions of the structural elements than of the intensity of the force or the strength of the element. Thus, the question to be answered is, *Can force, strength or stress be used as a reliable measure of deformation or strain?* If the response of the building remains in its linear elastic range of behavior, the answer is yes, but if the building is subjected to significant nonlinear inelastic deformations the answer is no, except for very special cases.

The above discussion shows that it will not be a problem to use this ELF approach when the performance-based EQ-RD is controlled by the desire for the building to remain serviceable even under the maximum credible EQGM. On the other hand, it would appear that strictly speaking it will not be possible to use this approach when the performance design objective controlling the EQ-RD is associated with life safety or impending collapse performance level. However, from the observed behavior of buildings which have been designed according to current ELF approach in numerous damaging EQGMs, it would appear that for many regularly (or nearly so) configured buildings, this current approach is adequate. This apparent paradox is explained in the results presented in Appendix A, where it is shown that under very restrictive types of building, not only regarding this configuration but also regarding the type of EQGMs and the effects of other loading acting on the building, it is possible to use forces and/or moments as measures of deformations (displacements and/or rotations). As the restrictions that would have to be imposed on the types of buildings to which current (1994 UBC and NEHRP) ELF approaches could be applied successfully are very large, the ideal is to modify the current approach to make it more applicable. The needed modifications are discussed below and in Appendix C.

E.3 Disadvantages of the Force/Strength (ELF) Approach

As discussed above, the ELF approach as adopted by current seismic codes, when analyzed in the light of the new proposed performance-based EQ-RD philosophy or Comprehensive Design philosophy, has some disadvantages, of which the main are the following.

1. The design is based on just one performance objective, and thus considers just one level of EQGM, which is usually the one associated with a recurrence period of 475 years.

2. Assuming linear elastic behavior, the strength for which the numerical EQ-RD is conducted is based on reducing the strength required through a strength reducing factor, R, that depends only on the type of structural system, which has been shown not to be realistic.

Modifications of Current ELF Approach to Adapt It to Performance-Based EQ-RD Philosophy. To eliminate the above disadvantages, the following improvements or modifications, discussed in detail in Appendix C, are proposed. It should be noted that with these modifications the new force/strength approach can be applied to all types of structures. A simplified force/strength approach that can be applied to certain simple regular building structures is discussed in Section 3.4 and Appendix D.

1. ***Introduce the four performance design objectives considered in the recommended comprehensive design approach (Chapter 2 and Figs. 2.1 and 2.2).***

2. ***Define the strength and displacement SIDRS.*** According to the seismic region and the site conditions, define the strength and displacement design spectra for each of the four performance design objectives. It should be noted that the strength SIDRS are based on the deformation ratio, , that can induce the acceptable damage level (or damage index), rather than just on the maximum that the building can resist under monotonically increasing deformation. Alternative methods of specifying (rather than defining) the strength SIDRS for each of the four design objectives are given in Appendix C.

3. ***Parallel to 2.: Conduct conceptual overall design,*** and then in accordance with 1. and 2. check the acceptability of this design.

4. ***Estimate the fundamental period, T, of the building to be designed,*** use the displacement SIDRS to determine the maximum acceptable T_{max} that can be used for each of the four performance design objectives.

5. ***Select what appear to be the controlling performance design objectives, and conduct the preliminary design.*** Based on the spectra defined in 2. and the T_{max} found in 4., select at least two design objectives, Service (or functional) and Life Safety (or impending collapse), to conduct the Preliminary EQ-RD using ELF methods. Using these methods, the preliminary design is conducted using demanded forces (strength) rather than the minimum required stiffness that is used in the case of the comprehensive design approach. *This is the main difference between these two approaches.*

6. ***Conduct acceptability checks of the preliminarily designed building.*** As indicated in the flow chart of Fig. 2.1, determine if the preliminarily designed building will perform acceptably according to the desired performance levels under the associated EQGMs established in the adopted design criteria (objectives). The following different methods of structural and stress/strain analyses can be used:

   - a check of the stress ratios, using the results from 3D linear elastic dynamic analysis;
   - the use of static linear and nonlinear analyses (pushover analyses), and
   - the use of nonlinear dynamic time-history analyses, using either MDOFS or equivalent SDOFS.

Selection of the method of analysis depends on the performance level (limit state) that is checked. For acceptability checks corresponding to the *service performance level*, where no significant damage should be permitted, linear elastic methods of analysis can be used to check that the deformation, the rate of deformation and the stress ratios do not exceed the established acceptable values.

For the other performance levels, particularly *Life Safety and Impending Collapse*, where significant inelastic deformations are accepted, the ideal method of analysis is the nonlinear dynamic time history. As discussed in Appendices A, C and D, only in the case of simple regular low-rise buildings in which EQGMs are the main type of excitations controlling their design, linear elastic methods may be used. However, it would be highly desirable to use at least the static nonlinear pushover method of analysis discussed in Section 4.5, according to the following procedure.

(i) Conduct a linear elastic analysis (ideally a 3D) and determine the periods, T_i, and modes of vibration of the building.

(ii) Conduct a nonlinear pushover analysis of a building from which the lateral strength ($V_{max} \cong V_{yielding}$) is estimated.

(iii) With T_1 and V_{max} and using the established *strength (C_s) SIDRS* for the different performance levels involving significant damage (i.e., Life Safety and Impending Collapse) estimate the maximum required global ductility ratio, μ_g, for the structure when it is subjected to the critical EQGM associated with each of these performance levels.

(iv) With T_1 and the maximum required μ_g and using the established displacement (S_d) SIDRS for the different performance levels, estimate the maximum required displacements at a certain selected height of the building (usually the roof) when it is subjected to the critical EQGM associated with each of these performance levels. Check degree of agreement between these estimated S_d and those obtained using the estimated μ_g directly with the lateral resistance (V)-displacement (Δ) curve obtained from the pushover nonlinear analysis.

(v) Using the results obtained in step (iv), check the local performance of the critical structural members, i.e., the critical regions where the inelastic deformations are developed for each of the performance levels that need to be considered. It has to be noted that unfortunately the nonlinear pushover method in general does not give an adequate estimation of the concentration of lateral story deformation, and therefore local deformation, that can occur as a consequence of the effects of nonlinear vibrations in the higher modes. Furthermore, nonlinear pushover analysis does not give any idea of the actual dissipation of energy due to the effects of repeated cycles of inelastic (plastic deformations). As discussed below, there is a need to find reliable modification factors that can be used to correct the results of a nonlinear pushover analysis as a consequence of these above two effects.

(vi) Once it is judged that the preliminarily sized building complies with the main acceptability criteria, or that even in the case that it could not strictly satisfy all the requirements, compliance with these missing requirements could be achieved by improvements in the detailing of the members, the final design and detailing and the necessary acceptability checks can be conducted as for the case of the Comprehensive EQ-RD, as is indicated in the flow chart of Fig. 2.1, except that the only nonlinear method of analysis that is used is the static pushover method.

E.4 Research, Development and Implementation Needs

From the above discussion, it is clear that the general improved or modified force/strength approach proposed above for conducting performance-based EQ-RD is very similar to the proposed Comprehensive EQ-RD approach, except for the following two main aspects.

(1) Preliminary design (sizing) of members is conducted based on the minimum required lateral strength for each of the main limit states (performance levels), rather than on the minimum lateral stiffness as in the comprehensive EQ-RD approach.

(2) The acceptability checks of the preliminary design and final design are conducted using linear elastic and nonlinear pushover methods of analysis.

In view of this above similarity, the research, development and implementation needs for improving the reliability of the practical applications of this force/strength approach for achieving EQ-RD of buildings with predictable performance are the same as those presented in Section 2.12. Because the preliminary design is based on the use of lateral forces which involve use of displacement and strength SLEDRS and SIDRS, it is of utmost importance to carry out comprehensive studies to improve:

1. The development of reliable displacement and strength smoothed elastic and particularly inelastic response spectra for each of the four minimum required performance design objectives.

2. The identification, definition and therefore specification of the components of the strength modification factor, R, particularly the components due to overstrength (R_{OVS}) and dissipation of energy due to plastic deformation, i.e., $R_{\mu,E_{H\mu}}$. Advantages and disadvantages of referring the R to the first significant local yielding of the most critical stressed members, or to the yielding resistance of the entire structural system, should be investigated further.

3. The calibration and therefore specification of the factors that need to be introduced to consider the effects of:

    - elastic (β) and inelastic (β_3 and β_4) torsion, and
    - concentration of IDI and demands at one story due to the effects of higher modes in the case of MDOFS effects (β_2).

Since the acceptability check for the performance levels that involve significant structural damage is based on the use of the static nonlinear pushover methods of analysis, there is a need to conduct the studies identified in Section 4.5 of this report and in recent publications by Krawinkler and his research associates. Among the identified studies, the following should be emphasized:

1. Identification of the proper (most adequate) location for the target displacement.

2. Identification of reliable lateral load patterns that have to be used: constant load patterns vs. changing the incremental load pattern after each load-displacement increment.

3. Study advantages and disadvantages of using a method of analysis based on predetermined displacement increments rather than that based on lateral load increments.

4. Evaluation of the reliability of the equation for estimating the target displacements that have been proposed by Krawinkler and studies to calibrate the different coefficients involved in such equations.

5. Study of the sensitivity of the results obtained from the nonlinear pushover analysis regarding:

- the variation in the effects of other loads that can act simultaneously with the seismic forces, particularly on the local performance demands; and
- the effects of repeated cycles with reversals of inelastic deformations.

COMMENTARY

c.1 Introductory Remarks

As pointed out in Section E.1, this proposed General Force/Strength Approach is based on the approach currently most used for EQ-RD of buildings, which in the available literature is usually referred to as the *"Lateral Force Approach"* or the *"Equivalent Lateral Force (ELF) Approach."* This current approach is based on designing the building structure against the effects of lateral seismic forces/stresses, rather than against demanded deformation/strains. Applications of this approach usually employ linear elastic methods of analysis only. Because force/stress concepts are considered to be fairly easy for the average professional engineer to use, attempts have been made to use these concepts in the development of the following two force/strength approaches which can be used for performance based EQ-RD:

- *General Force/Strength Approach*
- *Simplified Force/Strength Approach*

While the first approach is very similar to the comprehensive EQ-RD approach although somewhat simplified, the second is a very simplified approach that, like the current seismic code approach, requires only the use of linear elastic methods of analysis. This second simplified approach is summarized in Appendix F and discussed in more detail in the commentary to this appendix.

c.2 General Force/Strength Approach for Performance Based EQ-RD Design

This approach, which is similar to the Comprehensive Design Approach, follows the iterative procedure illustrated in the flow chart of Fig. B-1. Thus, herein only its main differences from the Comprehensive Design Approach and its iterative procedure will be discussed in detail.

- *Check Suitability of Site, Discuss with Client the Performance Levels, and Select the Minimum Performance Design Objectives.* These two first steps of the iterative procedure, which have to be conducted simultaneously, are exactly the same as for the Comprehensive Design Approach discussed in Section B.2 and B.3.

- *Conceptual Overall Design (or Conception) of the Entire Building System and Its Acceptability Checks.* Same as discussed in Section B.4.

- *Numerical Preliminary Design.* Although similar to the Comprehensive Design Approach described in Section B.5, some basic differences are pointed out and discussed in detail below.

The flow chart of Fig. B-3 illustrates the main phases that should be considered in the overall EQ-RD design procedure. As can be seen, the iterative procedure starts with a preliminary design and ends with a final design and detailing.

The preliminary design is divided into two main phases:

- *establishment of the design EQs or EQGMs*
- *numerical design procedure*

Establishment of the Design EQGMs: This step is very similar to the one used for the Comprehensive EQ-RD Approach (see Section B.5 and Fig. B-4). The available data (time history of the EQGMs that are expected in the future with different return periods, T_r) are processed to obtain their strength and displacement SLEDRS and the SIDRS for the performance design objectives that are assumed to control the design. Usually at least two levels of EQGMs should be considered, which should be the ones associated with the following performance levels: Service (or Functional) and Life Safety (or Impending Collapse). Although for applying this proposed Force/Strength Approach it may be sufficient just to process the time histories of the EQGMs to obtain the strength (C_s) and displacement (S_d) SLEDRS and SIDRS, as will be discussed later it is highly desirable, conducting the design for the performance design objectives involving damage, to obtain in addition the corresponding $E_{H\mu}$ and γ spectra.

Numerical Preliminary Design Procedure: Although the proposed procedure is based on the same idea as the force/strength procedures specified in current seismic codes, there are some major differences. To clarify these differences, it is convenient first to summarize briefly the current seismic code force/strength procedures (1994 UBC and 1994 NEHRP).

Static Equivalent Lateral Force (ELF) Procedure

1. ***Estimation of seismic forces.*** For regular buildings, the lateral seismic forces can be derived as follows:

   (a) Base shear:

$$V = C_s W = \frac{C_{sp}}{R} W \qquad (1)$$

   where V is base shear; C_s is the design seismic coefficient; W is the weight of the reactive mass (i.e., the mass that can induce inertial forces); C_{sp} is the seismic coefficient equivalent to a linear elastic response spectra acceleration S_a ($C_{sp} = C_s R = S_a/g$); and R is the reduction factor.

   Estimation of the Fundamental Period of the Building. To determine the C_s or C_{sp}, it is necessary to estimate the T_1. This is done according to one of the following procedures.

   - T should be calculated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The 1994 UBC allows that the requirement may be satisfied using the following formula:

$$T = 2\pi\sqrt{(\sum_{i=1}^{N} w_i\delta_i^2)/(g\sum_{i=1}^{n} f_i\delta_i)} \tag{2}$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of formulas (5), (6) and (7) or any other rational distribution. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i.

- The value of T may be approximated from the following formula:

$$T_a = C_t(h_n)^{3/4} \tag{3}$$

where:
C_t = numerical coefficient that depends on the type of structural material and structural system that is used, and
h_n = the height in feet above the base level to the uppermost level of the main portion of the structure.

- For concrete and steel moment-resisting frame buildings not exceeding 12 stories in height and having a minimum height of 10 ft.,

$$T_a = C_t(h_n)^{3/4} \tag{4}$$

where N = number of stories.

Determination of the Reduction Factor R. The values for this factor, R, which the 1994 UBC, following the 1990 SEAOC recommendation, designates as R_w because it reduces the required seismic forces assuming linear elastic response to the allowable (or *working*) stress design basis, and which SEAOC refers to as a *system quality factor or structural system factor*, are given in tables and are constant values which vary only with the type of the lateral structural system and the structural material. As the reliability of the values assigned for R by NEHRP and for R_w by UBC has been seriously questioned, a methodology for improving the establishment of the values of R is introduced in the modified Force/Strength Approach proposed herein.

(b) Distribution of base shear over the height of the structure: the total force, i.e., the base shear, V, given by Eq. (1) shall be distributed over the height of the structure in conformance with formulas (5), (6) and (7).

$$V = F_t + \sum_{t=i}^{n} F_i \tag{5}$$

The concentrated force, F_t, at the top, shall, in addition to F_n, be determined from the formula

$$F_t = 0.07TV \leq 0.25V \tag{6}$$

F_t may be considered the zero where the period T 0.07 secs. The remaining portion of V shall be distributed over the height of the structure, including level N, according to the following formula:

$$F_i = \frac{(V - F_t)w_i h_i}{\sum_{i=1}^{n} w_i h_i} \tag{7}$$

where F_i is the force at level *i* (usually the floor level), w_i is the portion of W located at or assigned to level *i*, and h_i is the height above the base to level *i*.

2. ***Estimation of structural response to seismic forces.*** Structural response can be estimated using linear elastic analyses, either directly using the above statically equivalent lateral forces [Formulas (5)-(7)], or multiplying them by load factors, depending on the value of R that is used and whether the design will use allowable (service) stress or strength methods. As is discussed below, for the adaptation of this force/strength approach to performance-based EQ-RD, the use of the strength method and capacity EQ-RD philosophy is highly recommended.

   The static ELF procedure may be used:

   - for regular structures under 240 feet in height
   - for irregular structures not more than five stories or 65 feet in height
   - for structures having a flexible upper portion supported on a rigid lower portion where both portions of the structure considered separately can be classified as regular

   (a) Dynamic Lateral Force Procedure: The 1994 UBC specifies the following two procedures:

   - *Response Spectrum Analysis*
   - *Time-History Analysis*

   The methods of analysis used in these two procedures are based on linear elastic response and use accepted principles of dynamics.

Response Spectrum Analysis. An elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions are combined statistically to obtain an approximate total structural response.

Time-History Analysis. An analysis of the dynamic response of a structure at each time increment when the base is subjected to a specific ground motion time history.

As in the case of the static ELF procedure, the above two dynamic lateral force procedures consist in estimating the linear elastic response base shear, which then may be reduced to a *design base shear, V*, by dividing it by a factor not greater than the appropriate R (or R_w) factor for the structure. As pointed out in Section B.3, because the primary performance design objective considered in current seismic codes is to provide life safety under major EQGMs having a 475- year return period, the current strategy for achieving this primary performance design objective based on the use of the force/strength approach consists mainly in:

- designing a lateral force-resisting system with a lateral strength equal to or exceeding the minimum required

  $V = (C_{sp} / R)W$;

- comparing the lateral drifts in the building computed under the above minimum reduced forces multiplied by a deflection multiplication factor, C_d, with the maximum permissible drifts. If the computed drifts exceed prescribed limits, stiffen the structure to ensure compliance; and
- satisfy the prescribed detailing requirements for selected structural system and material type.

To extend the application of the basic ideas of the current force/strength procedure to the formulated performance-based EQ-RD, it is necessary to introduce a series of modifications of which the main ones are discussed below.

c.3 Modifications of Current Seismic Code Force/Strength Procedure to Comply with the Four Main Levels Involved in the Minimum Accepted Performance Design Objectives, i.e., Service under Minor EQGMs, Functionality under Moderate EQGMs, Safety under Major EQGMs and Impending Collapse under Extreme EQGMs

As indicated in Section E.3 and early in this commentary, once these four minimum performance design objectives have been selected, it is necessary to establish (define) not only the strength, C_s, design spectra (SLEDRS for the service performance levels and the SIDRS for the other three performance levels) but also the corresponding displacement, S_d, design spectra as well as the corresponding $E_{H\mu}$ and γ spectra. The following are different ways of specifying these spectra:

- Specify for each performance level different possible spectra, as illustrated schematically in Fig. E-1. Note that the reduced strength and displacement spectra for the three performance design objectives that permit structural damage are based on the value of acceptable damage function or damage index (DMI), and not just a function of the maximum ductility ratio, .
- For the strength design spectra, specify the reduced strength spectra for each performance level involving structural damage, i.e., the SLEDRS corresponding to the associated EQGMs divided by an R factor, as is done in the current seismic code force/strength approach, but obtain the specified values of R by considering all the important parameters that can affect such values as discussed below.

Proposed definition of R: To improve evaluation of the R factors, it is proposed to compute them using the following formula:

$$R = R_{\Delta T} \times (R_{OVS} + 1) \times R_{\mu,E_{H\mu}} \times R_{E_{H\Delta\xi}} \tag{8}$$

where:

$R_{\Delta T}$ = a reduction factor (or amplification factor if R<1.0) which depends on the effects of the change in the periods, T, of the entire structural system as this system undergoes damage responding to the EQGM time history.

R_{OVS} = an overstrength reduction factor depending on: the structural system (particularly lateral resistance redundancy and toughness; the building's fundamental period; the effects of the other loads (excitations) that can act simultaneously with the seismic excitations and/or can affect the design of the structure; and the design methods (allowable stresses, strength, limit or plastic design) used in the design of the entire system.

$R_{\mu,E_{H\mu}}$ = is an acceptable ductility ratio (μ_{accep}) reduction factor which considers that the acceptable ductility is controlled by not just the maximum monotonic deformation ductility ratio (μ_{mon}) that the structure can undergo, but also by the demanded energy dissipation due to plastic deformation, $E_{H\mu}$. This $R_{\mu,E_{H\mu}}$ depends on: the acceptable degree of damage, duration of the strong EQGM, the period of the entire building system, and the mechanical characteristics of the entire structural system (particularly on its lateral resistance redundancy and toughness), and thus it is interrelated with R_{OVS}. This $R_{\mu,E_{H\mu}}$ could also be denoted a R_{DMI}, because the acceptable value of $R_{\mu,E_{H\mu}}$ depends on the Damage Index, DMI.

$R_{E_{H\Delta\xi}}$ = is a damping reduction factor which depends on the amount of energy that can be dissipated through a possible increase $\Delta\xi$ of the equivalent hysteretic viscous damping coefficient, ξ, that was considered in the establishment of the SLEDRS, i.e., the C_{sp}.

If this new proposed definition of R is used, then given that $R_{\mu,E_{H\mu}}$ refers to the lateral strength or resistance of the entire system as it yields as a mechanism, it would be more rational to refer the R to this level of lateral resistance, as illustrated in Fig. E-2, rather than to the resistance of the first significant local yielding of the most critical stressed member, as was done originally in ATC 0-3 when the R factor was introduced for the first time. As indicated in this figure, if the R factor is referred to the yielding of the entire structural system is necessary to change the values that can be assigned to the R_{OVS}.

Specification of the R values:

- *Specify for each of the adopted performance levels (performance design objectives) the spectra values for each of the above different components involved in Eq. (8).* Although this looks like the more rational approach, there is the danger that the designer may be tempted to use just the combination of values that results in the larger possible value of the overall resulting R without analyzing the reliability of each of these values, particularly when they are combined together.

- *Specify just the spectra values of the total R for each performance level* (i.e., $R_{service}$, $R_{function}$, R_{safety} and $R_{imp.coll.}$) *and for the minimum acceptable degree of redundancy* (i.e., number of bays that are used as lateral-resisting system). These minimum values could be increased as the degree of redundancy is increased.

- *Specify just one constant conservative value for the total R for each of the adopted performance levels*, i.e., $R_{service}$, $R_{function}$, R_{safety} and $R_{imp.coll.}$.

  One way to simplify the application of this general proposed force/strength factor is to develop for each possible different site condition what can be termed the strength smoothed design spectra for the performance-based preliminary EQ-RD. As illustrated in Fig. E-3, what is specified is just the SLEDRS that corresponds to the extreme (maximum credible) EQGMs corresponding to the impending collapse performance level and then the reduced strength spectrum (through specified values of R factors) for each of the four performance design objectives. Obviously, to develop these spectra it would be necessary to conduct extensive studies regarding the values of R that have to be used.

Preliminary Design for Strength: The preliminary sizing of the structure is conducted based on the minimum required strength. Although this minimum required strength can be estimated using one of the above procedures (static or dynamic), it is recommended that for this general force/strength approach the dynamic procedure be used. The preliminary sizing can be carried out using one of the following methods:

- designing for just one limit state, the one that appears to control the design, and then checking for the others (for tall buildings in the Bay Area and Los Angeles, service or functional limit state might control the design. For low-rise buildings, safety or impending collapse limit state will control the design).
- designing simultaneously for the demands of two or more limit states. The selection of the method depends on the type of building and its occupancy function. For example, for Standard Occupancy and low-rise buildings (one- to three-story buildings with regular configurations and structural system and short bay-span and height), their preliminary design may be carried out for just the safety limit state (life safety under major EQGMs).

Note that while in the case of the comprehensive EQ-RD approach the preliminary sizing is done for the minimum acceptable stiffness, in this force/strength approach it is done for the minimum strength. This is the main difference between the two approaches.

Load Combinations: The critical combinations are the same as those considered for the comprehensive EQ-RD approach, which is discussed in Section B.5. The only new problem that may arise is when it is desirable to use a very simplified design method for preliminary design based on just the life-safety performance design objective and to use allowable stress design. In this case, it is suggested that one of the two following procedures be used to estimate E.

- Use the conservative approach of using the V_{saf} directly with the allowable stress combinations and the stresses compared directly with the conventional allowable stresses.
- Use the $V_{saf}(R_{saf} / R_{ser}) = V_{elastic}/R_{service}$.
  This last value for the base shear design is equivalent to what the UBC specifies, i.e., $V_{elastic}/R_w$.

Analysis of the Performance of the preliminary EQ-RD; Acceptability Checks: As discussed in Section E.3, it is recommended that to conduct the acceptability checks for the limit states involving damage at least the static nonlinear pushover method of analysis be used.

All the other steps needed for achieving the final design are the same as those for the comprehensive EQ-RD approach and are discussed in detail in Appendix B.

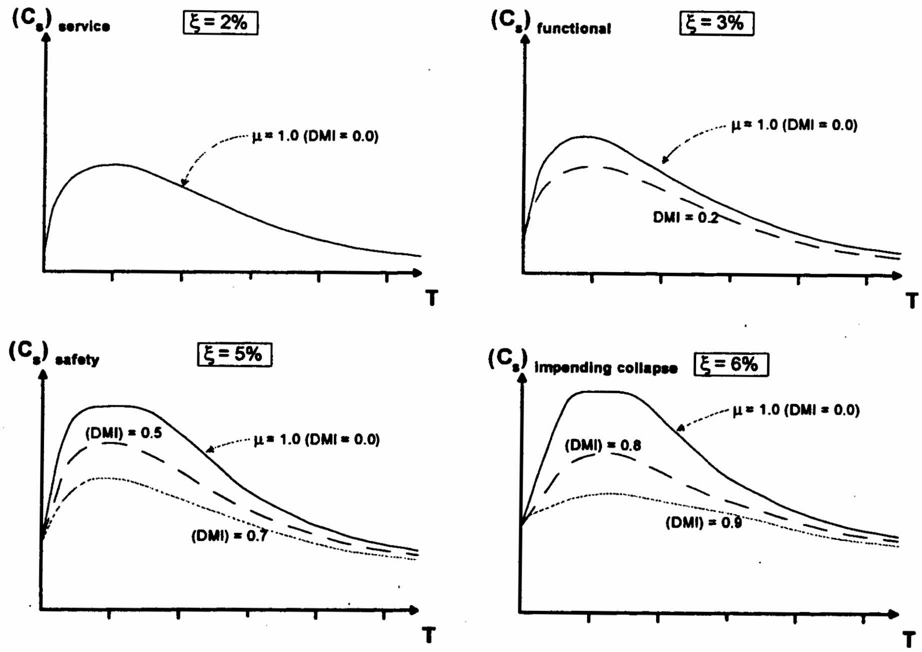


Figure E-1(a): Strength, C_s, Smoothed Design Spectra

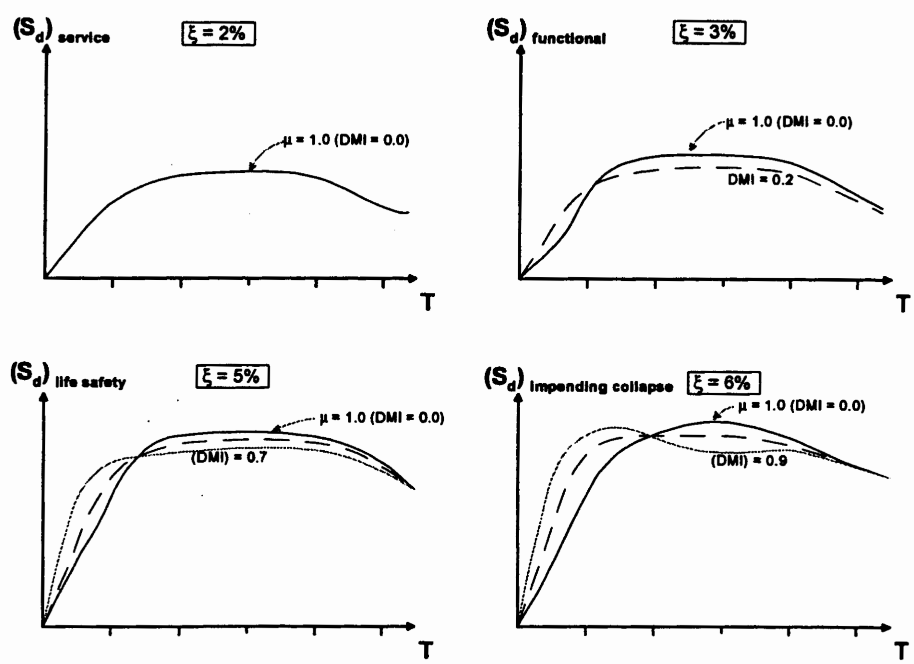


Figure E-1(b): Displacement, S_d, Smoothed Design Spectra

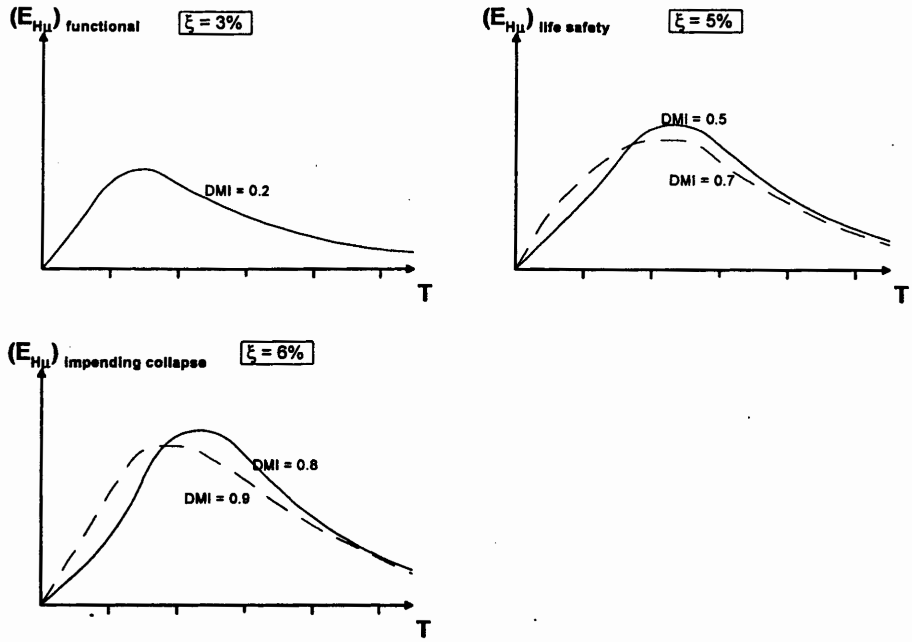


Figure E-1(c): Energy Dissipation, $E_{H\mu}$, Smoothed Design Spectra

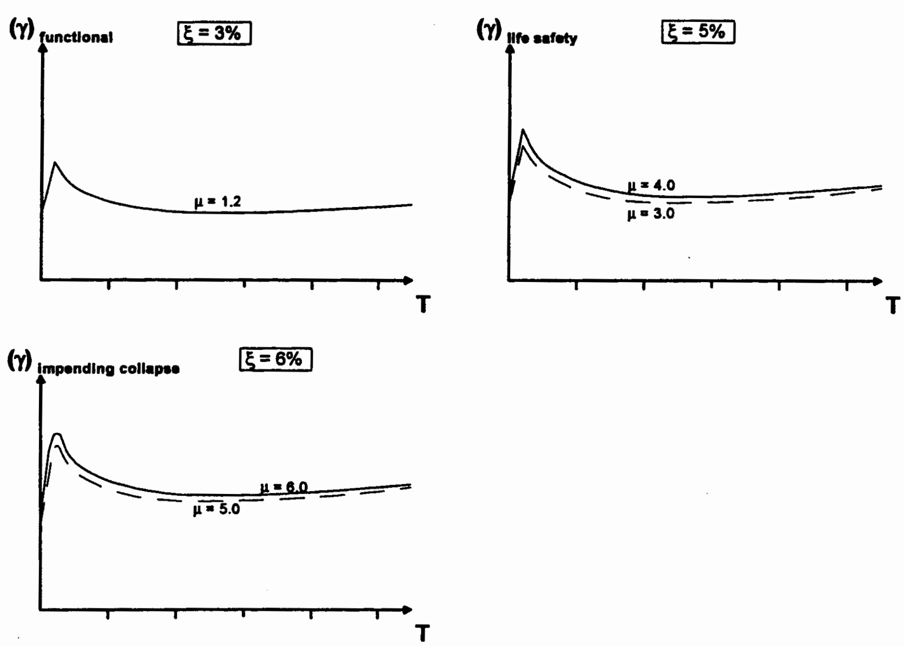


Figure E-1(d): Factor, γ, Smoothed Design Spectra

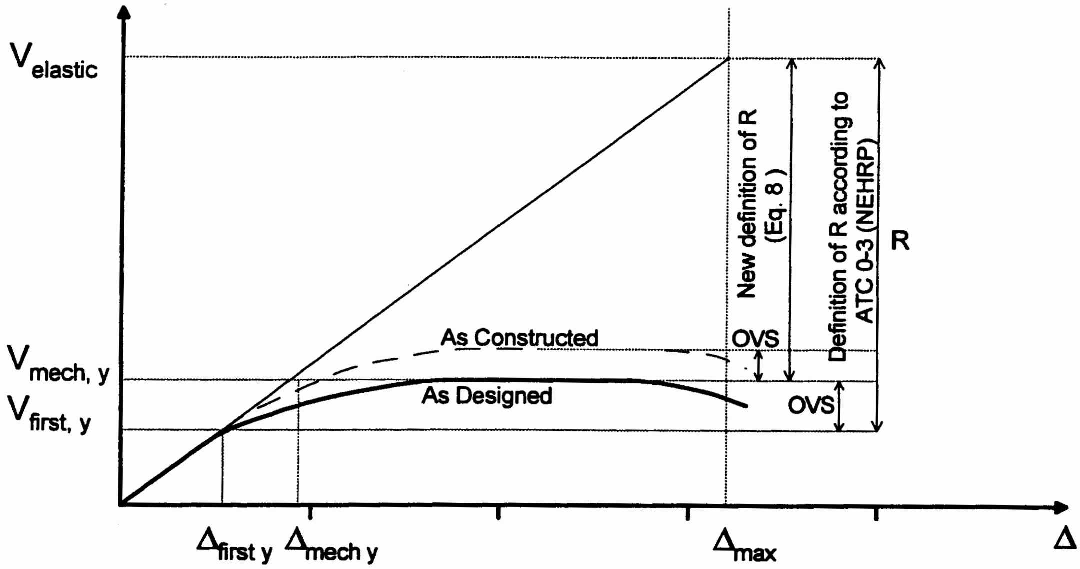


Figure E-2: Illustration of the Definition of the R Factor

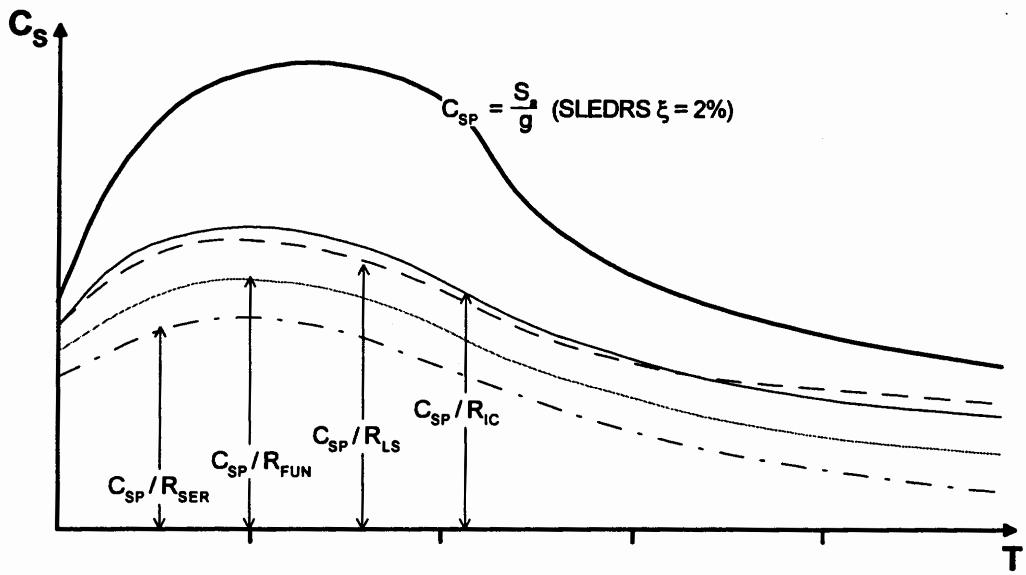


Figure E-3: Strength-Smoothed Design Spectra

APPENDIX F

SIMPLIFIED FORCE/STRENGTH DESIGN APPROACH

by

Roland Sharpe and Vitelmo V. Bertero

F.1 Overview of Approach

A. *Introductory Remarks*

Inspection of the behavior of engineered buildings during previous EQs around the world shows that there are certain types of buildings, usually low-rise buildings, that can have excellent performance under different levels of EQGMs, even if they have been designed using a very simple force/strength numerical design approach based on just the static equivalent lateral force (ELF) procedure. This excellent performance has been observed when the very simple numerical design approach has been backed up by strict enforcement of very restrictive code regulations regarding the entire building system, i.e.,

- the configuration of the building, the structural layout, the structural system, the structural material and detailing of the structural members and their connections and supports;
- the nonstructural components, their materials and detailing of their connections with the structural members, the equipment and the contents of the building;
- the foundation (type, material and detailing); and
- the site conditions (soil profile and topography).

The experience of the city of San Juan, Argentina during the 1977 Caucete earthquake demonstrates that simplified code provisions, when strictly enforced through intelligent and honest review of the design, control of the quality of materials and field inspection of the workmanship, can result in earthquake-resistant construction with excellent performance. This city, which was devastated by an earthquake in 1944, was rebuilt with the strict enforcement of a very simple code requiring a numerical design based on the use of an ELF static procedure, and did not suffer any significant damage during the similar EQGMs that occurred during the 1977 EQ.

Thus, what is proposed herein is the development of a simplified numerical design for engineered buildings which, backed up by associated strict code provisions regarding the applicability of such an approach (i.e., regarding the site conditions, the building configuration, the superstructure, the nonstructural components and their material and detailing, the foundation and the load combinations to be considered) can result in construction with seismic performances that comply with all the performance levels and performance design objectives envisioned in the general philosophy that has been formulated and on which the comprehensive performance-based EQ-RD approach has been developed.

B. *Proposed Simplified Force/Strength Approach*

Considering that at present the simplest design procedure for the design of engineered buildings is based on the use of the equivalent static lateral force procedure that considers just one performance level (the life safety level), it is proposed to keep this numerical design procedure but to present a detailed discussion of how to improve the regulations and their enforcement in the field regarding when and how such a procedure can be applied and result in construction that complies with the general philosophy of the performance-based EQ-RD and EQ-RC. The application of this simplified design approach to each of the building types in the adopted list of common types of buildings and the needed restriction in their configurations and siting will be investigated and developed (formulated). Briefly, this approach can be considered as the simplest performance-based EQ-RD of an engineered building whose numerical design is carried out by a procedure similar to the simplest numerical procedure specified in current seismic codes, but backed up with extensive and stringent prescriptive design, construction and maintenance rules. To achieve the above such simplified numerical EQ-RD procedure, it was necessary to simplify the general performance-based EQ-RD approach discussed in Appendix E, introducing the following main simplifications.

1. Performance Levels on Which the Numerical Preliminary and Final Designs Are Based. Although this proposed simplified approach considers the four performance design objectives of the comprehensive EQ-RD approach, as discussed in Chapter 2 and indicated in the flow chart of Fig. B-1, *the numerical preliminary and final designs are conducted for just the life safety design objective using just linear elastic analysis methods*. Again, this can only be done for buildings located in regions where according to the seismic activity the EQGMs associated with the life safety limit state will control the design and where the type of building (low-rise building with regular configuration, structural layout and structural system) and subjected to what can be considered normal load combinations which will allow use of just linear elastic methods of analysis to have a good idea of its actual nonlinear behavior under the expected severe and extreme EQGMs. Based on these restrictive conditions, attempts have been made to develop simple formulas using forces that can be used to check the expected deformations of the designed building. This is done in the Commentary.

2. Acceptability Checks of the Preliminary and Final Design. The needed acceptability checks are conducted using just the static linear elastic pushover method of analysis, computing the stress ratios at the potential critical regions of the structure as well as the deformations of the members (vertical and lateral) and stories *that occur under the lateral forces associated with the base shear corresponding to each of the four levels of EQGMs* associated with the minimum four performance levels that need to be considered in performance-based EQ-RD. It is recommended that at least two different lateral load patterns be used for the pushover analysis; the inverted triangle or the one using modal analysis; and the rectangular or uniform lateral load pattern. In the Commentary, simple formulas based on computed linear elastic forces are offered to check the deformation capabilities of the designed structural members.

3. The Numerical Preliminary Design Procedure. This is based on the use of the ELF procedure of the force/strength approach of current seismic code procedures (1994 UBC and 1994 NEHRP). It is recommended that the design be conducted at the level of the first significant yielding rather than the allowable stress level as specified in the 1994 UBC. Thus, conceptually the system quality or structural system factor, R_w, of the 1994 UBC is replaced by the R as recommended in the 1994 NEHRP Recommendations. It should be clearly noted that although the basis for the numerical preliminary design are the same as those of the 1994 NEHRP and UBC ELF method the specific values of the R that are (or will be) recommended are quite different. These recommended values will be obtained following the procedure indicated in the general force/strength approach discussed in detail in Appendix E. A more detailed discussion of the numerical preliminary design procedure for this proposed simplified force/strength approach is offered in the Commentary where some extra conservative suggestions are also offered to simplify even more this numerical procedure.

C. *Applicability of this Simplified Force/Strength Approach*

This proposed simplified approach may be applied for the EQ-RD of new buildings as well as for the seismic repair and upgrading of existing buildings of the following types: 1, 3, 4, 6, 7, 8, 9, 10, 13, 14, and 15 provided that these buildings are in compliance with the limitations imposed in the associated regulations covering the acceptable: height, overall configuration, foundation, site conditions and load conditions controlling their design. Configuration requirements for having regular buildings similar to but even more restrictive and stringent than those in present 1994 UBC will be formulated.

F.2 Advantages of the Simplified Force/Strength Approach

Because the methods of analysis and numerical procedures used in this approach are practically the same as those presently used by most EQ-resistant designers, it will be fairly easy to understand and apply by the average EQ-resistant designer. Furthermore, as this approach will be based on the use of:

- an improved list of guidelines for the conceptual design;
- more realistic and reliable smoothed strength and displacement spectra for the numerical design;
- more realistic and reliable load combinations at each of the performance levels (limit states);
- more comprehensive acceptability checks considering at least the four minimum levels of EQGMs associated with the four desired performance levels and using an improved list of acceptable values for the engineering

parameters that control the behavior (degree of damage) under each level of EQGM;

- a very detailed list of practical requirements regarding sizing and particularly detailing of the structural members, their connections and support, as well as of the architectural components and their connections with the nonstructural members;
- very stringent requirements regarding quality control of materials, workmanship and maintenance of the building.

It is expected that this will lead to the design and construction of certain types of engineered building with more predictable performance than is obtained from applying present seismic code approaches.

When compared with the Comprehensive and General Force/Strength Approach, besides the considerable simplification in the numerical design procedure and in the analyses of of acceptability checks, it eliminates the need for peer review and/or independent review.

F.3 Disadvantages of the Simplified Force/Strength Approach

The main disadvantages of this approach are: first, its application is limited to very specific types of buildings; and secondly that without intelligent and honest review (control) of the design and then field inspection, this approach may be misused by applying it to building types that do not comply with the rest of the restrictions involved in its application.

F.4 Research, Development and Implementation Needs

Because this approach is nothing but a very simplified version of the general force/strength approach discussed in Appendix E, the research, development and implementation needs for improving the reliability of the practical applications of this simplified approach are practically contained in the similar needs discussed in Appendix E. Because the simplifications introduced are based on carrying out just a preliminary objective, the Life Safety, and then conducting all the acceptability checks under the forces demanded by the effects of the EQGMs associated with each of the general four performance levels, using just a linear elastic static pushover analysis, it is of great importance to emphasize the need for the following studies.

- Identification of the types of buildings (height, overall configuration, structural layout, structural system, foundation and nonstructural components) site location (seismic region) site conditions, and load combinations for which such simplified approach can be applied. This can be done by calibrating their simplified designs against designs conducted using the comprehensive EQ-RD approach.

- Simple but still realistic and reliable strength smoothed design spectra, which will allow estimation of the base shear required by each of the four performance design objectives.
- Simple but still realistic and reliable lateral displacement smoothed design response spectra that can be used together with the static linear elastic pushover method of analysis to conduct the necessary acceptability checks.
- Investigation of the most realistic and reliable load pattern that can be used with the linear elastic pushover method of analysis or if it is necessary to use at least two different lateral load patterns.
- Investigation and development of a simple method to include the effects of the duration of strong EQGMs on the inelastic behavior of the types of buildings that can be designed according to this simplified approach, i.e., how the demanded E_H can be included in the specified values of R.
- Investigation of how the effects of the extected inelastic behavior (particularly the effects of repeated cycles of reversal yieliding deformations at the performance levels involving significant acceptable damage ((like the life safety and impending collapse) can affect the distribution of the required forces and moments along the span of the member and lead to demands in strength as well as in deformations more severe than those obtained based on just conducting the required linear elastic pushover analysis.
- Development of prescriptive rules for the design of the structural members (such as requiring the use of the philosophy of capacity design for design against shear and axial forces) and particularly minimum requirements for the sizing and detailing of the member along its entire span, to avoid the possibility that, due to the effect of repeated cycles of reversal deformations, inelastic deformations might develop in regions that have not been identified through the required static linear elastic pushover analysis, and therefore are not been properly detailed to to survive such uexpected inelastic deformations.

COMMENTARY

c.1 Introductory Remarks

As pointed out in the above Appendix F, this proposed approach for performance based EQ-RD, although developed following the basic concepts involved in the General Force/Strength Approach discussed in Appendix E and thus also following the comprehensive iterative procedure illustrated in the flow chart of Fig. B-1, involves a proposed numerical EQ-RD methodology that is the simplest one that can be used in the design of an engineered building and still result in a designed building of more predictable performance than can be obtained using the force/strength approach of current seismic codes. The numerical EQ-RD design methodology, which is discussed in detail below, is basically the same static ELF methodology that is used in the current seismic codes (1994 UBC and 1994 NEHRP Recommendations), and therefore it is hoped that it will be fairly easy to understand, follow and apply by the average professional earthquake-resistant designer. However, as is always the case when the procedure involved in a comprehensive approach is simplified without sacrificing the reliability of the resulting design, it is necessary to introduce limitations and/or restrictions in the applicability of such simplified approach. Thus, the application of such simplified numerical EQ-RD methodology is limited to certain types of engineered buildings.

The restrictions in the applicability of this simplified approach are considerably more stringent than those specified in the application of the static ELF procedure in current seismic codes. Thus, as was pointed out in the Research Needs presented in Section F.4, it will be necessary to conduct extensive calibration studies to develop reliable recommendations regarding the types of engineered buildings to which the simplified numerical EQ-RD methodology discussed below can be applied and still result in designed buildings which will perform under each of the EQGM levels involved in the four performance design objectives, in compliance with the corresponding desired performance levels.

Because to ensure a good performance during EQGMS it is not only necessary to have a sound EQ-RD but also a good Earthquake-Resistant Construction (EQ-RC), there is a need for more detailed and stringent requirements regarding quality assurance during construction as well as for monitoring the maintenance and function (occupancy) of the building during its service life.

c.2 EQ-RD Methodology and Procedure

Basically, the methodology and procedure proposed for this simplified force/stress approach are the same as those summarized in the flow chart of Fig. B-1. The first three steps of this iterative design procedure are as follows:

- *Check Suitability of Site*
- *Define Performance Levels*
- *Define Design Objectives*

which cannot be conducted independently of each other because they are interrelated (to check suitability of sites it is necessary to evaluate the seismic hazards that can control the performance at each of the desired performance levels), are conducted in a manner similar to that described for the General Force/Strength Approach or for the Comprehensive Approach.

It has to be noted that as according to this simplified approach the EQ-RD is conducted for just one performance design objective, the Life Safety, the application of this method should be strictly speaking restricted to buildings whose sites are in seismic regions where the EQGMs corresponding to this design objective control the overall design. The EQ-RD of the entire structural system, as in the case of the General Force/Strength and Comprehensive EQ-RD Approaches, consists in the iterative procedure indicated in the flow chart of Fig. B-1. Thus the following designs must be conducted:

- *a conceptual overall design*
- *a numerical preliminary design*
- *a final design and detailing*

These designs must be conducted together with the corresponding acceptability checks for each of these three types of design.

It should be noted that because of the more detailed and stringent constraints on the applicability of this simplified approach with respect to the General Force/Strength Approach and the Comprehensive Approach, the importance and the difficulties of attaining a sound (proper) overall conceptual design can be significantly higher than those involved when the more general or comprehensive approach is used.

These extra difficulties also emphasize the need for developing specific and detailed guidelines regarding the restrictions on height, number of stories, configuration, structural layout, structural system, structural material, foundation and the nonstructural components and their materials. Once a conceptual design *that is acceptable for applying this proposed simplified force/strength approach is attained,* then the numerical EQ-RD procedure can be started with a preliminary design.

c.3 Numerical Preliminary Design

As discussed in Section F.1, the preliminary EQ-RD is conducted for just one performance design objective, the Life Safety. Thus, the processing of the EQGM data is somewhat simplified with respect to that required for the case of the Comprehensive EQ-RD Approach and even with respect to that required for the General Force/Strength Approach.

The numerical EQ-RD that is proposed is based on the use of the static ELF procedure of the force/strength approach of the 1994 UBC, except that the design is conducted at the level of the first significant yielding using the strength method rather than at the allowable stress level. Thus, the factor R_w is replaced by the reduction factor R, as recommended in the 1994 NEHRP Recommendations.

A. *Static Equivalent Lateral Force (ELF) Procedure*

Seismic Base Shear, V: The V in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \qquad (1)$$

where:

W = weight of the reactive mass and which is considered equal to the mass corresponding to total dead load and applicable portions of other loads.

C_s= the design seismic response coefficient for the major EQGMs associated with life safety.

Although the values of C_s can be determined by different procedures, the following three are suggested.

1. Using the following formula

$$C_s = C_{sp}/R \tag{2}$$

where:

C_{sp}= the seismic response coefficient for a linear elastic system, and which may be obtained from a specified pseudo-acceleration (smoothed linear elastic design response spectra) for the selected site condition.

R = the response modification factor which can be determined from a smoothed spectra or using approximate formulas, or even tables, as has been discussed in detail in References 1 and 2.

2. Directly from strength-smoothed inelastic design response spectra (SIDRS) in which the C_s is given as a function of acceptable value of the Damage Function (DMF) or Damage Index (DMI) according to the structural system and structural material that is used (see Fig. F-1).

3. Using a conservative constant value for C_s (or for C_{sp} and R), i.e., a value independent of the period, T, of the structure, as is illustrated in Fig. F-2. Note that because it is expected that most of the buildings that can be designed using this Simplified Force/Strength Approach will be low-rise buildings, which will usually have fundamental periods varying from 0.2 secs up to a maximum of 1 sec, and buildings located on soil types that in current codes are classified as S_1, S_2 and S_3, this suggested approximate procedure, although conservative, would not lead to preliminary designs that would differ significantly from those obtained using Fig. F.1 or Procedure 1.

   The advantage of using a constant value of C_s is that it eliminates the need for estimating the T for conducting the preliminary design, as is shown below.

It should be noted that in the current seismic codes that value of C_s is determined according to the following two procedures, depending on which seismic regulations are used.

i) <u>According to the 1994 NEHRP:</u>

$$C_s = 1.2C_v/RT^{2/3} \tag{3}$$

where:

C_v= the seismic coefficient from a table similar to Table 1.4.2.4b of 1994 NEHRP

R = the response modification factor from a table similar to Table 2.1.2 of 1994 NEHRP

T = the fundamental period of the building determined as discussed below.

ii) According to the 1994 UBC:

$$C_s = \frac{C_{sp}}{R} = \frac{ZI(1.25S)/T^{2/3}}{R} \quad (4)$$

where the value of $(1.25S)/T^{2/3}$ need not exceed 2.75 and where:

Z = seismic zone factor (see 1994 UBC Table 16-I)

I = importance factor (see 1994 UBC Table 16-K)

S = site coefficient for soil characteristics (see 1994 Table 16-J)

T = fundamental period of vibration, in seconds, of the building in one direction under consideration, determined from the following equation for concrete and steel moment-resisting frame buildings not exceeding 12 stories in height and having a minimum story height of 10 ft. (3 m).

$$T = 0.1N \quad (5)$$

where:

N = number of stories.

Or, in general, from the following empirical equation:

$$T = C_t(h_n)^{3/4} \quad (6)$$

where:

h_n = height in ft. (m)

C_t = numerical coefficient that depends on the type of structural system and the structural material (see 1994 UBC or 1994 NEHRP)

T can also be determined by using an approximate method based on assumed distribution of lateral forces and deformations, such as Rayleigh's method (see Appendix C, Section C.2), but in this case the computed value should not exceed the values of T determined from Eq. 5 or 6, multiplied by a certain coefficient for upper limit, as recommended by 1994 NEHRP (see NEHRP Table 2.2.3).

Vertical Distribution of Seismic Forces: The lateral force, F_x (kips or kN) induced at any level in this proposed simplified approach may be computed using the same equations that are recommended in the 1994 NEHRP Recommendations, i.e.,

$$F_x = C_{vx}V \quad (7)$$

and

$$C_{vx} = \frac{W_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{8}$$

where:

C_{vx} = vertical distribution factor

V = total design lateral force or shear at the base of the building (kip or kN)

w_i and w_x = the portion of the total gravity load of the building, W, assigned to level i or x

k = an exponent related to the building period as follows: for buildings having a period of 0.5 seconds or less, k = 1; for buildings having a period of 2.5 seconds or more, k = 2; and for buildings having a period between 0.5 and 2.5 seconds, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

As most of the buildings that can be designed using this procedure may have T 1.0 secs, the k will vary from 1.0 to 1.33.

Horizontal Shear Distribution: The seismic design story shear in any story, V_x (kip or kN), shall be determined from the following equation:

$$V_x = \sum_{i=1}^{x} F_i \tag{9}$$

where:

F_i = the portion of the seismic base shear, V (kip or kN), induced at level i.

The seismic design story shear, V_x (kip or kN) shall be distributed to the various vertical elements of the seismic force resisting system in the story under consideration based on the relative lateral stiffnesses of the vertical resisting elements and the diaphragm, considering the rigidity.

Where diaphragms are not flexible, the mass at each level shall be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5% of the building dimension at that level perpendicular to the direction of the force under consideration. The effect of this displacement on the story shear distribution shall be considered.

Diaphragms shall be considered flexible for the purposes of distribution of story shear and torsional moment when the maximum lateral deformation of the diaphragm is more than two times the average story drift of the associated story. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm itself under lateral load with the story drift of adjoining vertical resisting elements under equivalent tributary lateral load.

It is suggested to prohibit use of flexible diaphragm by proper limitations in the configuration of the building and floor and roof systems that can be used in the buildings in which this simplified approach can be applied.

Horizontal Torsional Moments: Provisions shall be made for the increased shears that can result from horizontal torsion where diaphragms are not flexible. The most severe load combination for each element shall be considered for design.

The torsional design moment at a given story shall be the moment resulting from eccentricities between applied design lateral forces at levels above that story and the vertical resisting elements in that story plus and accidental torsion.

The accidental torsional moment shall be determined by assuming the mass is displaced as required above. It is suggested that to avoid significant torsional effects during the inelastic response of the structure, the minimum number of lines with lateral force-resisting components in each direction should be four.

Where torsional irregularity exists, the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor, A_x, determined from the following formula:

$$A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}} \right] \tag{10}$$

where:

δ_{avg} = the average of the displacements at the extreme points of the structure at level x (in or mm)

δ_{max} = the maximum displacement at level x (in or mm).

The value of A_x need not exceed 3.0.

It is suggested that torsional irregularity be prohibited on buildings on which this simplified approach may be applied.

Overturning: Every structure shall be designed to resist the overturning effects caused by earthquake forces specified above. At any level, the overturning moments to be resisted shall be determined using those seismic forces (F_x) that act on levels above the level under consideration. At any level, the incremental changes of the design overturning moment shall be distributed to the various resisting elements in the manner prescribed above. Overturning effects on every element shall be carried down to the foundation.

Preliminary Design for Strength: The preliminary design of the individual members of the structure shall be carried out at least against the minimum acceptable strength required by major EQGMs (which, as indicated, is that corresponding to the life safety performance level, with repairable damage).

Depending on the function of the building and/or the desire of the owner, the design against the major EQGMs can be conducted for either a functional or a service performance level.

Method of Analysis to Determine the Optimal Internal Force Demands: The internal forces in the members of the buildings should be determined using a linear elastic model assuming a fixed base. An approved alternative method may be used to establish the seismic forces and their distribution and the corresponding internal forces and deformations in the members shall be determined using a model consistent with the methods adopted.

Load Combinations: It is suggested that the 1994 NEHRP Recommendations, which have been discussed in detail in Appendix B, be used.

Sizing of Members: The individual members and their connections and supports shall be sized using a strength method and the capacity design philosophy.

Analysis of the Performance of the Preliminary Design: To determine if the preliminary design resulting from application of the above simplified preliminary strength design method considering just the requirements demanded by major EQGMs (when combined with the other loads that can act simultaneously) satisfies all the desired performance design objectives, one of the following different methods can be used, depending on the information available regarding the EQGMs (i.e., how they have been defined) at the different levels of their damage potential. To use some of these methods, it is necessary to compute periods and mode vibrations of the designed structure.

- If the time histories of the probable EQGMs that can occur at the site with the recurrence or return periods that correspond to the service, functional, life safety and impending collapse limit states are available, then 3D linear elastic dynamic analysis of the preliminary design can be conducted for each of these four levels of EQGMs, and the stress ratios at the critical section of the members at each floor level, as well as the deformation, can be determined.

- If the data available for the EQGMs consist of just the probable strength SLEDRS for each of the levels of EQGMs that need to be considered, then 3D linear elastic modal response spectra analysis can be conducted for each of the established performance levels to determine the stress ratios at the critical section of the members as well as the deformations at each floor level.

- If the only data available for the EQGMs are the required V_s and the corresponding lateral forces for each of the EQGM levels, then a 3D or 2D linear elastic static pushover analysis should be conducted to determine the stress ratio and floor displacements at each of the levels of the V_s.

Acceptability Checks: Acceptance Criteria: The acceptability checks under the EQGMs (in combination with other loads that can act simultaneously) corresponding to *the service performance level* are straightforward because the results obtained in any of the above methods of analysis can be compared directly with what are considered to be the acceptable values for the stress and deformations at this performance level. On the other hand, for the other three performance design objectives, particularly for *the life safety and impending collapse performance levels*, considerable judgement should be employed in interpreting the results obtained in the above linear elastic analyses and when these results are compared with the established values for the parameters controlling the performance (damage) at each of these performance levels.

In order to emphasize the importance of avoiding blind application of the demands resulting from the static linear elastic pushover analysis or even those obtained from the dynamic linear elastic modal spectra response analysis, some of the unexpected and undesirable inelastic behavior (performance) that can occur is illustrated with the results presented in Fig. F-3 for a simple one-story, one-bay special moment-resistant frame.

c.4 References

1. Miranda, E. and Bertero, V. V. (1994), "Evaluation of Strength Reduction Factors for Earthquake Resistant Design," Earthquake Spectra, Vol. 10, No. 2, pp 357-380.

2. Miranda, E. (1991), "Seismic Evaluation and Upgrading of Existing Buildings," PhD. Thesis, University of California at Berkeley.

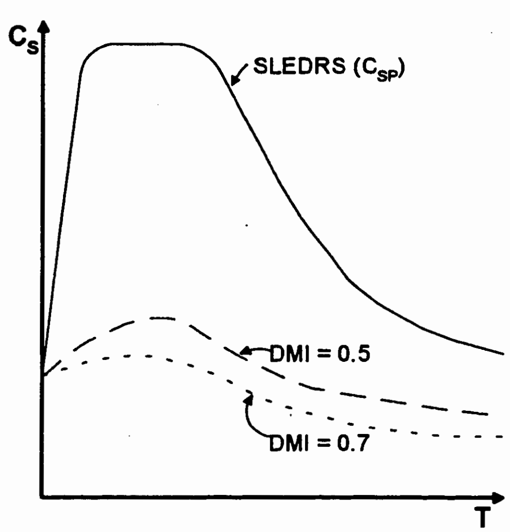


Figure F-1: Strength (C_s)-Smoothed Inelastic Design Spectra (SIDRS)

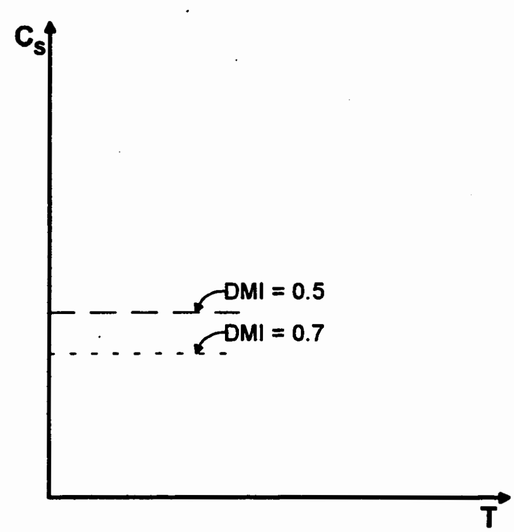
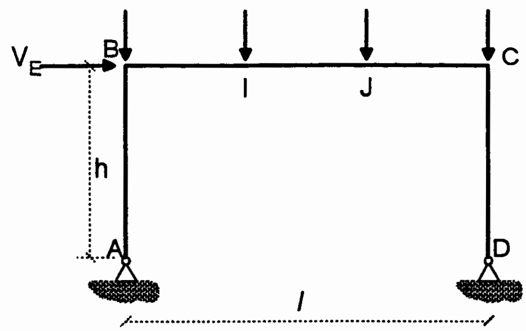


Figure F-2: Simplified (C_s)-Smoothed Inelastic Design Response Spectra (SIDRS)

Fig. F-3: **Example showing Effects of Gravity Loads on the Inelastic Behavior of Structures That Have Been Designed Using Linear Elastic ELF Approach When Subjected to Severe EQGMs**



<u>GIVEN</u>: **Gravity Loads: D, L & S**
Earthquake Load: E ⇒ V_E

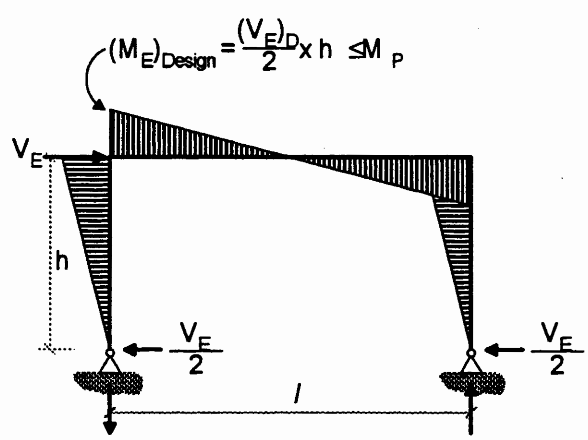
<u>REQUIRED</u>: **To Design for Life Safety and Estimate Behavior (Performance) Under Severe and Extreme EQGMs**

<u>SOLUTIONS</u>: **Load Combinations:**

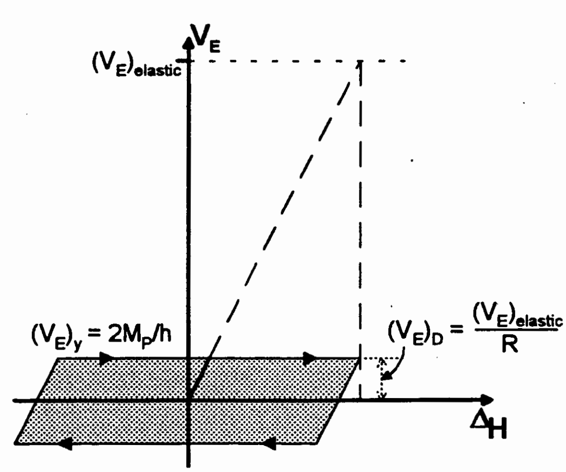
1.4D
1.2D + 1.6L + 0.55
1.2D + 0.5L + 0.25 + 1.0E
0.9D - 1.0E

Consider that the effects of gravity loads on the bending moments can vary from being negligible to being of the same order as the effects of the design EQ loads.

<u>Case 1</u>: **The bending moments due to gravity loads, M_G, are zero.**



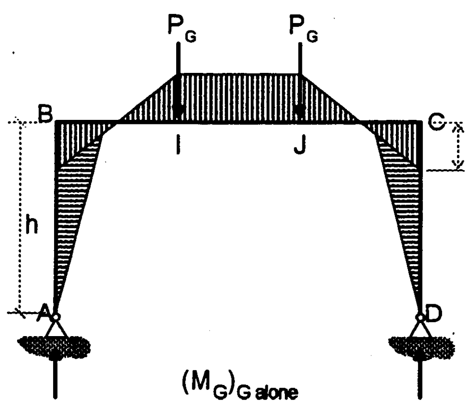
<u>DISCUSSION</u>. From the predicted performance, it is clear that when the only important loading condition is the EQ, the linear elastic ELF Approach, independently of the design method (linear elastic strength or limit design), <u>*will lead to a predictable performance*</u>. Furthermore, the acceptability checks of the deformation can be formulated and therefore conducted on the basis of the corresponding moments, i.e. forces rather than strains, deflections and/or rotations.



Case 2: The bending moments due to gravity loads are important.

Assume the M at sections B and C of the beam under gravity forces acting alone, $(M_G)_{G\ alone}$ is $(M_G)_{G\ alone} = (M_E)_D$. Furthermore, assume that the critical M_G under gravity forces that are acting simultaneously with the design EQ forces, $(M_G)_{G+E}$, is

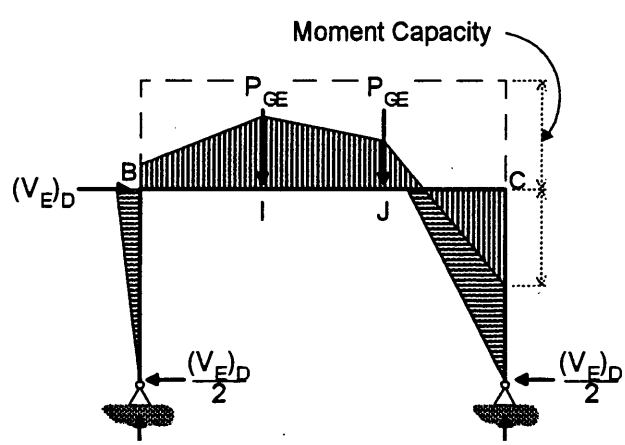
$$(M_G)_{G\ alone} = (M_E)_D = \frac{(V_E)_D h}{2}$$



$$(M_G)_{G+E} = \tfrac{2}{3}(M_G)_{G\ alone} = \tfrac{2}{3}(M_E)_D$$

Then the design of the beam will be controlled by the G and E acting simultaneously. The required M_p will depend on the design method.

$$(M_D) = \frac{5}{3}(M_E)_D \le M_P$$



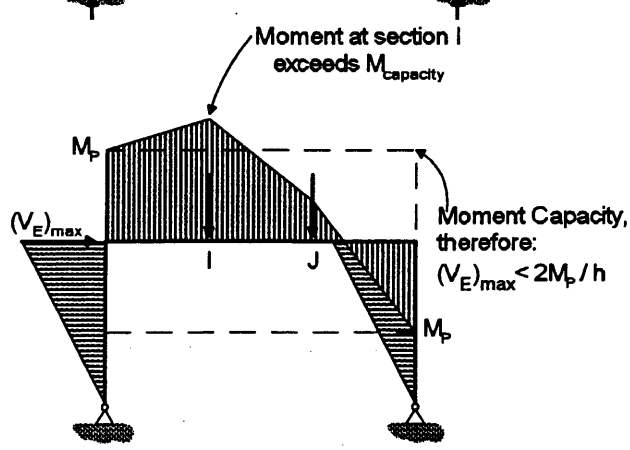
A. Linear Elastic Strength Method

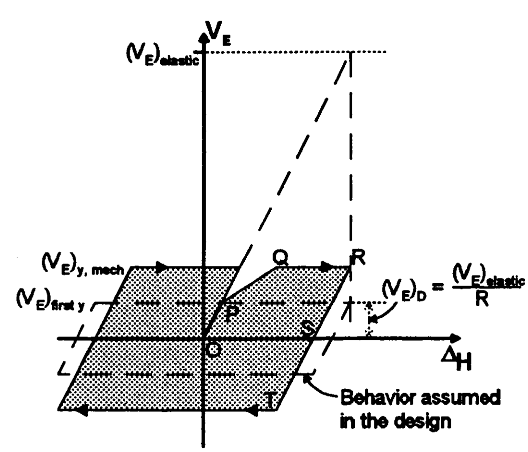
$$(M_p)_{req'd} = (M_G)_{G+E} + (M_E)_D$$

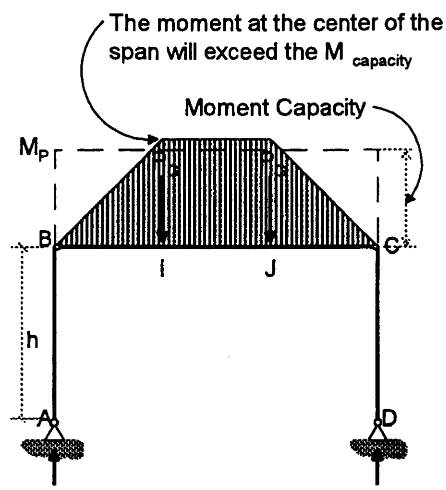
$$(M_p)_{req'd} = (\tfrac{2}{3} + 1)(M_E)_D$$

$$(M_p)_{req'd} = \tfrac{5}{3}(M_E)_D$$

Assuming that beam is of constant strength along its span, the critical sections, according to linear elastic behavior, are at the ends of the beam.

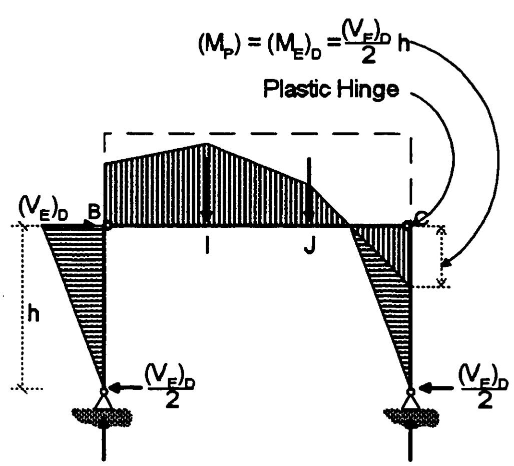


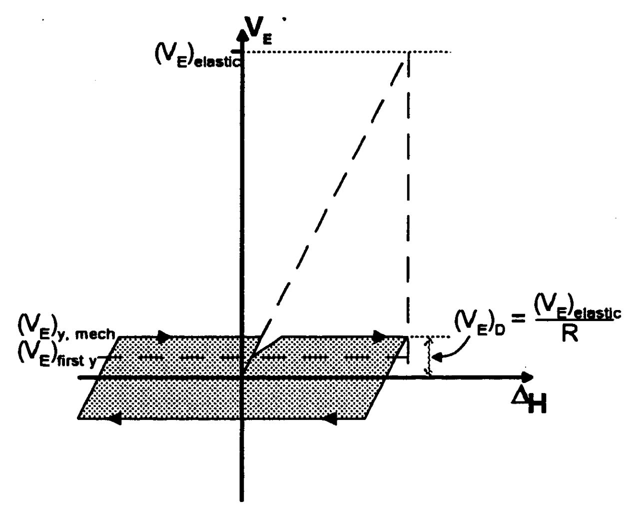




Moment diagram after first EQ unloading, i.e., at S of V vs. Δ_H curve when loaded with G alone.

B. Limit Design Method. If the beam is designed to have a constant strength along its entire span, its limit design will be controlled by the formation of plastic hinges at C (or B) and I (or J) depending on the direction of V_E. For the sake of simplicity, it is assumed that the beam will be reinforced at regions I and J to avoid formation of plastic hinges at these sections.

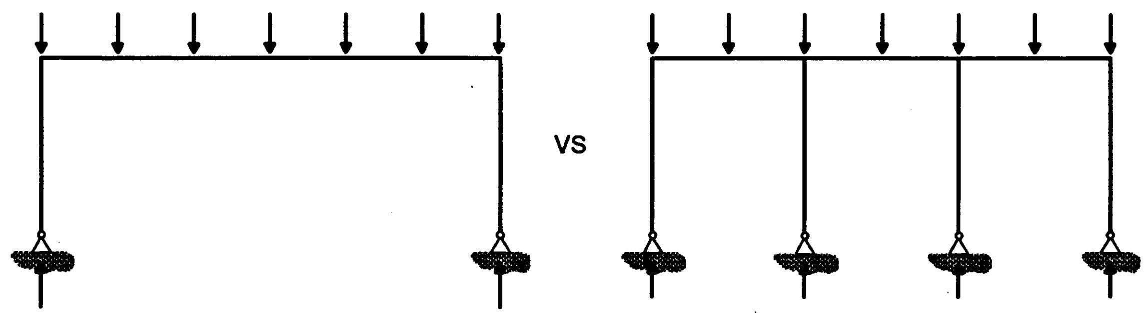




DISCUSSION. From the predicted performances it is clear that, when the effects of gravity loads are significant, the linear elastic ELF approach can result in a designed structure which under severe EQGMs can develop unexpected and undesirable inelastic deformations. Furthermore, the magnitude of these deformations depends on the design method and cannot be predicted in a reliable manner using equations based on moments (forces) that are determined through linear elastic analyses. In the case that the gravity loads are distributed rather than concentrated, moving plastic hinges can be developed under severe EQGMs. As shown and discussed in Appendix A, as the effects of gravity load increase, the use of linear relationships to check the performance of the entire system, critical stories and particularly of the components (elements) becomes less and less reliable, and therefore their use in EQ-RD cannot be justified.

Considering that the linear elastic ELF approach is only applicable to low-rise buildings, and that for this type of building the gravity loads are important in their design, if it is desired to use the suggested linear elastic relationship to check strength deformations and ductility ratio, it is of utmost importance to reduce the effects of these gravity loads as much as possible when compared to the effects of the EQ forces.

To reduce the effects of the gravity loads on the members, the span of the beams (or the span of the bays) should be reduced as much as functionally and economically feasible. This can be done by using more spans (columns) as is illustrated in the figure below. Therefore, there is a need to carry out studies investigating what is the maximum span that can be assigned to the bays of the structural system if it is desired to apply the equations that have been suggested for checking the design of the building structure when the elastic ELF approach is applied.



Using three rather than just one bay not only reduces the effects of the gravity loads but also increases the redundancy of the lateral system.

APPENDIX G

STATIC NONLINEAR PUSHOVER ANALYSIS

by

Joe Nicoletti

1. Overview of Procedure

Push over analyses are useful techniques to identify the nonlinear response of a structure as the various structural elements yield sequentially under load. A popular form of push over analysis uses iterative static loading in an inverted triangular distribution and monitors base shear versus roof displacement. A significant limitation of that technique is that the loading is based on a straight line fundamental mode shape that may be reasonably valid for the initial loading, but may not be representative of subsequent loading distribution if yielding occurs in the lower portions of the structure. Other limitations include the fact that it does not consider the contribution of higher modes or three-dimensional effects, such as torsion. In spite of these limitations the simplified approach is useful as an acceptance check for buildings of limited height and regular configuration that may be designed by the equivalent lateral force method.

The proposed technique is intended to minimize the above limitations and provide an acceptance technique with wider application. It is anticipated that, for many buildings, some form of three-dimensional linear elastic spectral response analysis will be performed for preliminary and possibly for final design. The proposed procedure would utilize the same mathematical structural model and design spectra used in the design process. This procedure consists of a nonlinear spectral response analysis by iteration and superposition. The acceptance criterion is based on the premise that the nonlinear (elastic/inelastic) displacements of the structure subjected to the spectral demands are equivalent to the displacements that would have occurred if the response of the structure had remained totally linear elastic. The procedure is similar to a push over analysis in that the structure is subjected iteratively to a set of forces and modified for each iteration to represent yielding or failed structural elements. Unlike a static push over analysis, this procedure monitors modal responses to any specific response spectrum. For many small regular buildings the two-dimensional spectral response, using only the fundamental mode, can be performed with hand calculations. The procedure is more easily implemented by computer analysis that permits consideration of the modified modal responses, three-dimensional response, P-Δ, and the elastic and inelastic torsional effects.

2. General Procedure

The procedure is illustrated by the following example below. As indicated in Figures G-1 through G-6, the elastic structure is subjected to the evaluation response spectrum. From the initial analysis, the linear elastic base shear and roof displacement (V_e and Δ_e) are extracted and plotted. The member responses from the analysis are examined and, by interpolation, values of the base shear and roof displacement (V_1 and Δ_1) are calculated at which significant yielding for one or more structural elements is initiated.

The elastic model is modified with plastic hinges for the yielding elements. The rotational ductility capacity of these elements is determined and tabulated for future monitoring. The modified structural model is subjected again to the spectral response analysis and an incremental base shear and roof displacement (V_{1-2} and Δ_{1-2}) are calculated and plotted by superposition, as shown in the figure.

The process is repeated and for each iteration the structure is monitored for the initiation of global instability. Instability is considered to occur if the structure overturns or if the rotational ductility of any structural element is exceeded. The structure in this example is acceptable as the displacement capacity (Δ_u) exceeds the elastic displacement (Δ_e) for the evaluation spectrum.

The above example illustrates the procedure for a reinforced concrete or structural steel moment frame. The limit states for the rotational ductility of the joints in the concrete frame may be defined by the allowable strains in the concrete and the reinforcement or the initiation of bond slip. For steel frames the limit states may be allowable tensile strain in the beam and column flanges, shear strain in the column web, or initiation of local buckling.

The procedure is applicable to any structure wherein the post-yield response of its elements can be approximated by a rotational hinge. As indicated in the above frame example, yielding may occur initially at one end of a flexural member prior to the other end. For a shear element, the post-yield response would be approximated by a rotational hinge at both ends of the element and the translation of upper hinge would be monitored to restrict the inelastic strength degradation of the element within acceptable limits.

2. Step-by-step Example

The following step-by-step procedure illustrates the use of static or pseudo-dynamic pushover analysis as an acceptability check applicable to any of the design approaches described in Chapter 3.

1. Model the elastic structure as shown in Figure G-1.
2. Modify design response spectrum as shown in Figure G-2.
3. Perform an elastic analysis. Plot the elastic base shear, V_e, and roof displacement, Δ_e. Also find corresponding values, V_1 and Δ_1, for first significant yielding as shown in Figure G-3.
4. Modify the elastic model with plastic hinges at ①. Determine the rotational ductility capacity of hinges at ① from available research data and/or by analysis (e.g., for reinforced concrete frames, the rotational ductility may be governed by: a) yield or bond slip of reinforcement or, b) by crushing of the concrete. For steel frames, it may be governed by local or global buckling of the member).
5. Perform iterative elastic analysis with the modified model and plot incremental base shear and roof displacement for next significant yielding, $V_{1\text{-}2}$ and $\Delta_{1\text{-}2}$ as shown in Figure G-4.
6. Repeat the procedure until a mechanism is formed, such as when plastic hinges ③ are formed (i.e., no additional shear capacity in first story) as shown in Figure G-5.
7. From the rotational ductility capacities at ①, ②, or ③, determine additional allowable roof displacement, $\Delta_{3\text{-}4}$ as shown in Figure G-6.
8. Idealize curve as shown to provide V_y and Δ_y.
9. Note for this example the global demand IDR (i.e., force reduction factor) is $\frac{\Delta_e}{\Delta_y} = \frac{V_e}{V_y}$. The allowable global displacement ductility is $\frac{\Delta_\mu}{\Delta_y}$. If Δ' represents roof displacement at which damage is initiated (e.g., exceeding 0.005 drift in any story), damage for this example is approximated by $\frac{\Delta_e - \Delta'}{\Delta_\mu - \Delta'}$.

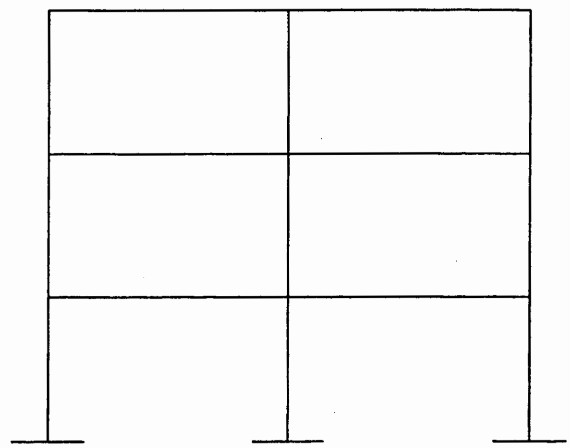


Figure G-1: Elastic Structure

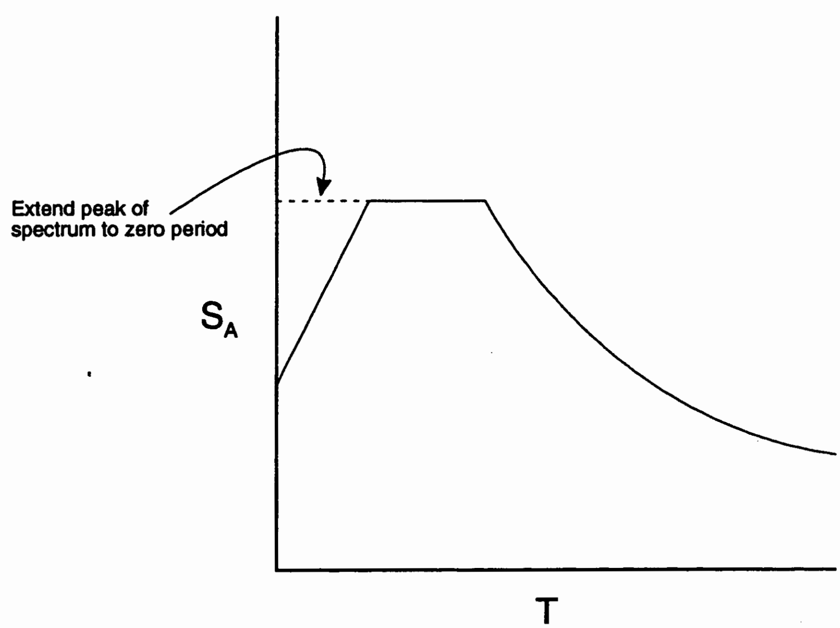


Figure G-2: Design Response Spectrum

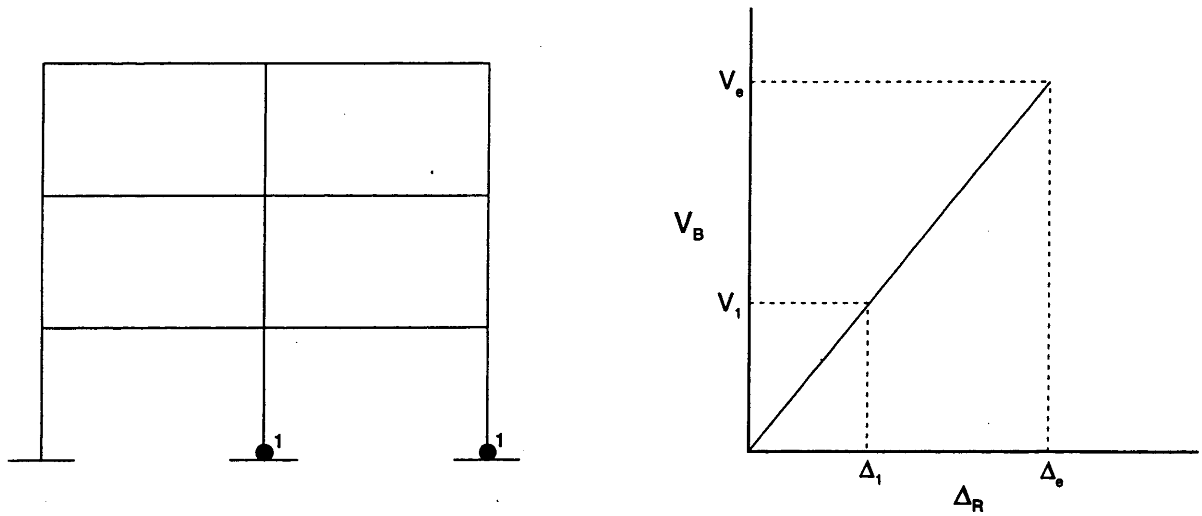


Figure G-3: State of First Significant Yielding

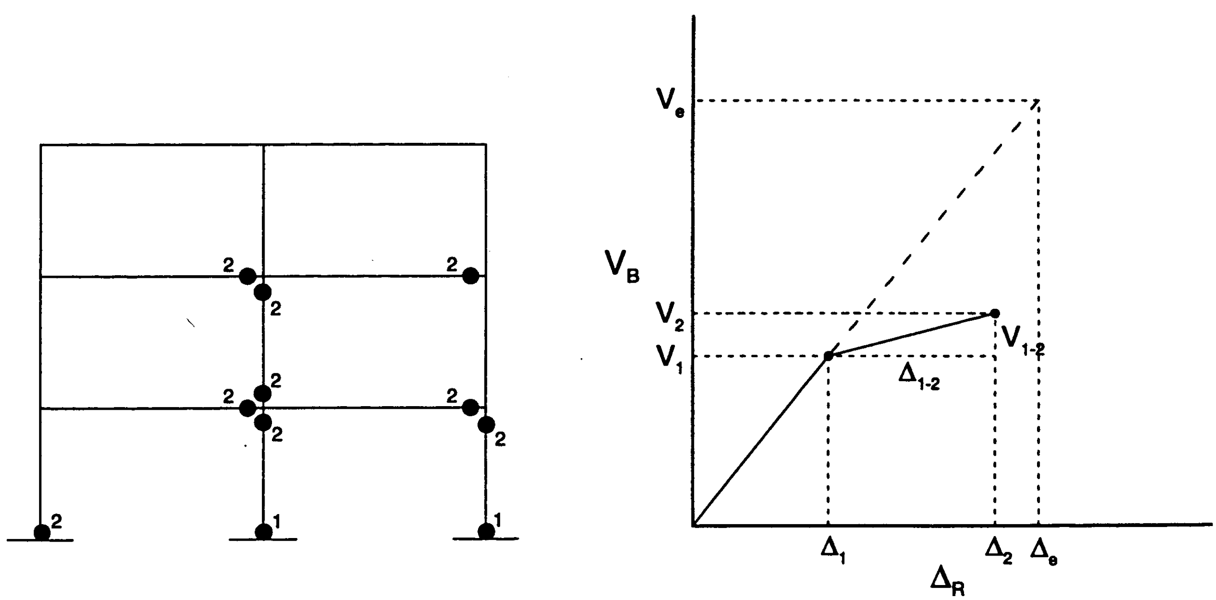


Figure G-4: Iterative Elastic Analysis Results

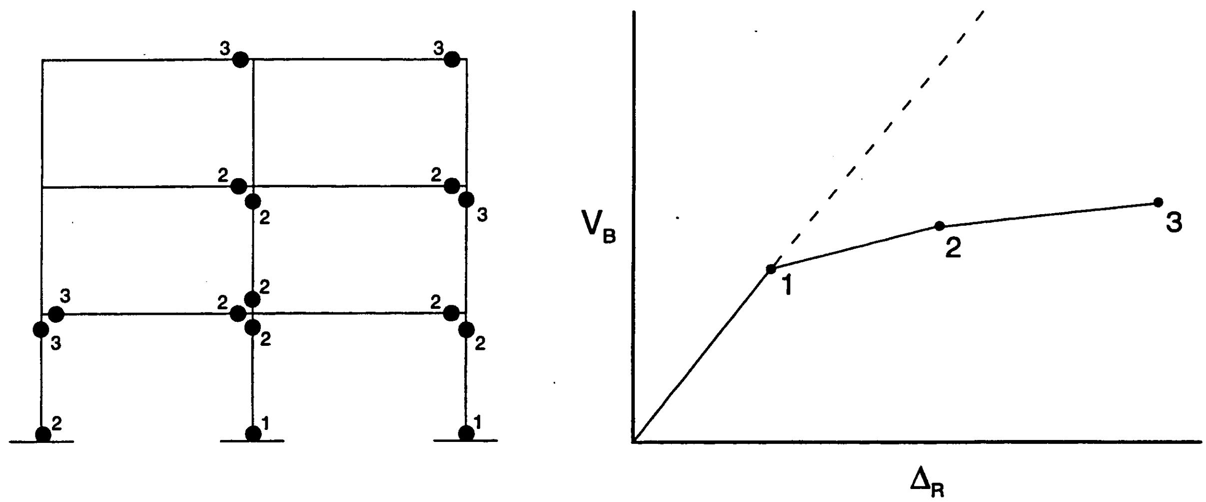


Figure G-5: Collapse Mechanism Formed

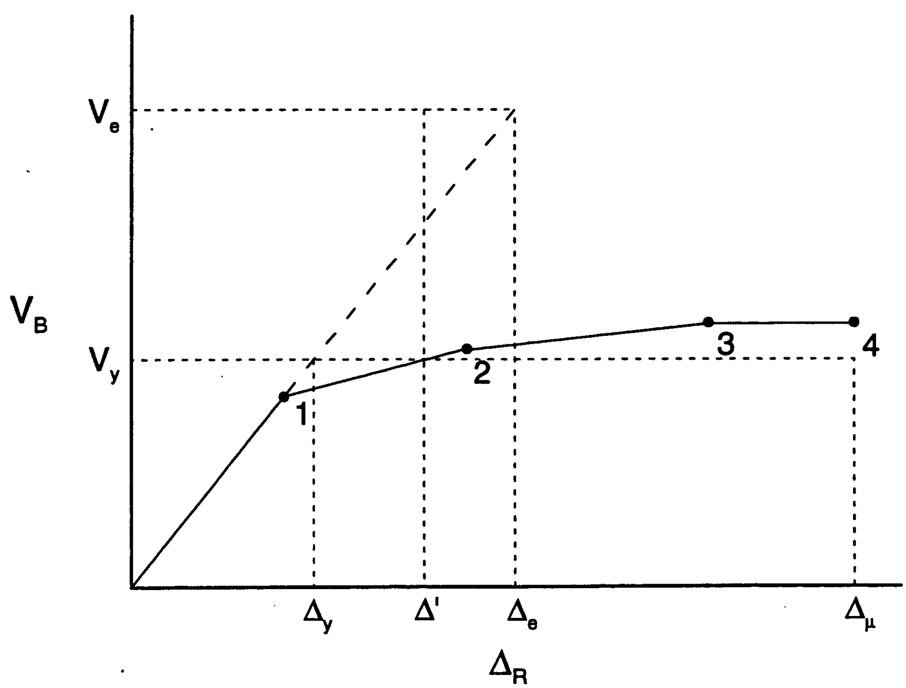


Figure G-6: Static or Psuedo-Dynamic Pushover Analysis

APPENDIX H

LINEAR AND NONLINEAR DYNAMIC ANALYSIS OF MULTI-DEGREE-OF-FREEDOM STRUCTURES (MDOFS)

by

Vitelmo V. Bertero

H.1 Introductory Remarks

The conceptually ideal method for checking the acceptability of the performance of a preliminarily designed (and particularly of a designed or already existing) structure under the different levels of EQGMs to which it can be subjected during its service life is discussed in detail in the performance-based comprehensive EQ-RD approach. It is the use of 3D linear and nonlinear time-history dynamic analysis of realistic mathematical models of the entire facility system, i.e., the soil surrounding the foundation, the foundation, the superstructure and the so-called nonstructural components and contents. A discussion of the state of the practice and of the art in the use of 3D dynamic linear and nonlinear analysis in EQ-RD follows, starting with the use of existing 3D linear dynamic analysis computer programs, presented in recent notes by Wilson et al. [1].

H.2 Three-dimensional Linear Dynamic Analysis

A. *State of the Practice*

The UBC requires linear dynamic analysis for a large number of different types of building structural systems constructed in seismic zones 2, 3 and 4. A 3D model shall be used for a dynamic analysis with highly irregular configuration, such as those having a plan irregularity and a rigid or semi-rigid diaphragm. Although the present 1994 UBC gives some guidelines regarding structural irregularity, application of accidental torsion and treatment of orthogonal loading effects, these guidelines appear to be vague, and large numbers of designers are having difficulty in interpreting both these guidelines and the UBC requirements for scaling the estimated dynamic (computed) base shears to the static base shears for each direction. The 1994 UBC requires that dynamic methods of analysis must be used in the principal horizontal directions. Because the design base shear can be different in each direction, the UBC scaled spectra approach can produce a different input in each direction for both regular and irregular structures. Therefore *the current code dynamic analysis approach can result in a structural EQ-RD that is relatively weak in one direction.*

B. *State of the Art*

In Reference 1, Wilson et al. present a 3D computer model and method of linear dynamic analysis which produces displacements and member forces that will satisfy the 1994 UBC. This method can be used for both regular and irregular structures. The major steps in the approach are as follows.

1. A 3D computer model must be created in which all significant structural elements are modeled. This model should be used in the early phases of design, since it can be used for both static and dynamic loads.

2. The 3D mode shapes should be repeatedly calculated during the design of the structure. The directional and torsional properties of the mode shapes can be used to improve the design. A well-designed structure should have a minimum amount of torsion in the mode shapes associated with the lower frequencies of the structure.

3. The direction of the base reaction of the mode shape associated with the fundamental frequency of the system is used to define the principal directions of the 3D structure.

4. The "design base shear" is based on the shortest period obtained from the computer model, unless the code requires the use of Method A.

5. Using the CQC method, the "dynamic base shears" are calculated in each principal direction doe to 100% of the Normalized Spectra Shapes. Use the minimum value of the base shear in the principal directions to produce one "scaled design spectrum." Also, consider using a scale factor based on overturning moments.

6. The dynamic displacements and member forces are calculated using the SRSS value of 100% of the scaled design spectra applied nonconcurrently in any two orthogonal directions.

7. A pure torsion static load condition is produced using the suggested vertical lateral load distribution defined in the code.

8. The member design forces are calculated using the following load combination rule:

   $$F_{DESIGN} = F_{DEAD\ LOAD} \pm [F_{DYNAMIC} + | F_{TORSION} |] + F_{OTHER}$$

   The dynamic forces are always positive and the accidental torsional forced must always increase the value of force. If vertical dynamic loads are to be considered, a dead load factor can be applied.

One can justify many other methods of analysis that will satisfy the current code. The approach presented in this paper can be used directly with the computer programs CSI-ETABS [2] and SAP90 [3] with their steel and concrete postprocessors. Since these programs have very large capacity and operate on inexpensive personal computers, it is possible for a structural engineer to investigate a large number of different designs very rapidly with a minimum expenditure of manpower and computer time.

C. *Concluding Remarks*

After being associated with the 3D dynamic analysis and design of several hundred structures during the past 30 years, the authors would like to take the opportunity to offer some constructive comments on the lateral load requirements of the current code. It is hoped that they will be considered before the next edition is released. In Reference 1, the authors make the following remarks.

1. *The use of the "dynamic base shear" as a significant indication of the response of a structure is fundamentally incorrect.* An examination of the modal base shears and overturning moments clearly indicates that base shears associated with the shorter periods produce relatively small overturning moments. Therefore, a dynamic analysis, which will contain higher mode response, will always produce a larger dynamic base shear relative to the dynamic overturning moment. Since the code allows all results to be scaled by the ratio of dynamic base shear to the static design base shear, the dynamic overturning moments can be significantly less than the results of a simple static code analysis. A scale factor based on the ratio of the "static design overturning moment" to the "dynamic overturning moment" would be far more logical. The static overturning moment can be calculated by using the static vertical distribution of the design base shear which is currently suggested in the code. *Responsible structural engineers should investigate the dynamic overturning requirements of tall structures in which higher-mode response exists.*

2. *For irregular structures, the use of the terminology "period (or mode shape) in the direction under consideration" should be discontinued.* The stiffness and mass properties of the structure define the directions of all 3D mode shapes. The term "principal direction" should not be used unless it is clearly and uniquely defined.

3. *There is no logical reason for scaling the results of a dynamic analysis.* The use of site-dependent spectral shapes would eliminate many of the problems associated with the current code.

4. *It is not necessary to distinguish between regular and irregular structures when a 3D dynamic analysis is conducted.* If an accurate 3D computer model is created, the vertical and horizontal irregularities and known eccentricities of stiffness and mass will cause the displacement and rotational components of the mode shapes to be coupled. A 3D dynamic analysis, based on these coupled mode shapes, will produce a far more complex response, with larger forces, than the response of a regular structure. It is possible to predict the dynamic

force distribution in a very irregular structure with the same degree of accuracy and reliability as the evaluation of the force distribution in a very regular structure. Consequently, if the design of an irregular structure is based on a realistic dynamic force distribution there is no logical reason to expect that it will be any less earthquake resistant than a regular structure designed using the same dynamic loading. The major reason why many irregular structures have a documented record of poor performance during earthquakes is that their designs were often based on approximate two-dimensional static analyses.

H.3 Three-dimensional Nonlinear Dynamic Analysis

A. *State of the Practice*

As pointed out in the discussion of the comprehensive EQ-RD approach, conceptually the ideal methods of analysis for designing against the requirements imposed by any of the performance design objectives involving acceptance of larger deformation, and particularly acceptance of inelastic deformation, is by creating realistic 3D nonlinear computer models of the entire building system and conducting time-history dynamic analyses of these models. Although, as discussed below under the State of the Art, there are at present a series of computer programs available to conduct 3D nonlinear dynamic analyses, very few building designers have had experience with the use of these 3D programs. In the last decade, some engineers have had considerable experience with the use of 2D nonlinear programs, particularly DRAIN-2D [4] and lately DRAIN-2DX [5].

B. *State of the Art*

There are some general 3D nonlinear computer programs that can be used in performance-based EQ-RD. These programs can be classified into the following two groups.

- ***General programs***. Some of these programs are: ABACUS, ADINA, ANSYS, FACTS, DRAIN-3DX, and SEASTAR.

- ***Special-purpose programs***. There are many of these programs, such as: ARCS (for RC sections), NEABS (for bridges) and IDARC-3D.

A special-purpose program will usually be more efficient and easier to use, but less flexible. However, none of these available programs is easy to use. Major improvements are needed before the current programs become really useful tools for performance-based EQ-RD.

H.4 References

1. Wilson, E. L., Suharwardy, M. I. and Habibullah, A. (1995) "Seismic Analysis Method Which Satisfies the Current Lateral Force Requirements For Buildings," notes distributed in the Short Course on Advances in Earthquake Engineering Practice, May 31-June 4, 1994. Organized by EERC and Continuing Education in Engineering, University of California at Berkeley.

2. "CSI-ETABS: A Computer Program for the Three-Dimensional Analysis of Building Systems," Computers and Structures, Inc., 1918 University Ave., Berkeley CA 94704.

3. "SAP-90: A General Computer Program for the Three-Dimensional Analysis of Finite Element Systems," Computers and Structures, Inc., 1918 University Ave., Berkeley CA 94704.

4. Kanaan, A. E. and Powell, G. H (1973) "DRAIN-2D: A General Purpose Computer Program for Dynamic Analysis of Inelastic Plane Structures," *Report No. EERC/UCB-72/22*, Earthquake Engineering Research Center, University of California at Berkeley, April.

5. Powell, G. H. (1992) "DRAIN-2DX Version 1.00, User Guide Sections," Department of Civil Engineering, University of California at Berkeley.

6. Powell, G. H. (1994) "DRAIN-3DX Manual," Department of Civil Engineering, University of California at Berkeley.

H.5 Simplified Linear and Nonlinear Dynamic Analysis of Multi-Degree-of-Freedom Structures

A. *Introductory Remarks*

As discussed in detail in the performance-based comprehensive EQ-RD approach, to check the acceptability of the preliminary design and particularly of a designed (or already existing structure) regarding performance under the different levels of EQGMs to which it can be subjected during its service life, conceptually the ideal method is the use of 3D linear and nonlinear time-history dynamic analysis of realistic mathematical models of the entire facility system, i.e., the soil surrounding the foundation, the foundation, the superstructure and the so-called nonstructural components and contents. However, the 3D nonlinear dynamic analysis of a multi-degree-of-freedom system (MDOFS) is not only time-consuming but because it is based on many assumptions involving many uncertainties, the reliability of the predicted performance is questionable and very difficult to judge. These uncertainties include:

- the reliability of the model that is used and its main dynamic characteristics, particularly in its nonlinear (inelastic) range of a behavior,
- the dynamic characteristics of the ground motion, and
- the reliability of a method used in solving the large number of the differential equations involved.

It is very difficult to envision the dynamic response of a structure when it involves many degrees of freedom. In EQ-RD, the importance of being able to envision the dynamic performance of the entire facility system, i.e., to be able to feel its real behavior and therefore to judge such behavior qualitatively, cannot be overemphasized. A quantitative study gives only a quantitative answer, whose reliability depends on the basic assumptions made. Thus, to make rational decisions regarding the probable performance of a building based on just quantitative analysis, it is necessary to conduct several quantitative studies using different basic assumptions, and then to carry a comparative analysis of the different answers obtained. As this can not be done economically for most structures, simplified methods of 3D dynamic analysis and particularly of nonlinear time-history methods of analysis should be used. For cases of low, medium and high-rise buildings having regular configurations, structural layouts and structural systems, it is possible to use a reduced MDOFS, the "stick model" with just one degree of freedom per floor, or even just an equivalent SDOFS.

B. *Simplified Method of Linear and Nonlinear Dynamic Analysis of MDOFS Based on the Use of a Stick Model*

Usually, the simplified model used for this type of simplified analysis consists in modeling the real building with two-dimensional (2D) shear elements with only one translational degree of freedom per floor. Formulation of this simplified model requires knowledge of three main dynamic characteristics of the structure out of the following four: (a) reactive mass, (b) lateral stiffness, (c) vibration mode shape and frequency, and (d) the lateral strength-displacement relationship for each story.

The main purpose of this simplified stick model is to have a MDOFS model with a relatively few degrees of freedom when compared with the number of degrees of freedom for a detailed 3D finite element model, which therefore will permit the performance of time-history analyses quickly in order to conduct comparative parametric studies. To give an idea of the possible reduction in computer time, a stick model for a thirty-story reinforced concrete building (Anderson et al., 1991) consisted of 31 degrees of freedom versus 6816 degrees of freedom for the detailed 3D finite element model.

C. *Simplified Method of Linear and Nonlinear Dynamic Analysis of MDOFS Based on the Use of an Equivalent SDOFS*

For low-rise buildings with regular configuration, structural layout and structural system throughout each of their stories and throughout their heights, it is possible to reduce the analysis of their real MDOFS not only to the analysis of stick models, but even to that of equivalent generalized SDOFS models, by proper selection of their lateral deflection shape [Clough and Penzien (1993)].

D. References

1. Anderson, J. C. et al. (1991a) "Evaluation of the Seismic Performance of a Thirty-Story RC Building," *Report No. UCB/EERC-91/16*, July 1991. Earthquake Engineering Research Center, Univ. of California at Berkeley.

2. Clough, R. and Penzien, J. (1993) <u>Dynamics of Structures</u>, McGraw-Hill, Inc.

3. Miranda, E. (1991) "Seismic Evaluation and Upgrading of Existing Buildings," Ph.D. dissertation in Civil Engineering, University of California at Berkeley, April 1991.

APPENDIX J

TRI-SERVICE MANUAL METHODS

by

Vitelmo V. Bertero

J.1 Overview of Methods

The Tri-Services Manual, entitled, "Seismic Design Guidelines for Essential Buildings" (Ref. 1), recommends an approach for the design of essential buildings that, although it is not referred to directly as a performance-based EQ-RD, is based on two levels of EQGMs, and therefore can be considered a performance-based EQ-RD or, as will be shown later, can easily be adapted for such purposes. The first level of EQGM recommended in this manual, denoted as EQ-I, is smaller in damage potential than the recommended second level, EQ-II. The lateral force resisting structural systems are recommended to be designed to resist EQ-I by just elastic response (behavior) using the smoothed linear elastic design response spectra (SLEDRS) corresponding to EQ-I. Then the structural systems should be evaluated to check their ability to resist EQ-II by post-elastic (i.e., inelastic) behavior with prescribed limited deformation ductility ratios. This manual recommended the following two different methods to conduct the evaluation of the designed structure for EQ-II.

Method 1: Elastic Analysis Methods. The procedure used in this elastic method consists in the evaluation of the overstresses at the critical sections of the individual structural members calculating what are called the *inelastic demand ratios (IDR)*, which are equal to the ratio of Demand Forces (M_D, V_D, F_D) to the calculated yield or plastic capacities of the elements (M_C, V_C, F_C). If the IDR exceed acceptable values, or four other conditions, the structure must be analyzed in accordance with *Method 2*. The acceptable values for the IDR vary from 1 to 2.0 for essential facilities; 1.25 to 2.5 for high risk facilities; and from 1.25 3.0 for all other facilities. These values seem reasonable; however, in the practical application of this method, they are usually increased up to *8*. As is shown in Appendix A, these increased values are reached when the structures are designed according to present regulations and the effect of gravity loads is negligible.

Method 2: Capacity Spectrum Method. A step-by-step approach is recommended for use in estimating or approximating the inelastic capacity spectrum of any given structure. Then this capacity is compared to the demands of the EQ-II response spectrum. In Reference 3, Freeman recently pointed out that, "The Capacity Spectrum Method can be used to approximate the performance level that a building will be subjected to during an EQ." This method is discussed in some detail in this reference.

J.2 Advantages of Method 2

Although the numerical procedure for estimating the capacity spectrum can be questioned (particularly the determination of the change in the value of the fundamental period or the effective period of vibration), there is no doubt that conceptually Method 2, which requires the estimation of the capacity of the structure to resist lateral forces, is a step forward towards the improvement of earthquake-resistant design methodology. This method has gained popularity among professional designers, as well as in some research.

J.3 Disadvantages of Method 2 for Conducting the Performance Based EQ-RD of Structures

In the application of Method 2, as described and applied in the Tri-Services manual, the following two recommended procedures can be seriously questioned: *first*, the recommended procedure to estimate the *demand curve*, which is obtained by estimating a reduction (or de-amplification) in response amplification due to inelastic action; and *second*, the comparison of the *capacity spectrum* with the *demand curve*. The reduction in response is done by reducing the spectral values of the SLEDRS corresponding to EQ-II through the use of a larger value of the viscous damping ratio, , as illustrated in Fig. J-1. The real problem with this method is that conceptually the actual SIDRS cannot be obtained from a SLEDRS by only increasing the value of up to 10% or even up to 20%. Usually this method can be quite conservative, particularly for structures having a certain period, T [T close to the value of the predominant period (frequency) of the ground motion], and possessing a ductility displacement ratio equal to or larger than 1.5. It is recommended that to improve the present manual procedure for its use in performance-based EQ-RD, the computed capacity response spectrum be compared to the demand spectra obtained by estimating the SIDRS corresponding to the C_y, as well as the that results from the estimated lateral resistance deformation capacity curve illustrated in Fig. J-2.

The direct comparison of the *capacity spectrum* (estimated according to the Tri-Service manual) with the *earthquake demand curve* can be questioned because the estimated change in the effective period of vibration can only become effective in reducing the response of the structure in the event that the ground motion time history is such that it induces in the structure a response that consists of repeated cycles of deformation reversals whose peak intensity (magnitude) increases with time from the first yielding to a reversal requiring the maximum available displacement ductility. The comparison will be incorrect in the case of impulsive types of EQ ground motion, which would require the structure to develop its deformation in just one cycle of inelastic behavior.

J.4 Recommendations for Improving and Adapting Tri-Services Manual Methods to Performance-Based EQ-RD

It is believed that the use of these methods together with the improved static linear and nonlinear pushover analyses described in Appendix G can be easily adapted for practical applications to the performance-based EQ-RD philosophy based on the above suggested improvement to the method and the following modifications.

1. Plot, as illustrated schematically in Figs. J-3 and J-4, the strength and displacement SDRS corresponding to each of the design levels of EQGMs, i.e., the SDRS corresponding to each of the multiple performance design objectives (service, functional, safety and impending collapse). *These SDRS are the Design EQ Demand Curves or Spectra.*

2. Conduct the preliminary design of the structure for one (or more) of the performance design objectives that appear to control the design.

3. Check the acceptability of the resulting preliminary design against the requirements of each of the established performance design objectives by:

   - first, conducting a static linear and nonlinear pushover analysis of the preliminary design structures, according to the improved procedure recommended in Appendix G (Fig. J-5);
   - second, plotting the strength capacity spectrum on the design EQ strength demand spectra and the displacement capacity spectrum on the design EQ displacement demand spectra (Fig. J-6);
   - third, comparing the values of the available capacity strength at the different desired or established limit states (performance levels) given by the strength capacity spectrum of Fig. J-6a against the demanded strength at each of the desired performance levels also given in Fig. J-6a, it is possible to check if the preliminary design is acceptable from this point of view;
   - fourth, comparing the values of the resulting lateral displacements (at a selected height level of the structure) at each of the different desired limit states given by the lateral displacement-resistance curve of Fig. J-5 and spectral values of the demanded displacement response (Fig. J-6b), it is possible to check if the preliminary design is acceptable from the point of view of the damage that could result as a consequence of the lateral displacement.

It should be noted that comparison of the Design EQ Displacement Demand Spectrum at each of the desired performance design objectives with the established maximum acceptable value for the lateral displacement allows determination of the maximum acceptable fundamental period, T, for the structure to be designed. Thus, as soon as the effective period of the preliminarily designed structure is computed at each of the limit states involved in the performance design objectives, which can be done using the resistance-deformation curve obtained from the pushover analysis) by comparing them with the values of T that have been determined from comparing the displacement demand spectra with the corresponding displacement acceptable values, one can recognize if the preliminary design is acceptable or not from the point of view of its stiffness.

Because according to this approach the acceptability checks are conducted on the basis of SDOF design response spectra and the results from a static pushover analysis, it is clear that its application should be limited to structures whose seismic response can reliably be predicted using SDOFS analyses, i.e., structures with regular configuration, structural layout and structural system throughout the plan of each story and along the height, and where the effects of EQGMs combined with the effects of other loads cannot lead to significant effects of cumulative plastic deformations as discussed in Appendix A.

J.5 Research Needs

- The reliability of the demand curves by earthquake response spectra using the different values of linear viscous damping to represent effect of inelastic behavior has to be investigated.
- The reliability of the recommended procedure to determine the capacity spectrum curve, particularly the way that the equivalent effective period is computed, needs to be analyzed considering different types of EQGMs.
- How the procedure for preliminary EQ-RD can be applied considering the four performance levels needs to be considered.

J.6 References

1. Departments of the Army, the Navy and the Air Force (1988), "Seismic Design Guidelines for Essential Buildings," September.

2. Departments of the Army, the Navy and the Air Force (1988), "Seismic Design Guidelines for Upgrading Existing Buildings," September.

3. Freeman, S. A. (1994), "Capacity Spectrum Method," Proceedings of the 1994 Fall Seminar on the Development Art of Seismic Engineering, organized by SEAONC, November 8, 15 and 22, San Francisco CA.

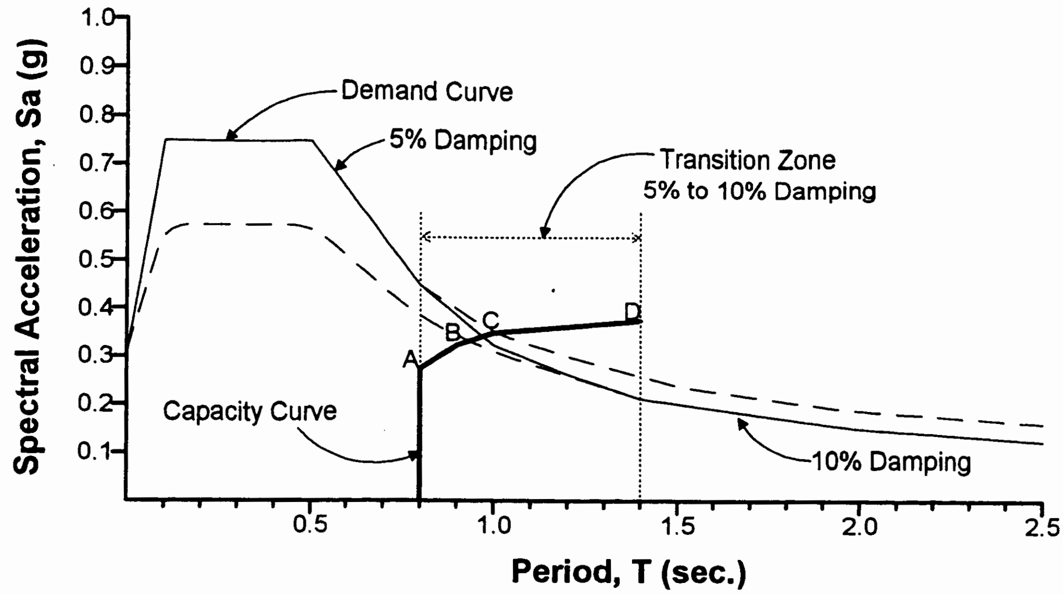


Figure J-1: Capacity Spectrum Method [1]

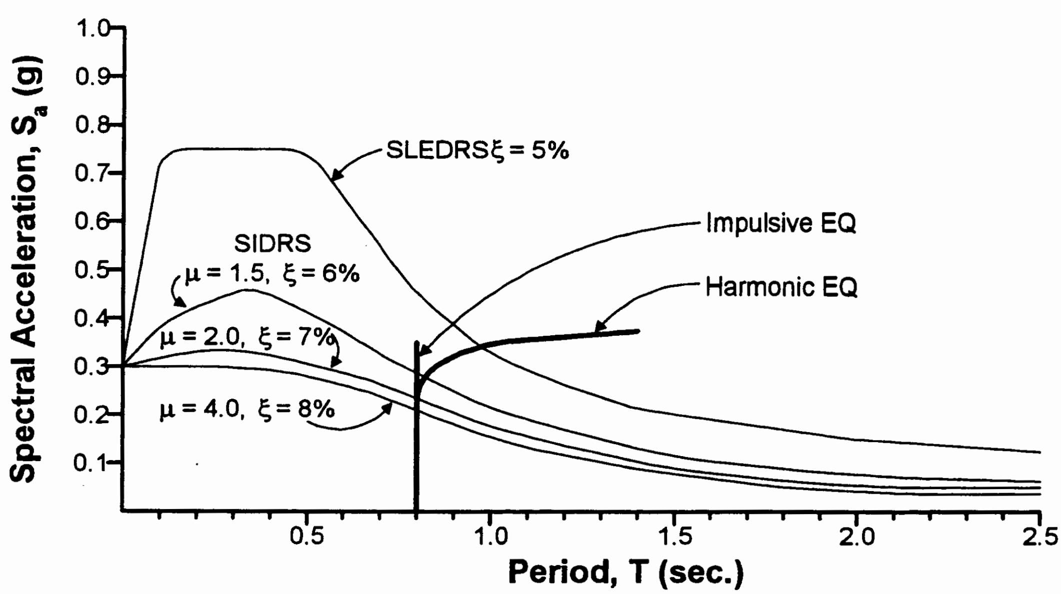


Figure J-2: Recommended Capacity Spectrum Method

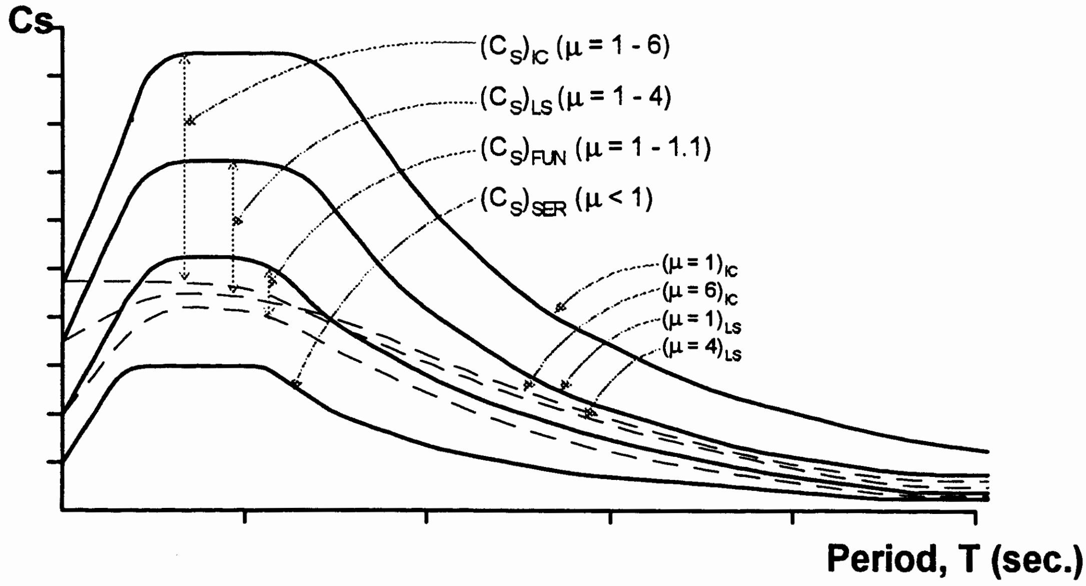


Figure J-3: Strength-Smoothed Design Response Spectra

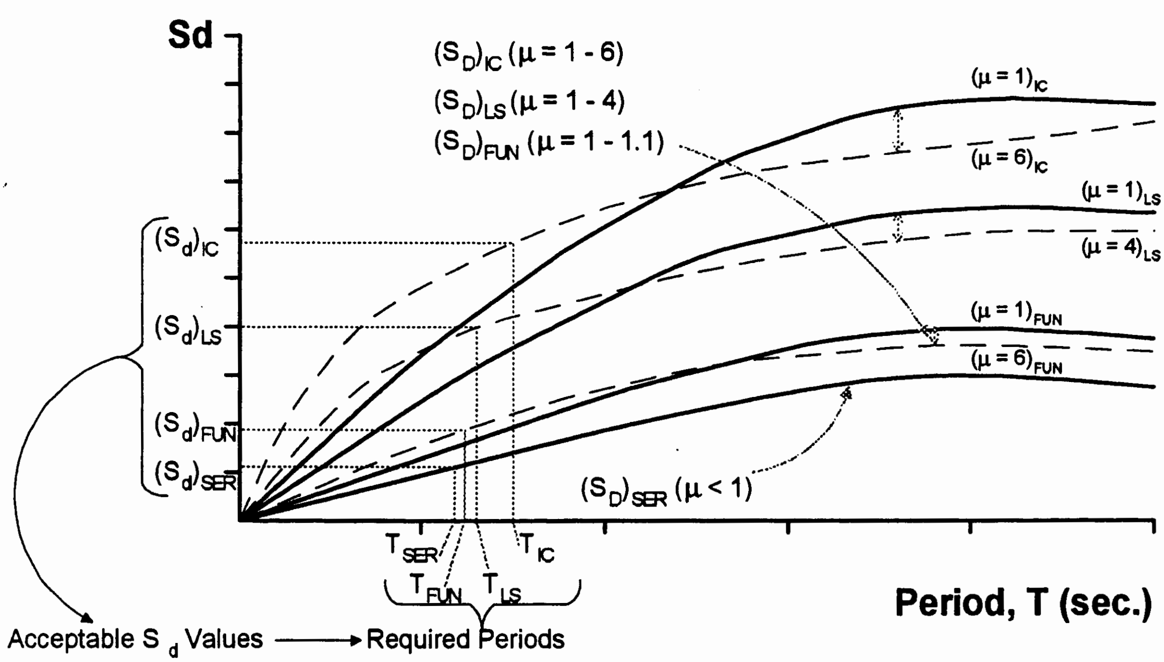


Figure J-4: Displacement-Smoothed Design Response Spectra

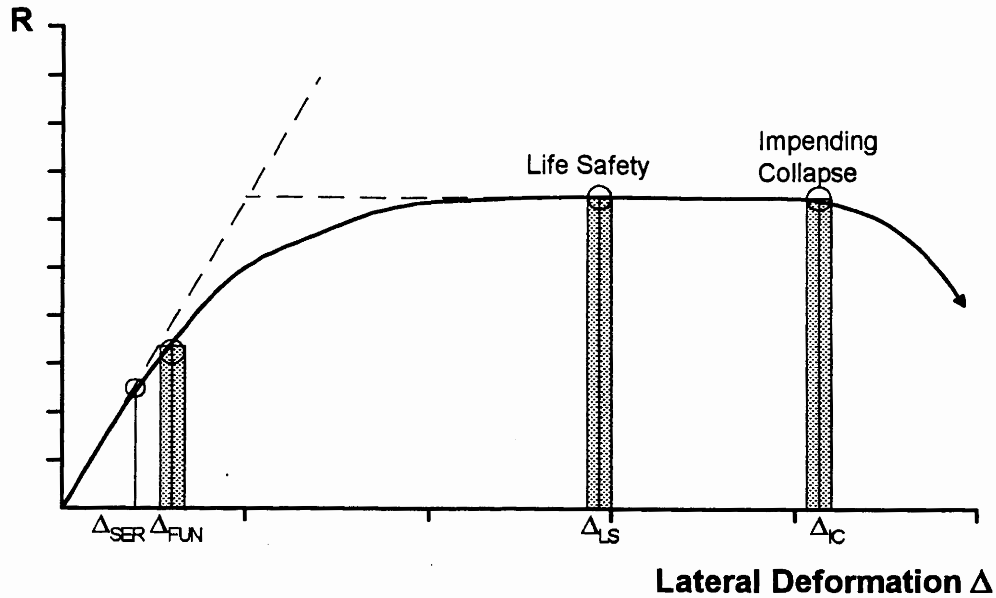


Figure J-5: Monotonically Increasing Lateral Deformation-Resistant Relationship: Pushover Analysis

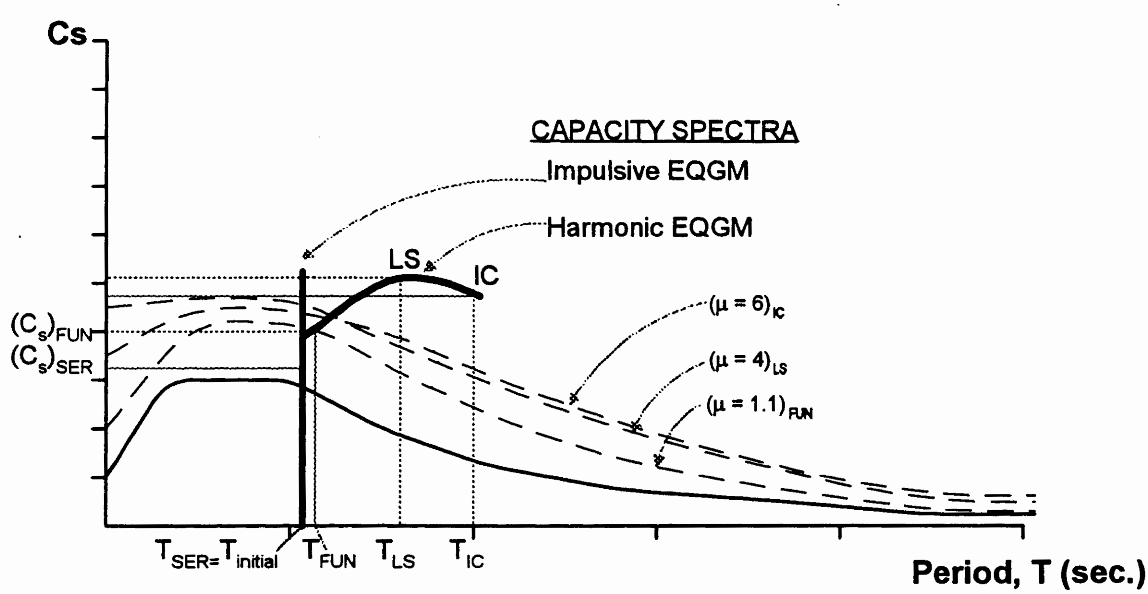


Figure J-6a: Comparison of Capacity Spectra with Design Response Spectra

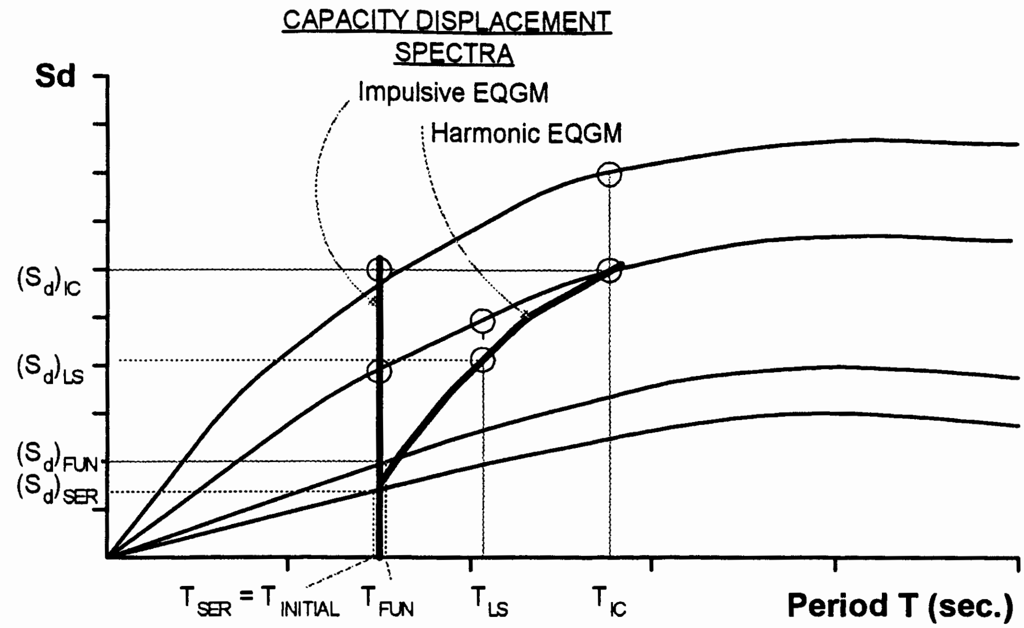


Figure J-6b: Comparison of Displacements Corresponding to Capacity Spectra with Displacement Design Spectra Demands and with Acceptable Values for the Displacements

Structural Engineers Association of California

VISION 2000

Performance Based Seismic Engineering of Buildings

Part 3: Preliminary Northridge Lessons

Prepared for:

California Office of Emergency Services

Prepared by:

Structural Engineers Association of California
Vision 2000 Committee

April 3, 1995
Final Report

Performance Based Seismic Engineering of Buildings

Part 3: Preliminary Northridge Lessons

prepared by

Structural Engineers Association of California

Vision 2000 Committee

Chris Poland, Chair
James Hill, Vice Chair
Roland Sharpe, Vice Chair

delegates:

Vitelmo Bertero
Joseph Nicoletti
Ronald Hamburger
Nabih Youssef
Anthony Court
Scott Stedman
Ajit Virdee
Robert Chittenden
Richard Phillips

editor:

Jeffrey Soulages

ad hoc members:

Robert Thacker
Vilas Mujumdar
Charles Thiel, Jr.

ex-officio members:

Ronald Nelson
Arthur Ross
Jane Andersen

contractors:

Richard Fallgren

555 University Avenue, Suite 126
Sacramento, CA

April 3, 1995

SEAOC Vision 2000 Committee

In recognition of the need for a new generation of design procedures, the Structural Engineers Association of California (SEAOC) has formed a special committee to look into the future and develop a framework for procedures that will yield structures of predictable seismic performance. This committee, called Vision 2000, first started their deliberations in 1992 and continues today with a multifaceted effort that is aimed at both the final goal and at the process that will deliver the profession to that end. While Vision 2000 intends to identify the framework, others are expected to translate it into design guidelines and codes as appropriate. Within SEAOC, the Seismology Committee will incorporate the new concepts, as they are available and appropriate, into future editions of the SEAOC Blue Book.

The first step of the Vision 2000 Committee was to establish the project parameters. These project parameters were incorporated into a Goal/Mission Statement. The goal of the Vision 2000 is as follows:

> **The goal of the Vision 2000 Committee is to develop the framework for procedures that yield structures of predictable seismic performance.**

This framework will explicitly address life-safety, damageability, and functionality issues. The intent will be to produce structures whose seismic performance is predictable with a reasonable degree of confidence provided the actual materials and construction accurately reflect the design.

The second step was to establish a work plan and timeline for both the short-term phase of the project and for the mid-term and long-term phases of the project. A copy of this Workplan is available from the SEAOC Office.

The third step was to create the Vision 2000 Report which includes the results from all phases of the short-term work of the committee. This report includes: performance based engineering definitions, interim recommendations on how to apply performance based engineering concepts using today's available standards, a conceptual framework for a future performance based design code, preliminary lessons from the Northridge Earthquake, and a road map on moving the SEAOC Blue Book towards performance based engineering concepts. This report is divided into four parts as follows:

- Part 1: Interim Recommendations
- Part 2: Conceptual Framework
- Part 3: Preliminary Northridge Lessons
- Part 4: Moving the Blue Book Towards Performance Based Engineering

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1. INTRODUCTION

1.1 Background

The primary goal of building codes is to provide life safety to the occupants. For seismic conditions, this has been traditionally accomplished by designing buildings with sufficient strength, integrity, and toughness to resist collapse or the generation of large falling debris. The basis of the current provisions is a combination of research, analytical studies and the performance of buildings in past earthquakes.

In recent years, California, due to its active seismicity and relatively modern inventory of buildings, has proven to be a good laboratory in which to judge the effectiveness of the current code provisions. What has been observed is both encouraging and challenging. While it has been observed that buildings designed to the current standards have performed at or above expectations, the usability of the buildings after the earthquake and the cost of repairs have been a surprise. It is now generally held that in areas of active seismicity, it is not sufficient to consider only a life-safety performance level for seismic design. Consideration needs to also be given to designing in a manner that limits the cost of repairs and the time needed to complete the repairs.

SEAOC has been an active participant in the development of building codes for the past 40 years. Most of the currently used code provisions can be traced back to the SEAOC Bluebook and the ATC 3-06 effort. SEAOC recognized the need for expanding their design guidelines a number of years ago and chose to do it with a special committee that would focus on the future of seismic design standards. That group, the SEAOC Vision 2000 Committee, was formed originally in 1992 and has set about the task of first defining and then assisting in the development of Performance Based Engineering standards.

Performance Based Engineering involves the complete design and construction support activities needed to construct buildings that will resist earthquakes in a predictable manner. It includes the selection of an appropriate suite of possible earthquakes and acceptable performance levels for each one. It involves determination of site suitability, conceptual design, preliminary design, final design, acceptability checks during design to validate that the designated performance objectives are being met, design review, quality assurance during construction, and provisions that require maintenance of the system during the life of the building. Each step is critical to the success of the design and each must be addressed to a level suitable to the performance objective selected.

In order for Performance Based Engineering procedures to be relevant and useful, they must be calibrated against the performance of real buildings in actual earthquakes. This calibration requires collection of data on structure type, ground motion intensity, and structural and other types of damage which can then be related to design performance objectives. Calibration, however, must be viewed as necessary but not sufficient for validating new procedures. Each earthquake is unique with new lessons learned after each occurrence. As new procedures are developed and tested, they must be permitted to identify new problems, even before they are observed, as long as they can accurately predict what has been observed historically. For example, a new analytical procedure that predicts the collapse of steel moment frame buildings in strong earthquakes must also be able to show the adequacy of all steel moment frames to resist collapse that were shaken by the Northridge Earthquake.

The Northridge Earthquake, which shook the entire Los Angeles metropolitan area with a wide range of ground motion intensities and resulted in damage as severe as any previously experienced in California, provides one opportunity to perform this calibration. The Vision 2000 Committee, recognizing that this calibration was important, established it as Task C. The objective of this task, as stated in the 1994 Vision 2000 Work Plan, is as follows:

> *Compile statistics on the general behavior of buildings in the Northridge earthquake in terms of their observed performance and in light of the strong motion each experienced. This information will permit a first-level calibration of the performance expectations for buildings designed to the currently available standards.*

The use and interpretation of this data must recognize the unique characteristics of the Northridge ground motion. The Northridge earthquake was a thrust fault earthquake that occurred in a modern, densely populated suburban environment. The ground motion was of rather short duration, though composed of both high frequency/high acceleration motion and at some locations large velocity pulses. The extrapolation of this information to other areas must be done with the understanding that it applies only when ground motion with similar characteristics is expected.

1.2 Overview

Considering the extent and wide range of damage to many different types of buildings in Northridge as well as in the greater Los Angeles area, an initial effort was made to collect, review, and summarize published data on the general performance of buildings compiled by various agencies, organizations and individuals. No new data or detailed building-specific Northridge information was initially collected in this first effort. Rather, existing reports and summaries prepared by others were re-visited and re-classified, specifically characterizing building behavior at different performance levels in light of the performance goals being developed by the Vision 2000 Committee.

Many excellent reports have been published containing both general and specific detailed lessons learned from the Northridge earthquake. They have been summarized, loaded into a data base for easy retrieval, and are reported on in Chapter 2, Preliminary Northridge Lessons. Unfortunately, this approach fell short in achieving the Vision 2000 objective of gathering data that could be used to calibrate analysis and design techniques. The primary shortcoming was the inability to relate the described performance of various building types to intensity of ground shaking since conclusions in any single document are commonly drawn from a small sample of anonymous buildings with little specific information.

The focus of the effort was therefore expanded to gather and summarize building-specific performance. It was again found that an insufficient number of completed detailed case histories were available in Northridge reconnaissance and summary reports to allow a meaningful evaluation of the performance of various building types in light of Vision 2000 performance objectives. Most published reports of earthquake damage to specific buildings lack the necessary detail regarding construction of structural and nonstructural systems and extent of damage to the building to make a complete characterization of its performance in terms of the Vision 2000 performance levels; i.e. Near Collapse, Life-Safe, Operational, and Fully Operational.

In light of this conclusion, the emphasis for the Northridge performance calibration effort was again expanded to the collection of the data from first-hand, post-earthquake observations of specific buildings conducted by various consulting engineering offices and public agencies. The primary source of building data was through individual structural engineers and their organizations. The information gathered was summarized on a data collection guideline and entered into a second data base for general review and is available for future use. The data is summarized in Chapter 3, Buildings Damage Observed after the Northridge Earthquake.

1.3 Performance Levels

At the very heart of performance based engineering are the performance levels. The definitions given below define each of these ranges in terms of the maximum damage and lowest performance which would qualify within the intent of the performance level. These definitions are used to catalog the performance of buildings after the Northridge Earthquake studied in Chapter 3 of this report.

A. *Fully Operational*

A performance level in which essentially no damage has occurred. If a building responds to an earthquake within this performance level, the consequences to the building user community are negligible. The building remains safe to occupy and it is expected that post-earthquake damage inspectors utilizing the ATC-20 methodology would post the building with a green placard. The building is occupiable and all equipment and services related to the building's basic occupancy and function are available for use. In general, repair is not required.

B. *Operational*

A performance level in which moderate damage to nonstructural elements and contents, and light damage to structural elements has occurred. The damage is limited and does not compromise the safety of the building for occupancy. Post-earthquake damage inspectors utilizing the ATC-20 methodology would be expected to post the building with a green placard. It would be available for occupancy for its normal intended function, immediately following the earthquake, however, damage to some contents, utilities and nonstructural components may partially disrupt some normal functions. Back-up systems and procedures may be required to permit continued use. Repairs may be instituted at the owners' and tenants' convenience.

C. *Life Safe*

A performance level in which moderate damage to structural and nonstructural elements and contents has occurred. The structure's lateral stiffness and ability to resist additional lateral loads has been reduced, possibly to a great extent, however, some margin against collapse remains. No major falling debris hazards have occurred. Egress from the building is not substantially impaired, albeit elevators and similar electrical and mechanical devices may not function. In the worst case, post-earthquake damage inspectors, using the ATC-20 methodology, would be expected to post such a building with a yellow placard. In such cases the building would not be available for immediate post-earthquake occupancy. The building would probably be repairable, although it may not be economically practical to do so.

D. *Near Collapse*

An extreme damage state in which the lateral and vertical load resistance of the building have been substantially compromised. Aftershocks could result in partial or total collapse of the structure. Debris hazards may have occurred and egress may be impaired, however, all significant vertical load carrying elements (beams, columns, slabs, etc.) continue to function. In the worst case, post-earthquake damage inspectors, using the ATC-20 methodology, would be expected to post such a building with a red placard. The building will likely be unsafe for occupancy and repair may not be technically or economically feasible.

E. *Partial/Total Collapse*

An extreme damage state in which the vertical load resistance of the building has partially or totally collapsed. Post-earthquake damage inspectors, using the ATC-20 methodology, would be expected to post such a building with a red placard. The building will be unsafe for occupancy and the building probably will need to be demolished.

Table 1-1 through 1-6 further define these performance levels in terms of the various components of the building.

In this context, it is not necessary to identify the seismic hazard or the performance objective. While there is considerable debate, the Northridge Earthquake is most often referred to as a moderate event. In Vision 2000 terms, as defined in Part 1, it was likely somewhere in between an "occasional" and "rare" event.

System Description	Performance Level				
	10 Fully Operational 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3	2 Collapse 1
Overall building damage	Negligible	Light	Moderate	Severe	Complete
Permissible transient drift	< 0.2%+/-	< 0.5%+/-	< 1.5%+/-	< 2.5%+/-	> 2.5%+/-
Permissible permanent drift	Negligible.	Negligible.	< 0.5%+/-	< 2.5%+/-	> 2.5%+/-
Vertical load carrying element damage	Negligible.	Negligible.	Light to moderate, but substantial capacity remains to carry gravity loads.	Moderate to heavy, but elements continue to support gravity loads.	Partial to total loss of gravity load support.
Lateral Load Carrying Element damage	Negligible - generally elastic response; no significant loss of strength or stiffness.	Light - nearly elastic response; original strength and stiffness substantially retained. Minor cracking/yielding of structural elements; repair implemented at convenience.	Moderate - reduced residual strength and stiffness but lateral system remains functional.	Negligible residual strength and stiffness. No story collapse mechanisms but large permanent drifts. Secondary structural elements may completely fail.	Partial or total collapse. Primary elements may require demolition.
Damage to architectural systems	Negligible damage to cladding, glazing, partitions, ceilings, finishes, etc. Isolated elements may require repair at users convenience.	Light to moderate damage to architectural systems. Essential and select protected items undamaged. Hazardous materials contained.	Moderate to severe damage to architectural systems, but large falling hazards not created. Major spills of hazardous materials contained.	Severe damage to architectural systems. Some elements may dislodge and fall.	Highly dangerous falling hazards. Destruction of components.
Egress systems	Not impaired.	No major obstructions in exit corridors. Elevators can be restarted perhaps following minor servicing.	No major obstructions in exit corridors. Elevators may be out of service for an extended period.	Egress may be obstructed.	Egress may be highly or completely obstructed.

Table 1-1a: General Damage Descriptions by Performance Levels and Systems

System Description	Performance Level				
	Fully Operational	Operational	Life Safe	Near Collapse	Collapse
Mechanical/Electrical/ Plumbing/Utility Systems	Functional.	Equipment essential to function and fire/life safety systems operate. Other systems may require repair. Temporary utility service provided as required..	Some equipment dislodged or overturned. Many systems not functional. Piping, conduit ruptured.	Severe damage and permanent disruption of systems.	Partial or total destruction of systems. Permanent disruption of systems.
Damage to contents	Some light damage to contents may occur. Hazardous materials secured and undamaged.	Light to moderate damage. Critical contents and hazardous materials secured.	Moderate to severe damage to contents. Major spills of hazardous materials contained.	Severe damage to contents. Hazardous materials may not be contained.	Partial or total loss of contents.
Repair	Not required.	At owners/tenants convenience.	Possible - building may be closed.	Probably not practical.	Not possible.
Effect on occupancy	No effect.	Continuous occupancy possible.	Short term to indefinite loss of use.	Potential permanent loss of use.	Permanent loss of use.

Table 1-1b: General Damage Descriptions by Performance Levels and Systems

Elements	Type	Performance Level			
		10 Fully Functional 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Concrete frames	Primary	Negligible.	Minor hairline cracking (0.02"); limited yielding possible at a few locations; no crushing (strains below 0.003).	Extensive damage to beams; Spalling of cover and shear cracking (< 1/8") for ductile columns. Minor spalling in non-ductile columns. Joints cracked < 1/8" width.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some non-ductile columns. Severe damage in short columns.
	Secondary	Negligible.	Same as primary.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some non-ductile columns. Severe damage in short columns.	Extensive spalling in columns (possible shortening) and beams. Severe joint damage. Some reinforcing buckled.
Steel moment frames	Primary	Negligible.	Minor local yielding at a few places. No observable fractures. Minor buckling or observable permanent distortion of members.	Hinges form; local buckling of some beam elements; severe joint distortion. Isolated connection failures. A few elements may experience fracture.	Extensive distortion of beams and column panels. Many fractures at connections.
	Secondary	Negligible.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.	Extensive distortion of beams and column panels. Many fractures at connections.	Extensive distortion of beams and column panels. Many fractures at connections.
Braced steel frames	Primary	Negligible.	Minor yielding or buckling of braces. No out-of-plane distortions.	Many braces yield or buckle but do not totally fail; Many connections may fail.	Extensive yielding and buckling of braces. Many braces and their connections may fail.
	Secondary	Negligible.	Same as primary.	Same as primary.	Same as primary.

Table 1-2a: Performance Levels and Permissible Structural Damage - Vertical Elements

Elements	Type	Performance Level			
		10 Fully Functional 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Concrete shear walls	Primary	Negligible.	Minor hairline cracking (0.02") of walls. Coupling beams experience cracking < 1/8" width.	Some boundary element distress including limited bar buckling; Some sliding at joints; damage around openings; some crushing and flexural cracking; Coupling beams - extensive shear and flexural cracks; some crushing, but concrete generally remains in place	Major flexural and shear cracks and voids; sliding at joints; extensive crushing and buckling of rebar; failure around openings; severe boundary element damage; Coupling beams shattered, virtually disintegrated
	Secondary	Negligible.	Minor hairline cracking of walls, some evidence of sliding at construction joints, Coupling beams experience cracks < 1/8" width, minor spalling.	Major flexural and shear cracks; sliding at joints; extensive crushing; failure around openings; severe boundary element damage; Coupling beams shattered, virtually disintegrated.	Panels shattered virtually disintegrated.
Unreinforced masonry infill walls	Primary	Negligible.	Minor cracking (< 1/8") of masonry infills and veneers. Minor spalling in veneers at a few corner openings.	Extensive cracking and some crushing but wall remains in place; no falling units; Extensive crushing and spalling of veneers at corners of openings.	Extensive cracking and crushing; portions of face course shed.
	Secondary	Negligible.	Same as primary.	Same as primary.	Extensive cracking and crushing; portions of face course shed.

Table 1-2b: Performance Levels and Permissible Structural Damage - Vertical Elements

Elements	Type	Performance Level			
		10 Fully Functional 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
URM bearing walls	Primary	Negligible.	Minor cracking (< 1/8") of masonry infills and veneers. Minor spalling in veneers at a few corner openings. No observable out-of-plane offsets.	Extensive cracking, noticeable in-plane offsets of masonry and minor out-of-plane offsets.	Extensive cracking. Face course and veneer may peel off. Noticeable in-plane and out-of-plane offsets.
	Secondary	Negligible.	Same as primary.	Same as primary.	Same as primary.
Reinforced masonry walls	Primary	Negligible.	Minor cracking (< 1/8"). No out-of-plane offsets.	Extensive cracking (< 1/4"), distributed throughout wall. Some isolated crushing.	Crushing; extensive cracking; damage around openings and at corners; some fallen units.
	Secondary	Negligible.	Same as primary.	Crushing; extensive cracking; damage around openings and at corners; some fallen units.	Panels shattered virtually disintegrated.
Wood stud walls	Primary	Negligible.	Distributed minor hairline cracking of gypsum and plaster veneers.	Moderate loosening of connections and minor splitting of members.	Connections loose, nails partially withdrawn, some splitting of members and panel; veneers shear off.
	Secondary	Negligible.	Same as primary.	Connections loose, nails partially withdrawn, some splitting of members and panel.	Sheathing shears off, let-in braces fracture and buckle, framing split and fractured.
Foundations	General	Negligible.	Minor settlement and negligible tilting.	Total settlements < 6 inches and differential settlements < 1/2 inch in 30 feet.	Major settlements, and tilting.

Table 1-2c: Performance Levels and Permissible Structural Damage - Vertical Elements

System	Performance Levels			
	10 Fully Functional 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Metal deck diaphragms	Negligible.	Connections between deck units and from deck to framing intact. Minor distortions.	Some localized failure of welded connections of deck to framing and between panels; minor local buckling of deck.	Large distortion with buckling of some units and tearing of many welds and seam attachments.
Wood diaphragms	Negligible.	No observable loosening, withdrawal of fasteners, or splitting of sheathing or framing.	Some splitting at connections, loosening of sheathing, observable withdrawal of fasteners, splitting of framing and sheathing.	Large permanent distortion with partial withdrawal of nails and splitting of elements.
Concrete diaphragms	Negligible.	Distributed hairline cracking and a few minor cracks (< 1/8") of larger size.	Extensive cracking (< 1/4") and local crushing and spalling.	Extensive crushing and observable offset across many cracks.

Table 1-3: Performance Levels and Permissible Structural Damage - Horizontal Elements

Element	Performance Level			
	10 Fully Operational 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Cladding	Negligible damage.	Connections yield; some cracks or bending in cladding.	Severe distortion in connections; distributed cracking, bending, crushing and spalling of cladding elements; some fracturing of cladding; falling of panels prevented.	Severe damage to connections and cladding; some falling of panels.
Glazing	Generally no damage; isolated cracking possible.	Some broken glass; falling hazards avoided.	Extensive broken glass; some falling hazard.	General shattered glass and distorted frames; widespread falling hazards.
Partitions	Negligible damage; some hairline cracks at openings.	Cracking to about 1/16" at openings; crushing and cracking at corners.	Distributed damage; some severe cracking, crushing and wracking in some areas.	Severe wracking and damage in many areas.
Ceilings	Generally negligible damage; isolated suspended panel dislocations or cracks in hard ceilings.	Minor damage; some suspended ceilings disrupted, panels dropped; minor cracking in hard ceilings.	Extensive damage; dropped suspended ceilings, distributed cracking in hard ceilings.	Most ceilings damaged; most suspended ceilings dropped, severe cracking in hard ceilings.
Light fixtures	Negligible damage; pendant fixtures sway.	Minor damage; some pendant lights broken; falling hazards prevented.	Many broken light fixtures; falling hazards generally avoided in heavier fixtures (> 20 lbs +/-).	Extensive damage; falling hazards occur.
Doors	Negligible damage.	Minor damage.	Distributed damage; some racked and jammed doors.	Distributed damage; many racked and jammed doors.
Elevators	Elevators operational with isolated exceptions.	Elevators generally operational; most can be restarted.	Some elevators out of service.	Many elevators out of service.

Table 1-4: Performance Levels and Permissible Damage - Architectural Elements

Element	Performance Level			
	10 Fully Operational 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Mechanical equipment	Negligible damage, all remain in service.	Minor damage. Some units, not essential to function out-of-service.	Many units non-operational; some slide or overturn.	Most units non-operational; many slide or overturn; some pendant units fall.
Ducts	Negligible damage.	Minor damage, but systems remain in service..	Some ducts rupture; some supports fail, but ducts do not fall.	Most systems out of commission; some ducts fall.
Piping	Negligible damage.	Minor damage. Minor leaking may occur.	Some pipes rupture at connections; many supports fail; few fire sprinkler heads fail.	Many pipes rupture; supports fail; some piping systems collapse.
Fire alarm systems	Functional.	Functional.	Not functional.	Not functional.
Emergency lighting systems	Functional.	Functional.	Functional.	Not functional.
Electrical equipment	Negligible damage.	Minor damage; panels restrained, isolated loss of function in secondary systems.	Moderate damage; panels restrained from overturning; some loss of function and service in primary systems.	Extensive damage and loss of service.

Table 1-5: Performance Levels and Permissible Damage - Mechanical/Electrical/Plumbing Systems

Element	Performance Level			
	10 Fully Operational 9	8 Operational 7	6 Life Safe 5	4 Near Collapse 3
Furniture	Negligible effects.	Minor damage; some sliding and overturning.	Extensive damage from sliding, overturning, leaks, falling debris etc.	Extensive damage from sliding, overturning, leaks, falling debris etc.
Office equipment	Negligible effects.	Minor damage; some sliding and overturning.	Extensive damage from sliding, overturning, leaks, falling debris etc.	Extensive damage from sliding, overturning, leaks, falling debris etc.
Computer systems	Operational.	Minor damage; some sliding and overturning. Mostly functional.	Extensive damage from sliding, overturning, leaks, falling debris etc.	Extensive damage from sliding, overturning, leaks, falling debris etc.
File cabinets	Negligible damage.	Minor damage; some sliding and overturning.	Extensive damage from sliding, overturning, leaks, falling debris etc.	Extensive damage from sliding, overturning, leaks, falling debris etc.
Bookshelves	Negligible damage.	Minor damage; some overturning and spilling.	Extensive damage from leaks, falling debris, overturning etc.	Extensive damage form leaks, falling debris, overturning etc.
Storage racks & cabinets	Negligible damage; overturning restrained.	Moderate damage; overturning restrained; some spilling.	Extensive damage from leaks, falling debris, overturning, spilling etc.	Extensive damage form leaks, falling debris, overturning, spilling etc.
Art works, collections	Minor damage; overturning restrained.	Moderate damage; overturning restrained; some falling.	Extensive damage from leaks, falling debris, overturning, spilling etc.	Extensive damage form leaks, falling debris, overturning, spilling etc.
Hazardous materials	Negligible damage; overturning and spillage restrained.	Negligible damage; overturning and spillage restrained.	Minor damage; overturning and spillage generally restrained.	Severe damage; some hazardous materials released.

Table 1-6: Performance Levels and Permissible Damage - Contents

2. PRELIMINARY NORTHRIDGE LESSONS

A substantial amount of information was collected in the initial effort to obtain data on the performance of buildings in Northridge from reports compiled by others . A brief summary of the methodology and results of the initial effort is presented for future reference.

Summary tables with selected information contained in the Northridge Lessons Database are provided in Appendix A.

2.1 Review Criteria

The Northridge Lessons Database was designed to collect building performance data indirectly, making use of published observations and conclusions by reconnaissance teams visiting the heavily damaged areas. In order to capture information of particular use to the Vision 2000 effort, the following general review categories were established for reviewing the source documents.

<u>Lessons Learned Category:</u>

Lessons were classified according to the following categories:

- Recorded ground motions
- Performance of structures for each performance objective (life-safety, damage control, and immediate occupancy)
- Design criteria
- Construction parameters
- Inspection procedures
- Type of structural system

<u>Building Type Category:</u>

The fifteen common building types from FEMA-178 (FEMA-1992) were adopted with additional categories for non-bearing or retaining walls, manufactured housing, and lessons not specific to a single building type (e.g., ground motion).

<u>Building Systems Category:</u>

Specialty categories to capture information on materials, structural elements, base isolation, etc.

<u>Key Words:</u>

A maximum of 5 key words for each lesson, if applicable, to capture detail for future use.

Northridge Lesson Classification:

Classification with respect to past lessons learned from recent major earthquakes (EERI-1986; NRC-1994).

2.2 Ground Motion Estimates

Ground motion, and in some cases, site conditions at specific buildings were developed along with the general data collection effort. This was done to provide a more uniform characterization of these parameters than might be achieved by individual building reviewers interpreting data, particularly ground motion, from various sources. Ground motion is characterized in terms of peak horizontal ground acceleration (PHGA), epicentral distance, and Modified Mercalli Intensity (MMI).

Northridge PHGA and approximate epicentral distance were obtained from the ground acceleration contour map (see Figure 2-1) developed for the SAC Joint-Venture investigation of welded steel moment-frame buildings. Where free-field or instrumental records are available for a building, the SAC ground acceleration contour maps were used instead as more representative of local ground motions for comparison purposes. Modified Mercalli Intensity was obtained from the preliminary Northridge isoseismal map published by the Joint OES-FEMA Disaster Field Office (OES-1994). Site soil conditions were based on site-specific data, where possible, or published generalized maps of geologic conditions in the Los Angeles region (LACO-1990; USGS-1985).

2.3 Source Documents

Published summary documents reporting on ground motion and the performance of structures in Northridge were collected as sources for extracting preliminary Northridge lessons. The Northridge Lessons database was not completed and not all source documents collected were reviewed. The source documents for the lessons learned listed in the summary tables in Appendix A were extracted from the following documents:

N001	-	Hauksson (1994)
N002	-	EERI (1994)
N003	-	EERC (1994)
N004	-	NIST (1994)
N009	-	Dames & Moore (1994)
N010	-	Jones, L. et al (1994)
N013	-	CDMG (1994)
N030	-	SEAOSC (1994)
N039	-	SEAOSC (1994)
N041	-	SEAOSC (1994)
N053	-	TMS (1994)

2.4 Database and Characterization of Collected Information

Preliminary Northridge lessons have been entered into a computer database using the Microsoft Access data management program, Version 2.2. Separate files have been maintained for data collected from published documents during initial efforts to extract "Lessons Learned" from reports compiled by others. The Northridge Lessons Database contains 521 individual lesson statements from Northridge documents and 109 previous lessons learned from pre-Northridge documents. Summary tables listing the preliminary Northridge and previous earthquake lessons are provided in Appendix A. The summary tables do not include the actual lesson statement, since some are quite lengthy. The available fields in the database are illustrated by a typical example of an output table for a single lesson.

Since many of the lessons found in the various source documents relate to the same topic, it is necessary to combine and consolidate similar lessons into a single "lesson learned" statement on each separate issue. This is facilitated by sorting the database with final editing to achieve a concise statement of each individual issue. This process was illustrated by extracting all lessons for wood-frame residential buildings (W1) from the database. A total of 72 separate lesson statements was listed. These 72 lessons were then categorized into 14 separate issues related to light wood-frame buildings. The resulting summary is provided in the table entitled "Northridge Lessons Summary for W1" in Appendix A.

The complete Northridge Lessons Database is available for the SEAOC Office in Sacramento, California.

Northridge Attenuation Grid

PGA Contours ; Soil Relation

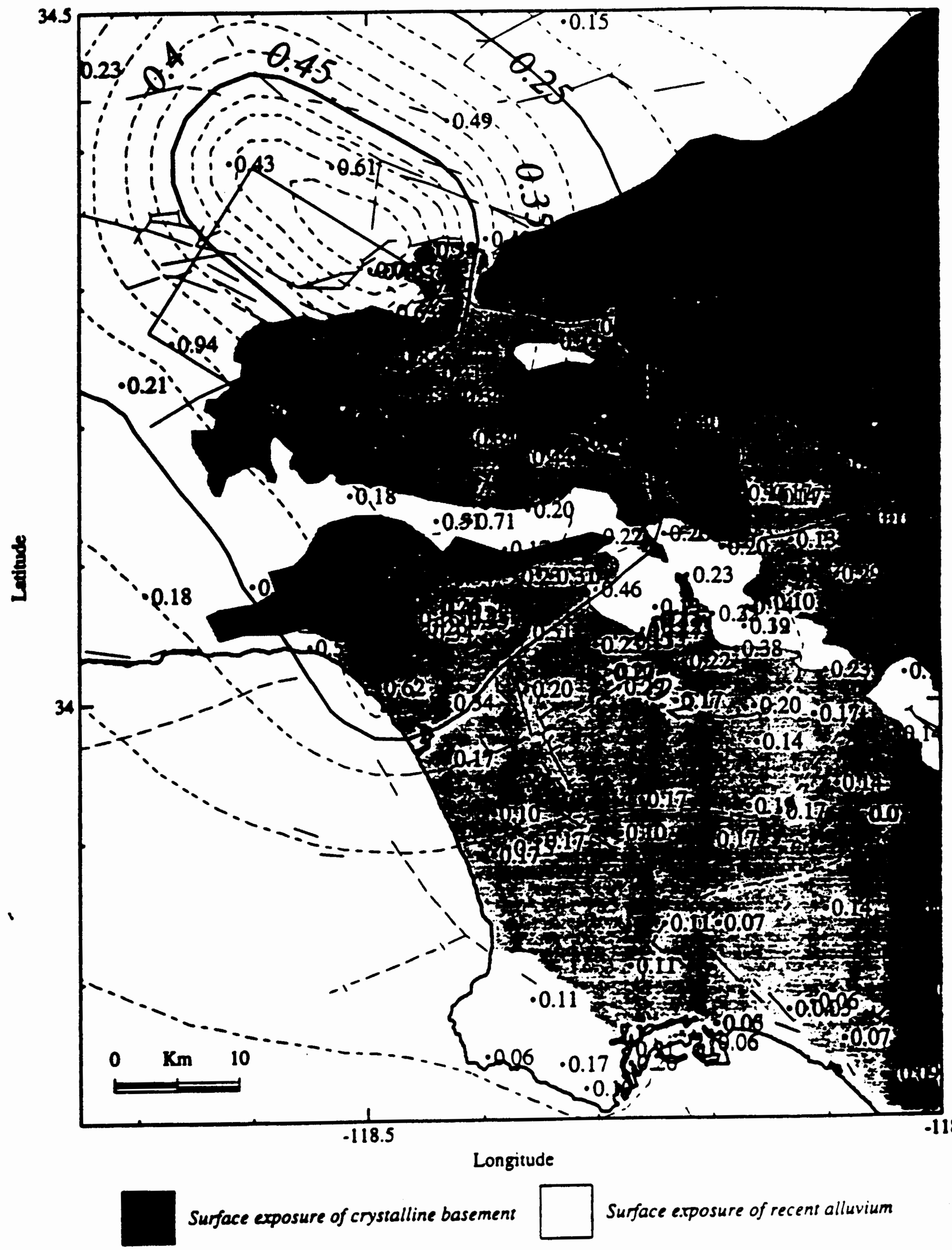

Figure 2-1: Northridge Attenuation Grid

3. BUILDING DAMAGE OBSERVED AFTER THE NORTHRIDGE EARTHQUAKE

After each damaging earthquake, and after the basic reconnaissance work is done and reported, local consulting engineers begin the detailed process of evaluating the extent of the damage and the style of repairs. This work is rarely written up in report form, and is almost never available to the research community. It is not uncommon to find that the extent of damage is quite different than surmised in the reconnaissance reports.

In an attempt to gather actual, credible damage information, members of the Structural Engineers Association of California (SEAOC) were asked to submit detailed information on the damaged buildings that they were working on. Data on 185 buildings were collected within the time constraints of this project. A review of the data will show that it includes all of the "headline" damaged buildings and a fairly representative sample of other damaged and undamaged buildings. It would be very worthwhile to continue the collection of information and continue to build this database. The data collection process, database development, and some preliminary observations on the data and recommendations for future study are presented in the following sections.

3.1 Data Collection

The data collection effort for this phase began with the development of a data collection guideline for use by the contributors. To ensure uniformity and completeness of data collection, a common format was needed for use by contributing individuals. Several existing data-collection forms were considered for this purpose, including the ATC-20 rapid and detailed safety assessment forms (ATC-1989) and the ATC-38 post-earthquake building performance assessment form as adopted by NIST/BFRL (NIST-1994). These and other existing forms considered were found to be inadequate for this project without modification since they were designed either for a very rapid screening process for safety purposes or for a detailed structural performance assessment. A unique form designed for use by this project was developed and offered to the contributing individuals at a planning workshop.

The planning workshop was held for engineers that offered to supply information. The subject of the Workshop was "Evaluation of Northridge Earthquake Building Damage at it Relates to Newly Developed Performance Categories by Vision 2000". The purpose was to engage the invited attendees in focused discussion on the following key issues:

A. Documenting the general behavior of buildings in the Northridge Earthquake in terms of their observed performance.

B. Use of the newly-developed Vision 2000 Committee general damage descriptions by performance levels and systems for rating building performance.

The Workshop was held February 18, 1995 from 9:00 a.m. to 3:00 p.m. at the Renaissance Hotel near Los Angeles International Airport. A total of twenty structural engineers participated, representing twelve different structural engineering consulting firms and six public or private agencies.

Four work groups were formed to test the building documentation and performance rating methodology developed prior to the Workshop. Each group was provided with five or six completed preliminary "Northridge Building Database" forms and were instructed to arrive at a consensus rating on the performance of each building based on the building data provided. General session briefings were held to share comments and recommendations resulting from the group discussions.

The Workshop concluded that some significant revisions were needed in the preliminary data collection form in order to obtain the data required for calibrating the performance of buildings in Northridge against the Vision 2000 building system performance categories. While the building data portion of the work was found to work well, there was a need for:

- A numerical rating scale for performance levels, e.g., a scale of 1 to 10 where 10 is fully functional and the lower range of the scale represents collapse or near collapse.

- An additional performance level for partial or total collapse for building performance below the minimum acceptable performance addressed by Vision 2000 performance objectives.

- Complete damage data for each individual building (some of the test-case buildings in the Workshop had insufficient damage data provided to assign performance categories).

- Adequate information on building location to estimate the severity of ground shaking at the site during the Northridge earthquake.

- Documenting the timeliness of the damage data collected (e.g., date the inspection was done).

- Guidelines and instructions to accompany the data collection form and to aid in interpreting building damage and assigning damage state categories.

The Workshop was a successful and essential element in meeting the objective of the Preliminary Northridge Lessons effort. The findings of the Workshop were incorporated in a revised data collection form which was subsequently found to work well for collecting building data in a uniform format from a number of different contributing offices and agencies. One of the greatest benefits to the project was in enlisting the interest and the participation of the Workshop attendees in providing the needed building and damage data. Complete information on 185 buildings was contributed by 16 different structural engineering offices and public or private agencies.

Subsequent to the workshop, a single page data collection form was developed for this project to capture building performance data specifically for comparison with the Vision 2000 performance objectives. The form was designed to facilitate data entry into the database and to ensure uniformity and consistency of the information. The Northridge Building Database form and accompanying 3-page key to data entry are included in Appendix B. The basic elements of the form include building identifier and location, building and site description, performance rating, and sources of the information provided.

3.2 Database Development

Preliminary Northridge lessons and information collected on specific buildings have been entered into a computer database designed using the Microsoft Access data management program, Version 2.2. The Northridge Building Database provides a record for each individual building which contains all information provided on the data collection form. The primary record identifier is the building name, with arbitrarily-assigned building number also used for data plotting purposes. An example of a typical record from the Northridge building database is attachment shown in Figure 3-1.

The data collection guideline, table of performance levels, definition of terms, and a listing of the data gathered is contained in Appendix B. Building data is listed in order of assigned building number. Fields containing important building data and summary ground motion and performance ratings are contained in two consecutive pages in the table.

The complete database is available from the Structural Engineers Association of California office in Sacramento, California.

3.3 Summary and Observations Related to the Compiled Building Data

A wide range of building types and occupancies is represented in the building database. Significant statistics regarding building type, number of stories, and occupancy are summarized in Figures 3-1 through 3-4. There is a good distribution of information from the various building types and occupancies. This sample does not represent a statistically balanced sample since it was gathered at the discretion of the contributing engineer. It does not represent the percentage distribution of damaged buildings; in fact, it is heavily influenced by buildings with significant damage. It does appear, however, that the sample size of the most common building types is sufficient to draw general conclusions on the performance of the building system categories of interest to Vision 2000.

Within the time constraints of the 1994 Vision 2000 project, only a limited amount of evaluation was possible on this data set. Of the various sorts and screens tried, four seemed to offer the most compelling information. These are illustrated in Figures 3-5 through 3-8. A brief discussion of each figure follows along with suggestions for additional study.

In each of the figures that follow, the general damage descriptions are represented by their numerical (1 to 10) for each category. The categories included all of the elements of the buildings; i.e., structural, nonstructural, systems, and contents, though each figure represents different subsets of categories. When a subset of categories is used, the rating is reported as the average for each building among the categories included.

A. *Structural Average vs. Peak Ground Acceleration*

The "Structural Average" referred to in Figure 3-5 and other figures refers to the average rating for the vertical load carrying elements and lateral load carrying elements. The peak ground acceleration represents the values defined in Section 2.2. The figure also illustrates the difference found related to building age. The numbers next to each symbol indicates the record number in the data base.

The following simple observations have been made for this sort of the data:

1. A surprising number of buildings experienced near, partial or total collapse (ratings of 4 or less) under less-than-severe earthquake ground motion levels. Most were in the 0.3g to 0.5g range with only 3 in the 0.6g to 0.7g range. Many of these buildings were post 1976 construction.

2. A surprising number of pre-1960 buildings had negligible structural damage (ratings of 9 to 10) even with ground motions of up to 0.6g.

3. There is only a slight indication that design to modern codes has had much impact on the performance of the structural system. The distribution of damage to pre-1960 buildings seems to be fairly uniformly distributed over the various peak ground accelerations. The distribution of 1960-1976 and post-1976 buildings has a slight shift to the right.

4. This data may be better understood is it were also sorted with regard to the model building type.

B. *Nonstructural Average vs. Peak Ground Acceleration*

The "Nonstructural Average" reported in Figure 3-6 and other figures refers to the average rating of the architectural, mechanical, electrical, plumbing and contents categories. The peak ground acceleration represents the values defined in Section 2.2. The figure also illustrates the difference found related to building age. The numbers next to each symbol indicates the record number in the data base.

The following simple observations have been made for this sort of the data:

1. There is no correlation with age of the building or design to modern standards. This is surprising given the damage control provisions in the post-1976 codes. This may be due to a general lack of effectiveness and/or a general lack of enforcement.

2. There is a surprising number of very low ratings (0 to 4) in areas with low peak ground accelerations; i.e. those less than 0.4g.

3. There are a number of buildings with an operational level of performance (8 to 10) that also experienced high peak ground accelerations. Surprisingly enough, these were mostly pre-1960 buildings.

4. This data may be better understood is it were also sorted with regard to the model building type and number of stories. It may be that the best performance occurred in low rise buildings with fairly rigid lateral force resisting systems.

C. *Structural Average vs. Nonstructural Average*

All of the terms used in Figure 3-7 are defined for the previous two. The symbols used to represent the data also indicate the level of shaking the building experienced. This sort allows for conclusions to be drawn on how well the performance of the structural system balances with the performance of the nonstructural system.

The diagonal lines in the figure represent a zone of balanced performance. Buildings falling within the lines should be judged to have similar performance in their structural and nonstructural systems.

The following simple observations have been made for this sort of the data:

1. The level of shaking does not stand out as a key parameter in this comparison.

2. Very rarely did the nonstructural system perform better than the structural system.

3. The majority of buildings demonstrated a balanced performance.

D. *Structural Average vs. Occupancy Average*

The structural average used in Figure 3-8 is defined as above for Figure 3-5. The occupancy average represents the average rating in the occupancy category and the egress category. The symbols used to represent the data also indicate the level of shaking the building experienced. The diagonal lines represent the zone of conformance between structural damage and occupancy as general assumed by engineers. Buildings with a rating of 7 or lower were not occupiable immediately after the earthquake.

The following simple observations have been made for this sort of the data:

1. The strong majority of buildings in the data set were not occupiable after the earthquake. There is no strong correlation between the condition of the structural system and the decision to close.

2. The buildings that remained open were mostly in an undamaged state; rating in the 8 to 10 range. No building remained open that was rated less than Life Safe.

3. A surprising number of buildings were not occupied even though they had little or no structural damage (rating 8 to 10).

4. One building was permanently closed even though it had very limited structural damage (rating = 9).

Northridge Building Database

LOCATION

Building Name:	Sherman Oaks Atrium	**Building No.:**	001
Address:	15315 Magnolia Blvd.	**Thomas Guide:**	
		Cross Street:	Sepulveda Blvd.
		City:	Sherman Oaks

BUILDING DATA

Bldg. Type Category:	S1	**Year Built:**	1965	**No. Stories:**	4	**Drawings:**	☒
Occupancy:	OFF	**Des. Code:**	LA City	**Basement:**	3	**Geotechnic:**	☐
X Dimension:	206 ft.	**X Orientation:**	0	**Foundation:**	SF		
Y Dimension:	158 ft.	**Y Orientation:**	90	**Base Isolation:**	☐	**Retrofit:**	☐

Vertical System: Concrete-filled metal deck floors, steel beams and tapered girders, steel columns anchored @ 1st level concrete deck. Subterranean parking levels, reinforced concrete flat plate and reinforced concrete columns.

Lateral System: Concrete diaphragms and welded steel moment-frames in both orthogonal directions (4 exterior moment-resisting frames in the north-south, 2 exterior and 4 interior moment-resisting frames east-west).

Special Features: H-shape plan configuration

SITE

Soil Conditions:	Holocene alluvium			**Structural Fill:**	☐
UBC Soil Profile:	S2	**Depth to Groundwater:**	30	**Hillside Site:**	☐

GROUND MOTION

Instrumented by:	None	**Station ID:**		**PHGA:**	0.38
Distance:	6	**Direction:**	SE	**MMI:**	VIII

PERFORMANCE

Northridge Posting: R

Overall:	4	Damage lateral system, veneer became falling hazard, major penthouse and elevator core damage.
Vertical:	4	Majority of gravity system fully intact except for some bolts at the moment connections.
Lateral:	5	Failed steel frame moment connections and column base anchorages in the N-S direction.
Architectural:	4	Major damage to stucco finishes at the elevator core and exterior brick veneer major falling hazard.
Egress:	6	Veneer falling hazard at rear of building over exit.
MEP Systems:	5	Damage to elevator core and DWP tranformer vault slab.
Contents:	6	
Occupancy:	5	Evacuation and relocation of tenants was necessary due to damage to moment frame connections.

Keywords: 1. welding 2: baseplates 3: veneer 4: 5: 6: 7: 8:

Comments: Moment Frames U.T. inspected. All connections @ 2nd floor in N-S direction failed or rejected, all E-W acceptable. Repaired by June 1994.

Unpublished Source:
Firm: Myers, Nelson, Houghton, Inc.
By: Richard Fallgren
Date: 27-Feb-95

Figure 3-1: Northridge Building Database Form

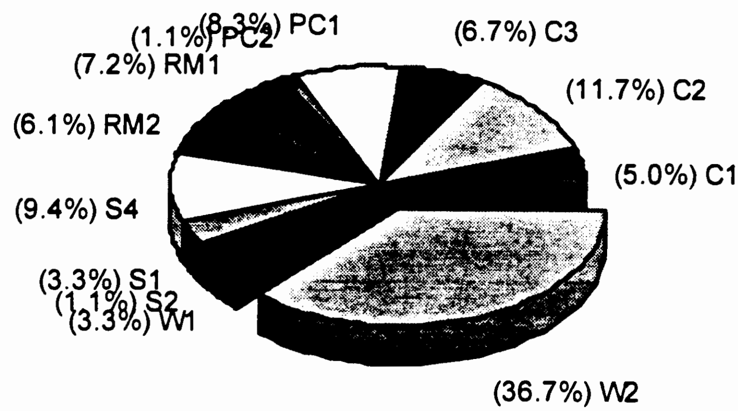

Figure 3-2: Data Distribution by Building Type

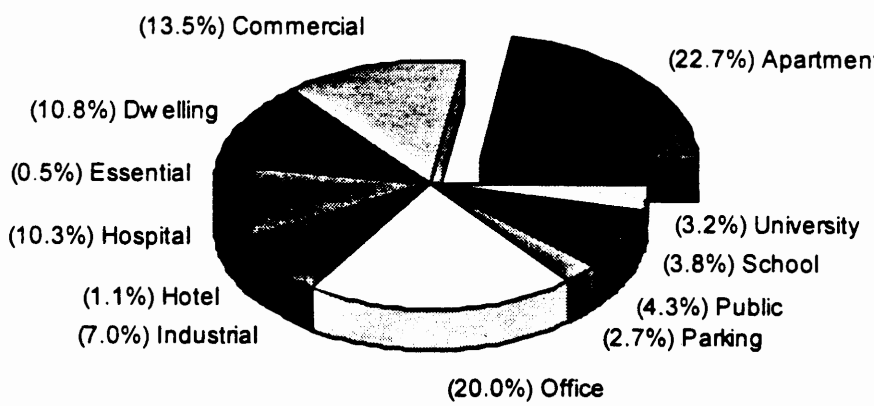

Figure 3-3: Data Distribution by Occupancy Type

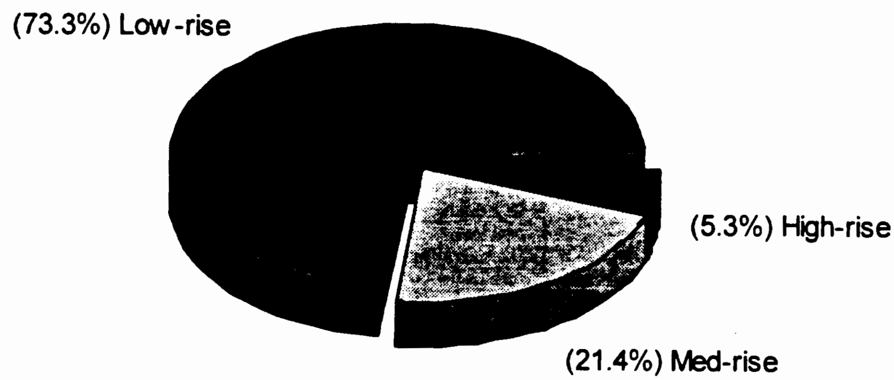

Figure 3-4: Data Distribution by Number of Stories

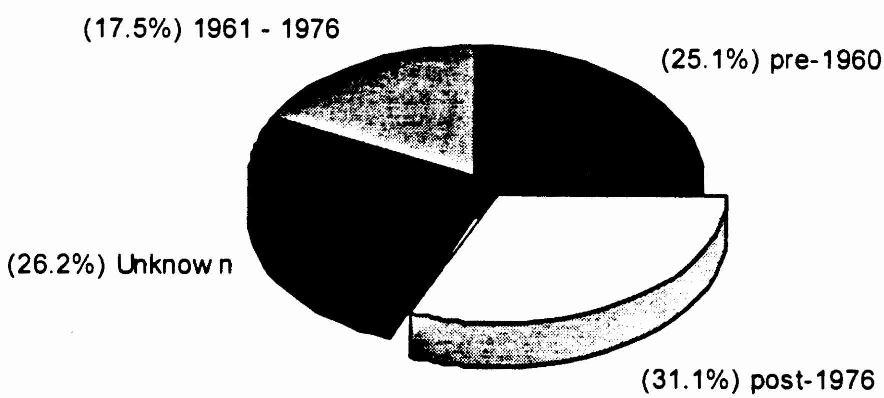

Figure 3-5: Data Distribution by Date of Construction

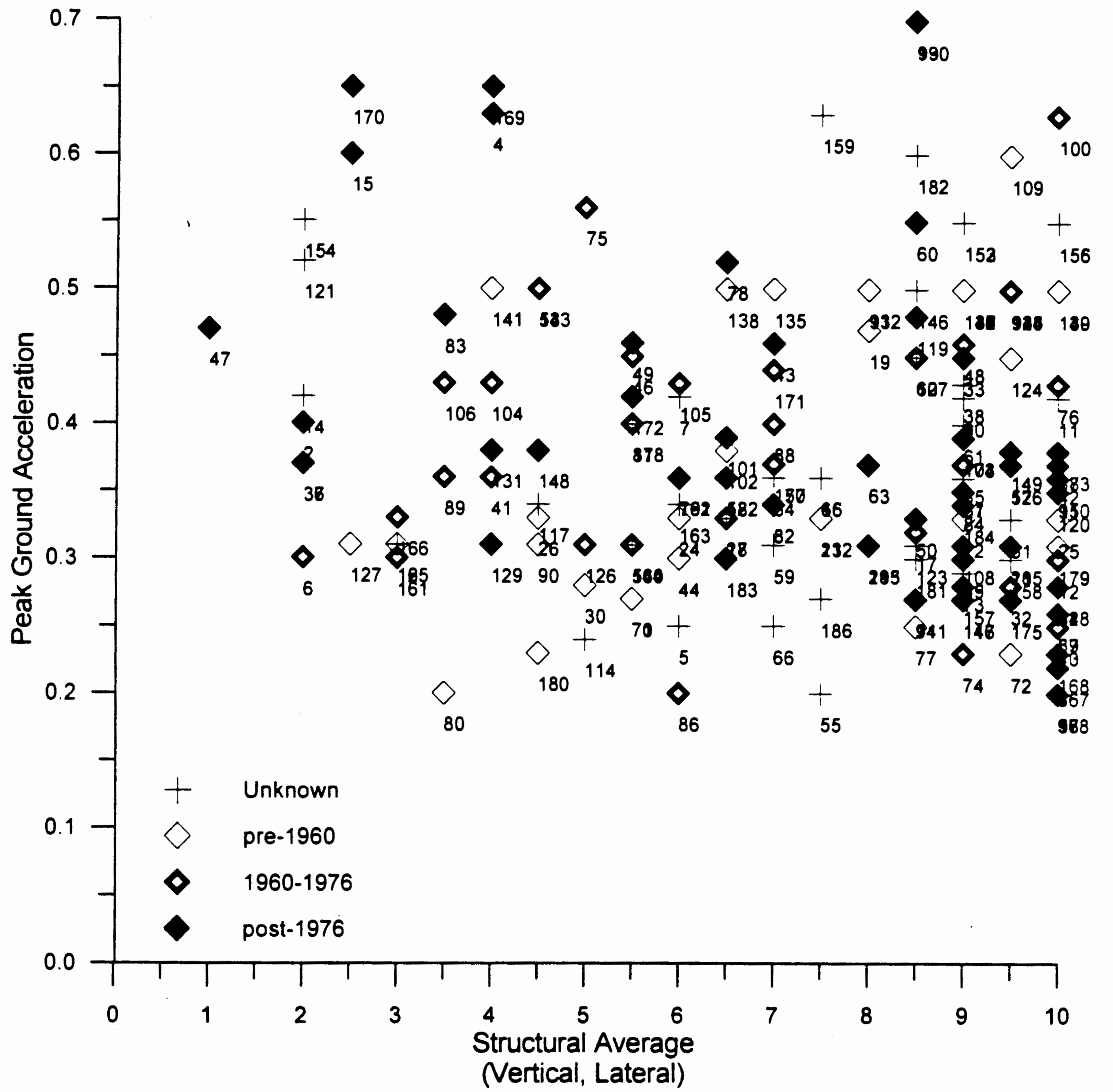

Figure 3-6: Structural Average vs. Peak Ground Acceleration

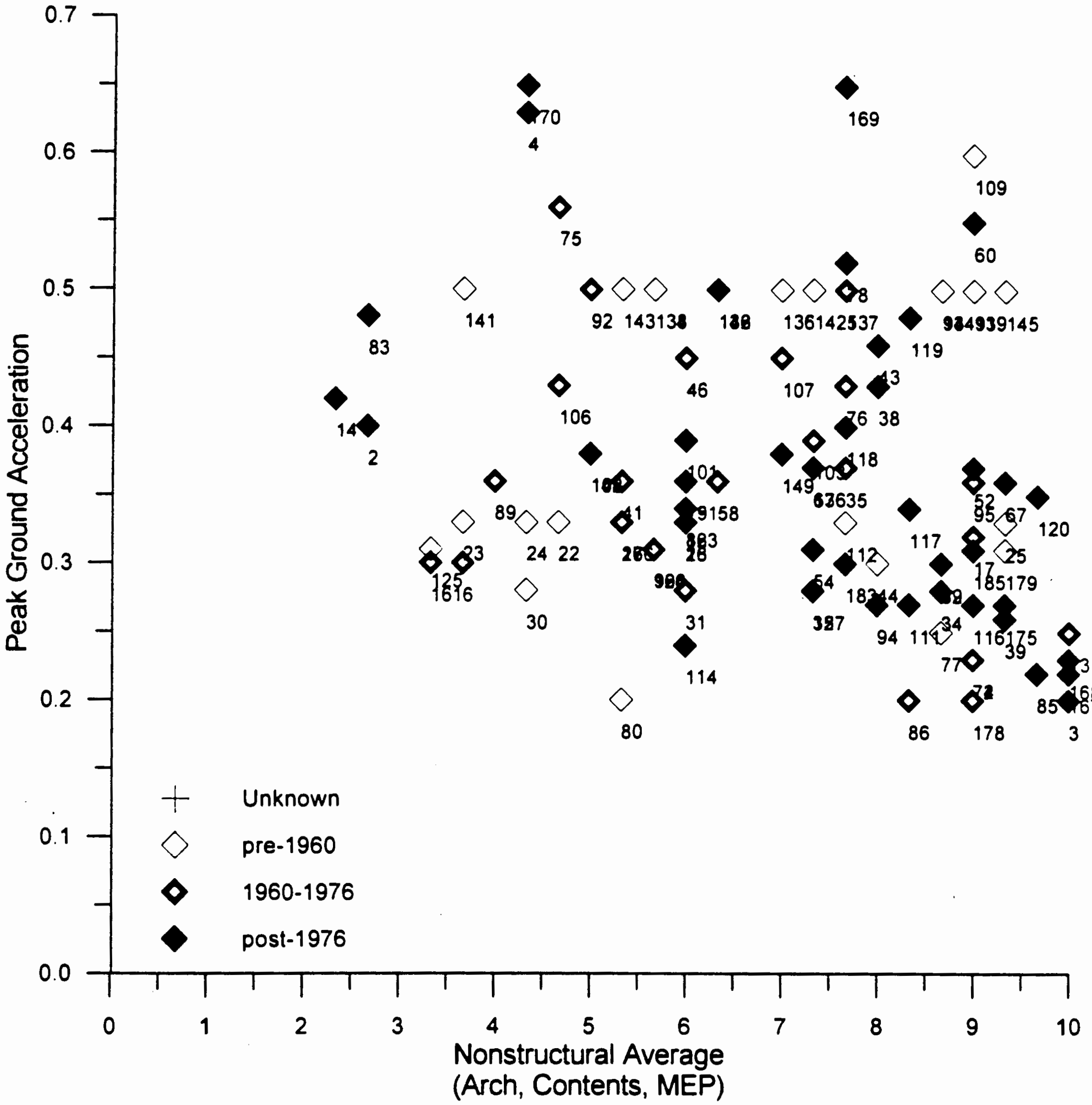

Figure 3-7: Nonstructural Average vs. Peak Ground Acceleration

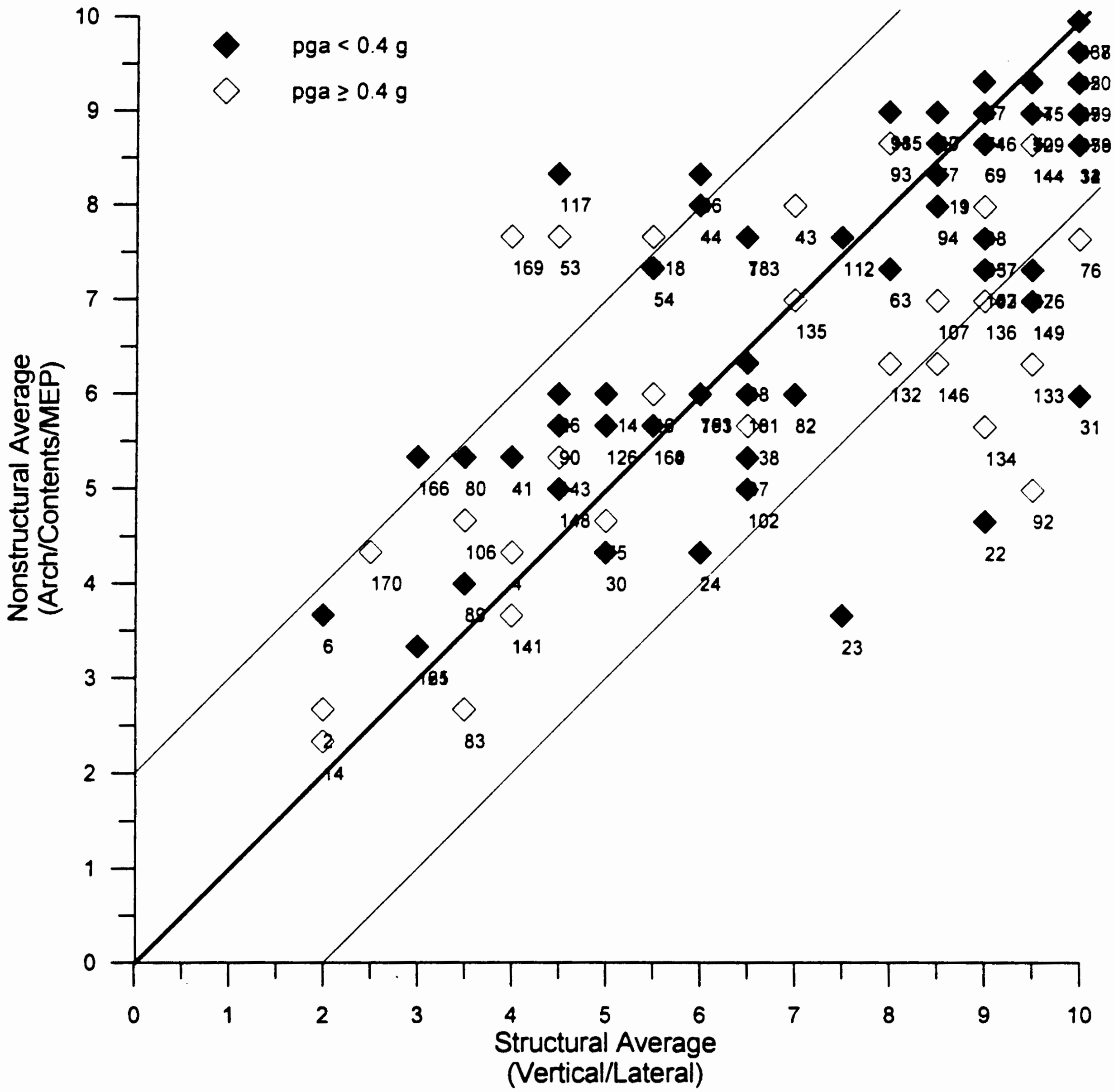

Figure 3-8: Structural Average vs. Nonstructural Average

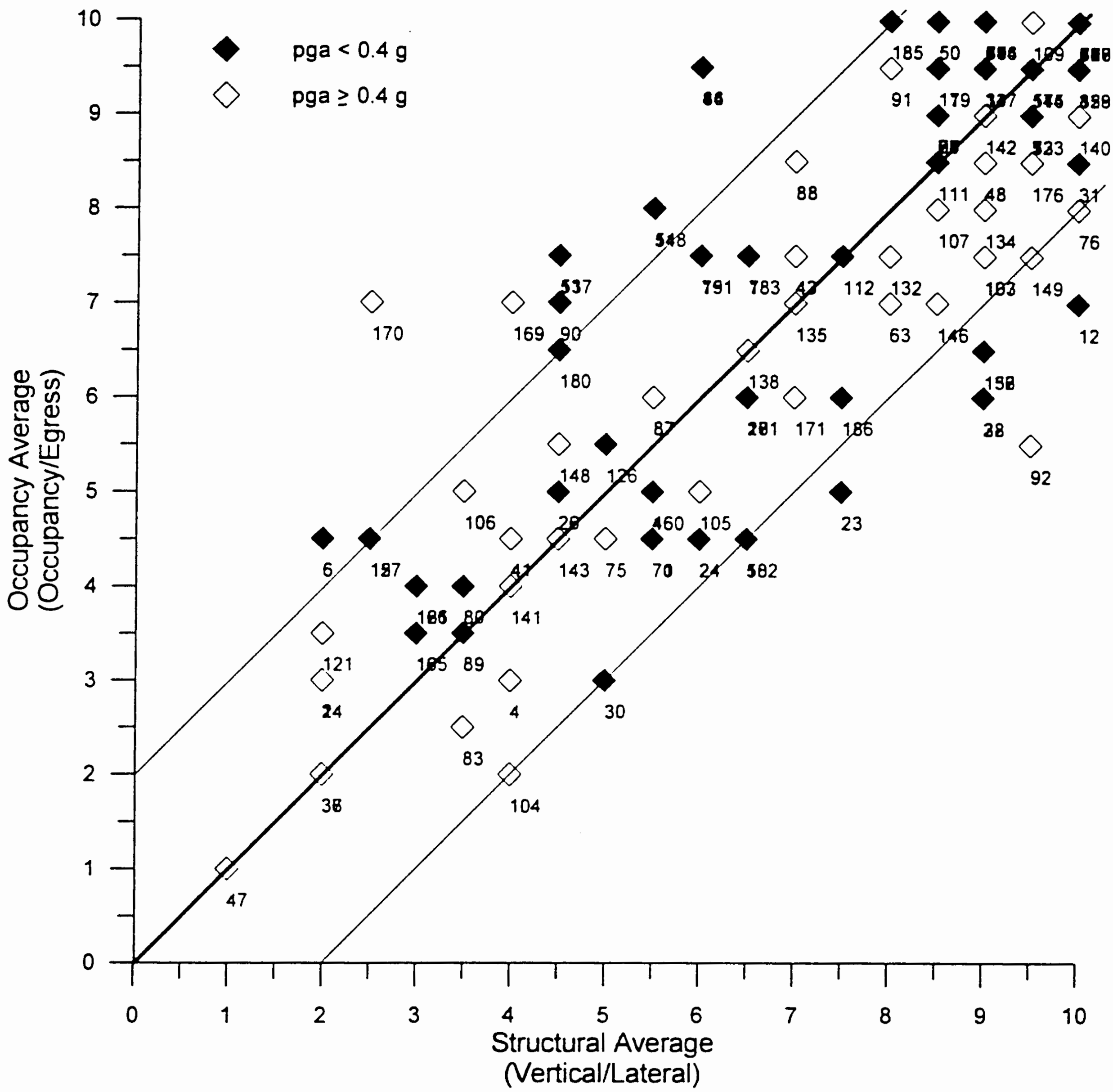

Figure 3-9: Structural Average vs. Occupancy Average

4. RECOMMENDATIONS

The collection, publication, and evaluation of building damage data from the Northridge Earthquake serves as a useful, preliminary methodology for calibrating the Vision 2000 performance objectives to the performance of actual buildings in real earthquakes. The following specific recommendations are made for improving and expanding this methodology for use in developing future uniform design performance objectives for buildings:

1. The population of buildings contained in the Northridge Building Database, while sufficient for the preliminary lessons identified in this project, is of limited size compared to the number of buildings, both damaged and undamaged, located in the area affected by the earthquake. This effort should be continued to expand the size of the Northridge Building Database while damage reports are still available and before inevitable repairs remove all evidence of damage sustained.

2. Future reconnaissance teams observing building damage in future earthquakes should be encouraged to collect and categorize their data in terms of the building system categories and performance levels developed in this project. It is important to document the performance of nonstructural systems, including architectural, equipment, contents, egress and occupancy, as well as that of structural systems. This data has not been collected or presented in a systematic manner in past earthquakes. The use of data collection forms similar to that developed for this project is recommended.

3. As discussed in Section 2.1, simulated Northridge earthquake ground motion was used to calibrate the performance of buildings in this project. The relationship of this simulated ground motion to actual building site ground motions, and to instrument-measured ground motions, is not fully understood. Additional research on the issue of actual ground motions at building sites in the Northridge earthquake is recommended.

4. The Workshop concluded that the summary performance level descriptions provided were inadequate in some cases to fully characterize damage to the various building systems. More specific and detailed descriptions of each performance level for each building system are needed to improve the reliability of building performance ratings.

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APPENDIX A

PRELIMINARY NORTHRIDGE LESSONS

Lessons

Lesson ID:	N009-045 p.019
Lessons Learned Category:	PLS
Northridge Lessons Learned Classifications:	A
Building Type Category:	Building Type 1: PC1
	Type 2:
	Type 3:
Building Systems Category:	RES
Performance Objective Met?:	No
Lesson:	Many concrete tilt-up buildings in the affected area were severly damaged or experienced collapse in this event.

Keywords:

No.1	collapse	No.4	
No.2		No.5	
No.3		No.6	

Northridge Lessons Summary for "W1"

19-Jan-95

Lesson Summary Number: 1

Lesson Class: C

Performance Objective Met?: yes

Lesson Learned PLS

Building Type Category: W1

Building System Category: SW

Lesson: The modern trend toward fewer, heavier shear walls in wood-frame constuction created large overturning forces and caused base anchors to fail.

Keywords:

overturning

anchors

Applicable Northridge Lessons:

N001-035

Applicable Previous Lessons:

Lesson Summary Number: 2

Lesson Class: A

Performance Objective Met?: NO

Lesson Learned PDAM

Building Type Category: W1

Building System Category: FDN

Lesson: Single family houses and apartment buildings shifted from their foundations.

Keywords:

shift on foundation

Applicable Northridge Lessons:

N004-028

N004-042

N009-049

N002-063

Applicable Previous Lessons:

P002-023

P002-112

Lesson Summary Number:	3	**Lesson Class:**	A	**Performance Objective Met?:**	NO
Lesson Learned	INSP	**Building Type Category:**	W1	**Building System Category:**	QUAL

Lesson: Damaged wooden buildings revealed that many were not constructed according to approved plans, which suggests that a lack of proper inspection was a major reason for much damage.

Keywords:

Applicable Northridge Lessons:

N010-036

Applicable Previous Lessons:

P001-039

P002-021

Lesson Summary Number:	4	**Lesson Class:**	A	**Performance Objective Met?:**	NO
Lesson Learned	PDAM	**Building Type Category:**	W1	**Building System Category:**	CONF

Lesson: Buildings on raised cripple wall perimeter foundations were severely damaged when cripple walls displaced laterally.

Keywords:

Applicable Northridge Lessons:

N002-058

Applicable Previous Lessons:

P001-028 P002-112

P002-023

Lesson Summary Number: 5 **Lesson Class:** A **Performance Objective Met?:** NO

Lesson Learned PDAM **Building Type Category:** W1 **Building System Category:** NSTR

Lesson: An extensive amount of masonry was lost on wood frame buildings where adhered systems or corrugated bent straps were used to anchor veneer.

Keywords:

masory veneer

Applicable Northridge Lessons:

N002-064

Applicable Previous Lessons:

P001-069

Lesson Summary Number: 6	**Lesson Class:** A	**Performance Objective Met?:** NO
Lesson Learned INSP	**Building Type Category:** W1	**Building System Category:** NSTR

Lesson: The most common and easily visible damage to single family homes was damaged and collapsed masonry chimneys, which points out the need for better inspection of code requirements for chimney reinforcement and strap ties.

Keywords:

codes

chimneys

anchorage

collapse

Applicable Northridge Lessons:

N004-027

N053-068

N002-057

N053-076

N009-049

N053-043

Applicable Previous Lessons:

P002-112

Lesson Summary Number:	7	**Lesson Class:**	A	**Performance Objective Met?:**	NO
Lesson Learned	INSP	**Building Type Category:**	W1	**Building System Category:**	NSTR

Lesson: The failures observed point to the need for more and better inspection of the structural system for masonry chimneys. In particular, inspection should address placement and lap splicing of reinforcement, installation of wall straps and anchors to the building frame, and the grouting process.

Keywords:

chimneys

anchorage

codes

Applicable Northridge Lessons:

N053-068

N053-043

N053-076

Applicable Previous Lessons:

P002-112

Lesson Summary Number: 8

Lesson Class: C

Performance Objective Met?: NO

Lesson Learned PLS

Building Type Category: W1

Building System Category: CONF

Lesson: Vertical system structural members that are not part of the lateral system failed due to excessive lateral deflection.

Keywords:

collapse

Applicable Northridge Lessons:

N004-022

N004-021

Applicable Previous Lessons:

P001-011

P001-036

Lesson Summary Number:	9	**Lesson Class:**	A	**Performance Objective Met?:**	NO
Lesson Learned	PLS	**Building Type Category:**	W1	**Building System Category:**	RES

Lesson: While collapse was infrequent, damage to multi-family dwellings contributed significantly to the over 25,000 dwelling units that became unhabitable. Special attention should be paid to improving the seismic performance of existing dwelling stock.

Keywords:

collapse

Applicable Northridge Lessons:

N004-075	N009-005
N004-017	N009-048
N002-057	N009-049
N002-012	N009-050

Applicable Previous Lessons:

P002-055	P002-004	P002-105
P002-078	P002-112	

Lesson Summary Number: 10 **Lesson Class:** A **Performance Objective Met?:** NO

Lesson Learned PLS **Building Type Category:** W1 **Building System Category:** CONF

Lesson: Multi-story (mostly 2 or 3 stories) apartment and condominium buildings performed poorly compared to single family residences. Both the older non-enginnered buildings and many newer buildings, especially those with first level tuck-under parking, suffered extensive damage. In many cases the first story partially or completely collapsed.

Keywords:

soft story	gypboard
collapse	shear wall
stucco	

Applicable Northridge Lessons:

N002-013	N004-018
N002-060	N010-033
N002-061	N010-051
N002-062	N009-051

Applicable Previous Lessons:

P001-028	P001-052	P002-077	P003-004
P001-015	P002-061	P002-112	

Lesson Summary Number: 11 **Lesson Class:** C **Performance Objective Met?:** NO

Lesson Learned DES **Building Type Category:** W1 **Building System Category:** FDN

Lesson: Development of a prescriptive foundation reinforcement policy for single family dwellings, for sites where liquefaction hazards at shallow depths(say less than 20') is clearly identified is recommended.

Keywords:

liquefaction	slab-on-grade
reinforcing	codes
foundation	ground deformation

Applicable Northridge Lessons:

N010-064	N041-022
N041-011	N041-003
N041-020	
N041-021	

Applicable Previous Lessons:

P002-077

Lesson Summary Number: 12 | **Lesson Class:** A | **Performance Objective Met?:** NO

Lesson Learned DES | **Building Type Category:** W1 | **Building System Category:** FDN

Lesson: As earthquake induced differential settlements can lead to costly repairs to structures(slab grout jacking and crack repair), measures to reduce its magnitude are clearly desirable. This could be accomplished by an increased compaction specification and a uniform fill thickness over the building pad zone.

Keywords:

structural fill	transition zone
codes	settlement
ground deformation	hillside dwelling

Applicable Northridge Lessons:

N039-016	N041-004	N041-008
N041-001	N041-005	N041-009
N041-002	N041-006	N041-010
N041-003	N041-007	N041-012

Applicable Previous Lessons:

Lesson Summary Number:	13	**Lesson Class:**	C	**Performance Objective Met?:**	NO
Lesson Learned	PLS	**Building Type Category:**	W1	**Building System Category:**	FDN

Lesson: Even though the number of hillside dwelling structural collapses or separations from the foundation was not large compared to the total number of buildings, they are still a matter of grave concern in terms of public safety and violate SEAOC's criteria that buildings shall not collapse during an earthquake. Damage of a lesser amount was still often severe enough to force evacuation of dwellings. Techniques to minimize this type of damage must be developed and implemented in order to protect life, health and property.

Keywords:

hillside dwelling	codes
foundation	stilt foundation
collapse	stepped foundation

Applicable Northridge Lessons:

N004-026	N039-004	N039-008	N039-012	N039-017
N039-001	N039-005	N039-009	N039-013	N039-018
N039-002	N039-006	N039-010	N039-014	
N039-003	N039-007	N039-011	N039-015	

Applicable Previous Lessons:

P002-112

Lesson Summary Number: 14

Lesson Class: A

Performance Objective Met?: NO

Lesson Learned DES

Building Type Category: W1

Building System Category: SW

Lesson: Primary seismic resistance provided by gypboard(or lath and plaster) on interior partition and stucco on exterior walls proved to be unreliable for large earthquake forces.

Keywords:

materials

gypboard

stucco

Applicable Northridge Lessons:

Applicable Previous Lessons:

Preliminary Northridge Lessons

27-Mar-95

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
1	N000-000	DES	RM1	NSTR	A					
2	N001-001	GM	GEB	FDN	A	soil	water table	damage		
3	N001-002	GM	GEB	FDN	A	faults	seismicity			
4	N001-003	GM	GEB	FDN	A	epicenter				
5	N001-004	DES	GEB	QUAL	A					
6	N001-005	GM	GEB	GEOL	C	edge effect	duration	site effects		
7	N001-006	GM	GEB	GEOL	C	seismicity				
8	N001-007	GM	GEB	GEOL	C	faults	seismicity			
9	N002-001	GM	GEB	GEOL	A	geology	shattered ridges			
10	N002-002	GM	GEB	GEOL	B	ground acceleration				
11	N002-003	GM	GEB	GEOL	A	ground acceleration	ground velocity			
12	N002-004	GM	GEB	FDN	A	liquefaction				
13	N002-005	GM	GEB	FDN	A	liquefaction				
14	N002-006	GM	GEB	FDN	C	site effect				
15	N002-007	PDAM	GEB	GEOL	A	landslide				
16	N002-008	GM	GEB	FDN	A	slope stability	structural fill			
17	N002-009	GM	GEB	FDN	A	liquefaction				
18	N002-010	GM	GEB	FDN	A	ground deformation	settlement	structural fill		
19	N002-011	GM	GEB	FDN	A	site effects	soft soils			
20	N002-012	PLS	W1	RES	C					

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
21	N002-013	PLS	W1	CONF	A	soft story				
22	N002-014	DES	GEB	QUAL	A	codes				
23	N002-015	GM	GEB	RES	A					
24	N002-016	GM	GEB	RES	A	frequency content	duration	ground acceleration		
25	N002-017	DES	GEB	RES	A	response spectra	ground acceleration	seismic demand		
26	N002-018	PLS	GEB	RES	C	collapse	codes	parking garage		
27	N002-019	PLS	GEB	QUAL	B	collapse	codes			
28	N002-020	PDAM	S1	MRF	D	braced frame	diagonal bracing	connections	welding	
29	N002-021	DES	GEB	NSTR	C	concrete	deformation compati			
30	N002-022	DES	GEB	CONF	C	connections				
31	N002-023	DES	GEB	MATL	C	design spectra	stucco	ground deformation		
32	N002-024	DES	S1	MATL	D	connections	ductility			
33	N002-025	DES	GEB	QUAL	C	ground acceleration	vertical acceleratio			
34	N002-026	INSP	GEB	QUAL	A	design	construction			
35	N002-027	PLS	S1	RES	A	collapse				
36	N002-028	PDAM	S1	MRF	D	plumbness				
37	N002-029	PDAM	S2	BF	C	tube section	chevron bracing			
38	N002-030	PDAM	S2	BF	D	base plate	anchor bolt			
39	N002-031	PLS	PC2	RES	C	parking garage	collapse	appendage		
40	N002-032	PLS	PC2	CONN	A	parking garage	collapse			
41	N002-033	PLS	PC2	CONF	C	parking garage	collapse	non-ductile		

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
42	N002-034	PLS	PC2	RES	C	parking garage	collapse	vertical acceleratio		
43	N002-035	PLS	PC2	RES	D	parking garage	collapse	codes		
44	N002-036	PLS	PC2	RES	A	parking garage	collapse			
45	N002-037	PLS	PC2	MRF	C	parking garage	collapse	deformation		
46	N002-038	PDAM	C1	RES	A	non-ductile				
47	N002-039	PLS	C2	SW	A	link beam	short column			
48	N002-040	PDAM	C1	CONF	A	short column	non-ductile			
49	N002-041	PLS	C1	CONF	A	short column	out-of-plumb			
50	N002-042	PLS	C1	CONN	A	non-ductile				
51	N002-043	PDAM	C1	CONF	C	punching shear	deformation	vertical acceleratio		
52	N002-044	PLS	URM	RET	A	collapse				
53	N002-045	PLS	URM	RES	A	parapet	collapse			
54	N002-046	PLS	URM	RET	C					
55	N002-047	PDAM	RM1	RES	A	connections	veneer	codes		
56	N002-048	PDAM	URM	SW	A	chimney	footing			
57	N002-049	PDAM	PC1	RES	A	codes	retrofit			
58	N002-050	PDAM	PC1	RES	A	codes	retrofit	connections	diaphragm	
59	N002-051	PDAM	PC1	NSTR	A	piping	ceilings			
60	N002-052	PDAM	PC1	RET	A	wall anchorage	diaphragm	codes		
61	N002-053	PDAM	PC1	RES	A	codes				
62	N002-054	PDAM	PC1	RES	A	wall anchorage	diaphragm	pilaster		

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
63	N002-055	PLS	PC1	NSTR	A	collapse	pounding	separation		
64	N002-056	PLS	PC1	RES	A					
65	N002-057	PDAM	W1	RES	A	codes	chimney			
66	N002-058	PDAM	W1	RES	A	cripple wall				
67	N002-059	PDAM	W1	SW	A	gypboard				
68	N002-060	PLS	W1	CONF	A	collapse	soft story			
69	N002-061	PLS	W1	CONF	A	soft story	collapse	stucco	shear wall	
70	N002-062	PLS	W1	MATL	A	collapse	soft story	stucco	gypboard	
71	N002-063	PLS	W1	CONF	A	collapse	shear wall	pilaster		
72	N002-064	PDAM	W1	NSTR	A	veneer	anchorage			
73	N002-065	PDAM	W2	HAZ	A					
74	N002-066	PFNC	GEB	BAS	A					
75	N002-067	PFNC	S2	BAS	A					
76	N002-068	PFNC	S2	BAS	A					
77	N002-069	PFNC	S2	BAS	A					
78	N002-070	PFNC	GEB	BAS	A					
79	N002-071	PLS	C2	SW	A	hospitals				
80	N002-072	PLS	C2	MATL	C	hospitals	columns			
81	N002-073	PDAM	GEB	NSTR	A	separation	contents			
82	N002-074	PDAM	S2	QUAL	C	hospitals	penthouse	bracing	codes	
83	N002-075	PLS	GEB	FDN	A	schools	manufactured housin	cripple wall		

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
84	N002-076	PLS	GEB	APP	A	schools	canopies	connections		
85	N002-077	PLS	GEB	HAZ	A	schools	codes	bracing	shear wall	ground deforma
86	N002-078	PLS	GEB	RES	A	schools	codes			
87	N002-079	PLS	C1	MRF	C	collapse	shear failure			
88	N002-080	PDAM	C3	SW	C	connections	reinforced masonry	retrofit		
89	N002-081	PDAM	S1	CONF	C	deformation	anchorage	base plates	connections	
90	N002-082	PLS	PC2	MRF	C	parking garage				
91	N002-083	PDAM	W1	SW	B					
92	N002-084	PFNC	GEB	NSTR	A	contents				
93	N002-085	PDAM	C1	MRF	C					
94	N002-086	PDAM	C1	MATL	C	concrete repair				
95	N002-087	GM	GEB	GEOL	A	codes	instrumentation			
96	N002-088	PLS	S1	CONF	C	connections	rotation			
97	N002-089	PDAM	PC2	CONN	C	parking garage	shear wall			
98	N003-001	PLS	GEB	RES	D					
99	N003-002	PLS	GEB	RES	C					
100	N003-003	PLS	GEB	QUAL	A	codes				
101	N003-004	PFNC	GEB	QUAL	C	codes	repairability			
102	N003-005	GM	GEB	GEOL	C					
103	N003-006	GM	GEB	GEOL	A	attenuation	bedrock	site effects		
104	N003-007	GM	GEB	GEOL	C	site effects	basin effects			

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
105	N003-008	GM	GEB	FDN	C	soft soils	amplification	stiff soils		
106	N003-009	DES	GEB	FDN	C	near-field	codes	design spectra	ground acceleration	ground motion
107	N003-010	DES	GEB	QUAL	C	codes	retrofit	existing building		
108	N003-011	DES	GEB	QUAL	A	site effects	existing building			
109	N003-012	GM	GEB	FDN	A	ground deformation	liquefaction	epicenter		
110	N003-013	GM	GEB	FDN	C	liquefaction	groundwater	ground deformation		
111	N003-014	GM	GEB	FDN	C	microzonation	liquefaction	ground displacement		
112	N003-015	GM	GEB	GEOL	A	landslide				
113	N003-016	DES	GEB	GEOL	C	landslide	codes			
114	N003-017	PDAM	GEB	FDN	C	structural fill	landslide	ground deformation	transition zone	
115	N003-018	PDAM	GEB	RW	A					
116	N003-019	GM	GEB	RES	C	seismic risk				
117	N003-020	GM	GEB	GEOL	A	ground acceleration	rock	soil	site effects	
118	N003-021	GM	GEB	GEOL	A	ground acceleration	vertical acceleratio			
119	N003-022	DES	GEB	RES	C	response spectra	codes	ground motion		
120	N003-023	GM	GEB	FDN	C	site effects	soft soils			
121	N003-024	GM	GEB	FDN	C	amplification	site effects	stiff soils		
122	N003-025	GM	GEB	GEOL	C	site effects	basin effects			
123	N003-026	GM	GEB	FDN	C	site effects	site response			
124	N003-027	GM	GEB	FDN	C	liquefaction	ground deformation			
125	N003-028	GM	GEB	FDN	A	liquefaction	ground deformation	lateral spreading		

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
126	N003-029	PDAM	GEB	FDN	A	liquefaction	ground deformation	latreal spreading		
127	N003-030	GM	GEB	FDN	C	liquefaction	ground deformation			
128	N003-031	GM	GEB	FDN	A	liquefaction				
129	N003-032	GM	GEB	FDN	A	liquefaction	duration			
130	N003-033	GM	GEB	FDN	C					
131	N003-034	GM	GEB	FDN	B	liquefaction	ground compaction	ground deformation		
132	N003-035	GM	GEB	FDN	C	liquefaction	microzonation			
133	N004-001	GM	GEB	RES	C					
134	N004-002	GM	GEB	RES	A	codes	ground acceleration	response spectra		
135	N004-003	GM	GEB	RES	A	codes	vertical acceleratio			
136	N004-004	GM	GEB	FDN	A	landslides				
137	N004-005	GM	GEB	FDN	A	liquefaction				
138	N004-006	GM	GEB	FDN	A	liquefaction	ground deformation	settlement	lateral spreading	
139	N004-007	DES	GEB	RES	A	codes	ground acceleration	response spectra		
140	N004-008	GM	GEB	QUAL	A	codes	ground acceleration			
141	N004-009	DES	GEB	QUAL	A	codes	ductility			
142	N004-010	DES	GEB	RES	C	codes	ground motion			
143	N004-011	DES	GEB	RES	C	codes	epicenter	ground acceleration	ground motion	
144	N004-012	DES	GEB	QUAL	A	codes				
145	N004-013	DES	GEB	QUAL	A	codes				
146	N004-014	GM	GEB	RES	A	codes	ground acceleration	vertical acceleratio		

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
147	N004-015	PLS	GEB	RES	A					
148	N004-016	GM	GEB	RES	C	directivity	epicenter			
149	N004-017	GM	W1	RES	C	collapse				
150	N004-018	PLS	W1	CONF	A	soft story				
151	N004-019	PDAM	W1	FDN	A	low-rise	stucco	lowrise		
152	N004-020	PDAM	W1	RES	A	low-rise	stucco			
153	N004-021	PLS	W1	CONF	C	collapse				
154	N004-022	PLS	W1	CONF	C	collapse				
155	N004-023	PLS	C2	SW	A	link beam				
156	N004-024	DES	C1	MRF	A	short column				
157	N004-025	PDAM	C1	MATL	C	column	repair	epoxy		
158	N004-026	PLS	W1	FDN	B	collapse	slope failure			
159	N004-027	PLS	W1	HAZ	A	collapse	chimneys			
160	N004-028	PDAM	W1	RES	A	falling hazards	shift on foundation			
161	N004-029	PDAM	S3	RES	A	manufactured housi				
162	N004-030	DES	C1	NSTR	B	hospitals	codes			
163	N004-031	PFNC	GEB	NSTR	A	critical facility	equipment support	piping		
164	N004-032	PLS	C1	MRF	A	colum	punching shear	collapse		
165	N004-033	PLS	C1	MRF	A	connections				
166	N004-034	PLS	PC1	RES	A	collapse				
167	N004-035	PLS	URM	RES	B	retrofit				

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
168	N004-036	PLS	URM	RES	A	collapse	falling hazards	retrofit		
169	N004-037	PLS	URM	RES	A	retrofit				
170	N004-038	PLS	GEB	RES	C	office buildings	mid-rise			
171	N004-039	PLS	C1	CONF	A					
172	N004-040	PLS	C1	CONF	A	short column				
173	N004-041	PLS	S1	RES	D	soft story	out of plumb	drift		
174	N004-042	GM	W1	FDN	A	shift on foundation				
175	N004-043	PLS	PC2	RES	A	parking garage	collapse			
176	N004-044	PLS	PC2	CONF	C	parking garage	collapse			
177	N004-045	PLS	C1	CONF	C	parking garage	collapse	post-tensioned		
178	N004-046	PLS	PC2	RET	A	parking garage	retrofit	ramp		
179	N004-047	PLS	PC2	RES	A	parking garage	short column	connections	ramp	collapse
180	N004-048	DES	PC2	CONF	A	parking garage	redundancy	ramp	short column	weak column
181	N004-049	DES	PC2	QUAL	A	parking garage	ductility			
182	N004-050	PLS	PC2	RES	A	parking garage	connections			
183	N004-051	PLS	GEB	RES	A	codes	collapse			
184	N004-052	DES	GEB	RES	A	non-ductile	hillside			
185	N004-053	PLS	S1	MATL	D	connections	parking garage			
186	N004-054	DES	GEB	QUAL	A					
187	N004-055	PFNC	GEB	NSTR	A	hospitals	schools			
188	N004-056	PLS	S3	FDN	A	manufacture housin	anchorage			

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
189	N004-057	PLS	GEB	CONF	A	soft story	collapse			
190	N004-058	DES	GEB	NSTR	A	collapse	drift			
191	N004-059	DES	PC2	CONF	A	parking garage	ramp	short column		
192	N004-060	DES	URM	RET	A					
193	N004-061	PLS	GEB	CONF	A					
194	N004-062	PLS	GEB	CONF	A					
195	N004-063	DES	GEB	CONF	A	codes				
196	N004-064	DES	GEB	QUAL	A	codes				
197	N004-065	PLS	URM	RET	A	non-ductile				
198	N004-066	PLS	GEB	RET	C	collapse				
199	N004-067	PLS	GEB	NSTR	A	fires	natural gas			
200	N004-068	PLS	GEB	NSTR	A	natural gas	water heaters			
201	N004-069	PLS	GEB	NSTR	A	electrical power	fires			
202	N004-070	PLS	GEB	HAZ	A	hazardous materials	fires			
203	N004-071	GM	GEB	RES	C	epicenter				
204	N004-072	DES	GEB	QUAL	C	codes				
205	N004-073	PLS	PC2	RES	A	parking garage				
206	N004-074	PLS	GEB	NSTR	A	fires	natural gas			
207	N004-075	PFNC	W1	RES	A					
208	N004-076	DES	GEB	RES	A	ground acceleration	codes	ground motion		
209	N004-077	DES	PC2	QUAL	C	parking garage	codes			

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
210	N004-078	PLS	URM	RET	A	falling hazards				
211	N004-079	PFNC	GEB	NSTR	A	hospitals	schools	codes		
212	N004-080	PLS	S1	CONF	D					
213	N004-081	GM	GEB	RES	A	codes				
214	N004-082	GM	GEB	RES	C	vertical acceleratio	codes			
215	N004-083	DES	GEB	QUAL	A	codes				
216	N004-084	PLS	GEB	CONF	A					
217	N004-085	DES	GEB	QUAL	C	codes	deformations			
218	N004-086	DES	URM	RET	A					
219	N004-087	DES	GEB	NSTR	A	codes				
220	N004-088	DES	S1	CONN	D	ductility	codes			
221	N004-089	DES	GEB	RET	A					
222	N009-001	GM	GEB	RES	A	duration				
223	N009-002	GM	GEB	QUAL	A	design level				
224	N009-003	PLS	GEB	RES	A	epicenter				
225	N009-004	PLS	GEB	FDN	A	liquefaction				
226	N009-005	PLS	W1	RES	A	collapse				
227	N009-006	DES	W1	SW	A	materials	connections			
228	N009-007	PLS	PC2	QUAL	A	parking garage	ductility	redundancy		
229	N009-008	PLS	S1	RES	D					
230	N009-009	GM	GEB	GEOL	D	faults	thrust			

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
231	N009-010	GM	GEB	GEOL	C	faults	directivity			
232	N009-011	GM	GEB	GEOL	C	faults	surface rupture			
233	N009-012	GM	GEB	GEOL	C	epicenter				
234	N009-013	GM	GEB	GEOL	C	faults	thrust			
235	N009-014	GM	GEB	GEOL	C	geology	faults	economic impact		
236	N009-015	GM	GEB	GEOL	C	ground deformation	faults			
237	N009-016	GM	GEB	GEOL	C	geology	faults			
238	N009-017	GM	GEB	GEOL	A	ground acceleration				
239	N009-018	GM	GEB	GEOL	C	ground acceleration	vertical acceleratio			
240	N009-019	GM	GEB	GEOL	A	geology	site effects			
241	N009-020	GM	GEB	GEOL	A	geology	amplification	ground acceleration	site effects	
242	N009-021	GM	GEB	GEOL	C	ground velocity	ground displacement	duration		
243	N009-022	GM	GEB	GEOL	C					
244	N009-023	PLS	GEB	FDN	A	site effects				
245	N009-024	PLS	GEB	FDN	A	liquefaction				
246	N009-025	GM	GEB	FDN	A	liquefaction				
247	N009-026	GM	GEB	FDN	B	liquefaction				
248	N009-027	GM	GEB	GEOL	A	landslides				
249	N009-028	GM	GEB	FDN	A	amplification	soft soils			
250	N009-029	PDAM	GEB	FDN	A	liquefaction	ground deformation	settlement		
251	N009-030	GM	GEB	GEOL	C	hillside	site effects	ridgeline	residential	

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
252	N009-031	PLS	PC2	RES	C	parking garage				
253	N009-032	PFNC	GEB	NSTR	A	electrical power				
254	N009-033	PFNC	GEB	NSTR	A	mechanical equipme	piping	penthouses		
255	N009-034	GM	GEB	GEOL	A	accelerograph				
256	N009-035	GM	GEB	RES	C	codes				
257	N009-036	DES	GEB	RES	C	codes	construction			
258	N009-037	PLS	URM	RES	A					
259	N009-038	PLS	GEB	RET	A					
260	N009-039	GM	GEB	RES	C	vertical acceleratio				
261	N009-040	GM	GEB	RES	C	vertical acceleratio	collapse			
262	N009-041	GM	GEB	RES	C	collapse				
263	N009-042	DES	GEB	RES	A	damage				
264	N009-043	PLS	URM	RES	B					
265	N009-044	PLS	URM	RET	A					
266	N009-045	PLS	PC1	RES	A	collapse				
267	N009-046	DES	GEB	RET	A					
268	N009-047	DES	PC1	SW	A					
269	N009-048	PDAM	W1	RES	C					
270	N009-049	PDAM	W1	RES	A	chimney	collapse			
271	N009-050	PLS	W1	RES	C					
272	N009-051	PLS	W1	CONF	C	collapse	soft story			

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
273	N009-052	DES	W1	MATL	A	shear wall				
274	N009-053	DES	W1	MATL	C	shear wall	codes			
275	N009-054	PLS	C1	RES	A	ductile reinforcing	collapse			
276	N009-055	PLS	C2	RES	A	materials	columns			
277	N009-056	PLS	PC2	RES	C	collapse	parking garage			
278	N009-057	PLS	PC2	RES	A	parking garage	ductility	redundancy		
279	N009-058	DES	PC2	RES	A	parking garage				
280	N009-059	DES	PC2	CONF	A	parking garage	non-ductile	short column		
281	N009-060	DES	PC2	CONF	C					
282	N009-061	DES	PC2	RES	A	materials	seismic joint	parking garage		
283	N009-062	PDAM	S2	BF	A	parking garage	cross bracing			
284	N009-063	PDAM	S2	CONN	A	connections				
285	N009-064	PDAM	S1	CONN	D	welding	base plate			
286	N009-065	DES	S1	CONN	D					
287	N009-066	PFNC	GEB	NSTR	A	mechanical equipme				
288	N009-067	GM	GEB	NSTR	A	mechanical equipme	contents			
289	N009-068	PFNC	GEB	NSTR	A	ceilings	lights	piping		
290	N009-069	PLS	URM	RES	A					
291	N009-070	DES	GEB	RES	C	vertical acceleratio				
292*	N009-071	PDAM	GEB	NSTR	A					
293	N009-072	DES	GEB	QUAL	C	codes				

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
294	N010-001	GM	GEB	GEOL	C	welding	seismicity			
295	N010-002	GM	GEB	GEOL	D	urban area	financial losses			
296	N010-003	GM	GEB	RES	C	urban area				
297	N010-004	GM	GEB	RES	A					
298	N010-005	GM	GEB	GEOL	D	faults				
299	N010-006	GM	GEB	GEOL	C	faults	hazards			
300	N010-007	GM	GEB	GEOL	C	faults				
301	N010-008	GM	GEB	GEOL	C	duration				
302	N010-009	GM	GEB	GEOL	C	faults	duration			
303	N010-010	GM	GEB	GEOL	A	epicenter	ground deformation			
304	N010-011	GM	GEB	GEOL	C	ground acceleration	accelerograph			
305	N010-012	GM	GEB	GEOL	C	ground acceleration				
306	N010-013	GM	GEB	GEOL	A	faults	vertical acceleratio	ground acceleration		
307	N010-014	GM	GEB	GEOL	A	vertical acceleratio	ground acceleration			
308	N010-015	GM	GEB	GEOL	C	faults	epicenter			
309	N010-016	GM	GEB	GEOL	A	directivity	faults			
310	N010-017	GM	GEB	GEOL	C	directivity	epicenter	ground velocity		
311	N010-018	GM	GEB	FDN	A	soft soils	site effects			
312	N010-019	GM	GEB	GEOL	C	site effects	basin effects	ground acceleration		
313	N010-020	GM	GEB	GEOL	A	fault rupture	ground deformation			
314	N010-021	GM	GEB	GEOL	A	fault rupture	ground deformation	liquefaction	slope failure	

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
315	N010-022	GM	GEB	GEOL	C	ground deformation	faults			
316	N010-023	GM	GEB	FDN	B	liquefaction				
317	N010-024	PLS	GEB	RES	A					
318	N010-025	PLS	URM	RES	D	failure				
319	N010-026	PDAM	GEB	NSTR	A	contents				
320	N010-027	GM	GEB	RES	A	highrise				
321	N010-028	PLS	URM	HAZ	A	collapse				
322	N010-029	PLS	URM	RES	A	retrofit				
323	N010-030	PLS	URM	RES	A	retrofit				
324	N010-031	PLS	C1	RES	A	non-ductile	codes	collapse		
325	N010-032	PLS	PC2	RES	A	parking garage	collapse	connections	design	
326	N010-033	PLS	W1	CONF	A	soft story	collapse			
327	N010-034	PLS	W1	SW	A	materials	stucco	cyclic loading		
328	N010-035	PLS	W1	SW	C	overturning	anchors			
329	N010-036	INSP	W1	QUAL	A					
330	N010-037	PLS	S1	CONN	D	welding				
331	N010-038	DES	S1	MATL	D	ductility	connections	welding		
332	N010-039	PLS	S1	CONN	D	collapse	out-of-plumb			
333	N010-040	PLS	S1	CONN	D	fireproofing				
334	N010-041	PDAM	GEB	NSTR	A	contents	ceilings	glazing	piping	equipment
335	N010-042	PLS	GEB	NSTR	A	design	hospitals	codes		

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
336	N010-043	PLS	GEB	NSTR	A	schools	falling hazards	lights		
337	N010-044	PFNC	GEB	NSTR	A	water damage	piping	schools		
338	N010-045	PFNC	GEB	NSTR	A	emergency power	hospitals			
339	N010-046	PDAM	GEB	BAS	A					
340	N010-047	PLS	GEB	BAS	C	ground motion				
341	N010-048	GM	GEB	GEOL	C	faults				
342	N010-049	GM	GEB	RES	C	codes	ground acceleration			
343	N010-050	GM	GEB	FDN	A	liquefaction	ground deformation	faults		
344	N010-051	PLS	W1	RES	A	soft story				
345	N010-052	PLS	S1	CONN	D	welding	ductility			
346	N010-053	GM	GEB	GEOL	C	hazards	fault-rupture			
347	N010-054	GM	GEB	RES	C	seismicity	hazards			
348	N010-055	GM	GEB	GEOL	C	faults				
349	N010-056	GM	GEB	RES	B	codes	faults			
350	N010-057	GM	GEB	GEOL	D	codes	near-field	duration		
351	N010-058	DES	GEB	RES	A	codes	inspection			
352	N010-059	GM	GEB	GEOL	C	codes	ground acceleration			
353	N010-060	PDAM	GEB	QUAL	A	collapse	codes			
354	N010-061	PFNC	GEB	NSTR	A	contents	piping	ceilings		
355	N010-062	PFNC	GEB	BAS	A					
356	N010-063	DES	GEB	GEOL	A	liquefaction	ground deformation			

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
357	N010-064	GM	W1	FDN	A	ground deformation				
358	N010-065	PLS	GEB	RES	A	repairs	cyclic loading			
359	N013-001	GM	GEB	GEOL	C	ground acceleration				
360	N013-002	GM	GEB	GEOL	A	ground acceleration	attenuation			
361	N013-003	GM	GEB	GEOL	C	thrust falts	ground acceleration			
362	N013-004	GM	GEB	GEOL	A	ground acceleration	vertical acceleratio			
363	N013-005	PLS	GEB	RES	C	drift	stiff building	low-rise		
364	N013-006	PLS	S1	RES	C	drift				
365	N013-007	PLS	C1	MRF	A	column	non-ductile			
366	N013-008	GM	GEB	RES	C	amplification	drift	flexible structure		
367	N013-009	GM	GEB	BAS	A					
368	N013-010	PLS	GEB	RES	C	magnitude				
369	N013-011	GM	GEB	GEOL	C	spectral acceleratio				
370	N013-012	GM	GEB	RES	A	duration				
371	N013-013	GM	GEB	FDN	C	site effects	soil sites			
372	N013-014	GM	GEB	GEOL	C	site effects				
373	N013-015	GM	GEB	GEOL	C	ground acceleration	ground velocity			
374	N013-016	PLS	C1	MRF	A	non-ductile	column	torsion		
375	N013-017	PLS	C1	MRF	A	non-ductile				
376	N013-018	GM	C2	SW	C	shear wall	diaphragm	parapet	vertical acceleratio	rocking
377	N013-019	GM	S2	BAS	A					

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
378	N016-001	DES	URM	RET	C	codes				
379	N016-002	GM	GEB	GEOL	C	urban area				
380	N017-001	GM	GEB	GEOL	C	thrust fault				
381	N017-002	GM	GEB	GEOL	C	epicenter	site effects			
382	N017-003	DES	C1	CONF	A	non-ductile	short column			
383	N017-004	PLS	GEB	RES	A	codes	collapse			
384	N030-008	DES	W1	SW	A	hillside dwellings	stepped foundation	materials	stucco	
385	N039-001	DES	W1	BF	D	hillside dwellings	stilt foundation	drift		
386	N039-002	DES	W1	BF	C	hillside dwellings	stilt foundation	anchors	grade beam	
387	N039-003	DES	W1	BF	C	hillside dwellings	stilt foundation	collapse		
388	N039-004	PDAM	W1	BF	D	hillside dwellings	stilt foundation	collapse	drift	
389	N039-005	DES	W1	BF	C	hillside dwellings	stilt foundation	connections		
390	N039-006	DES	W1	FDN	C	hillside dwellings	stepped foundation	grade beam		
391	N039-007	DES	W1	FDN	C	hillside dwellings	stepped foundation	anchors		
392	N039-009	DES	W1	SW	D	hillside dwellings	stepped foundation			
393	N039-010	INSP	W1	FDN	D	hillside dwellings	anchorage	grade beam		
394	N039-011	PFNC	W1	FDN	A	hillside dwellings				
395	N039-012	DES	W1	FDN	A	hillside dwellings	grade beam			
396	N039-013	DES	W1	MATL	A	hillside dwellings	collapse			
397	N039-014	PDAM	W1	FDN	A	hillside dwellings				
398	N039-015	PDAM	W1	FDN	A	hillside dwellings	grade beam	piles	caissons	

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
399	N039-016	DES	W1	FDN	C	hillside dwellings	structural fill	ridge effect		
400	N039-017	DES	W1	RET	A	codes	hillside dwellings			
401	N039-018	DES	W1	QUAL	D	hillside dwellings	collapse	codes		
402	N041-001	PDAM	W1	FDN	A	structural fill	hillside dwelling	ground acceleration		
403	N041-002	PDAM	W1	FDN	A	structural fill	hillside dwelling	ground deformation	transition zone	
404	N041-003	DES	W1	FDN	C	structural fill	transition zone	hillside dwelling	ground motion	ground accelerat
405	N041-004	DES	W1	FDN	A	structural fill	settlement			
406	N041-005	DES	W1	FDN	A	structural fill	settlement			
407	N041-006	PDAM	W1	FDN	A	structural fill	settlement	transition zone		
408	N041-007	PDAM	W1	FDN	A	structural fill	settlement	ground deformation		
409	N041-008	DES	W1	FDN	A	structural fill	codes			
410	N041-009	DES	W1	FDN	A	structural fill	codes			
411	N041-010	DES	W1	FDN	A	structural fill	transition zone	codes		
412	N041-011	DES	W1	FDN	C	structural fill	slab-on-grade	reinforcing	codes	
413	N041-012	DES	W1	FDN	A	structural fill	settlement	codes		
414	N041-013	PDAM	GEB	GEOL	A	landslides				
415	N041-014	DES	GEB	GEOL	A	slope failure	codes			
416	N041-015	DES	GEB	GEOL	A	slope failure				
417	N041-016	DES	GEB	GEOL	C	slope failure				
418	N041-017	PDAM	GEB	FDN	A	liquefaction				
419	N041-018	PDAM	GEB	FDN	A	liquefaction	slab-on-grade	settlement	ground deformation	lateral spreadin

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
420	N041-019	PDAM	GEB	FDN	A	liquefaction	lateral spreading	settlement		
421	N041-020	DES	W1	FDN	A	liquefaction	codes			
422	N041-021	DES	W1	FDN	A	liquefaction	slab-on-grade	reinforcing	codes	foundation
423	N041-022	DES	W1	FDN	A	liquefaction	reinforcing	foundation		
424	N053-001	PLS	RM1	FDN	C	ground motion				
425	N053-002	PDAM	RM1	FDN	A	intensity	ground motion			
426	N053-003	GM	GEB	GEOL	A	intensity	ground acceleration	vertical acceleration		
427	N053-004	GM	RM1	FDN	A	isite effects				
428	N053-005	PLS	RMW	STW	A	colfoundations	connections			
429	N053-006	PLS	RM1	CONF	A					
430	N053-007	PLS	RM1	CONF	A					
431	N053-008	DES	RM1	MATL	A	stiffness				
432	N053-009	PLS	RM1	APP	A	collapse	connections			
433	N053-010	PDAM	RM1	CONF	A	connections	diaphragm			
434	N053-011	PLS	RM1	SW	A					
435	N053-012	DES	RM1	SW	A	link beams	short pier			
436	N053-013	DES	RM1	SW	B	codes	multi-story			
437	N053-014	DES	RM1	QUAL	A	pilaster	slender walls	shear walls		
438	N053-015	PLS	RM1	SW	A	connections	slender walls			
439	N053-016	CONS	RM1	QUAL	A	slender walls	pilaster			
440	N053-017	DES	C3	QUAL	A	relative stiffness				

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
441	N053-018	DES	RM1	HAZ	C	connections				
442	N053-019	PLS	RM1	CONN	A	anchorage	diaphragm			
443	N053-020	PDAM	RM1	QUAL	C	relative stiffness	connections	diaphragm		
444	N053-021	PLS	URM	CONN	A	anchorage				
445	N053-022	DES	S1	NSTR	A	isolation joints	veneer			
446	N053-023	PLS	GEB	NSTR	A	veneer	anchorage			
447	N053-024	DES	GEB	QUAL	C	veneeer	anchorage			
448	N053-025	DES	GEB	NSTR	C	veneer	anchorage			
449	N053-026	DES	GEB	NSTR	C	veneer	anchorage	nailing		
450	N053-027	DES	GEB	QUAL	A	veneer	codes	nailing		
451	N053-028	DES	GEB	QUAL	C	veneer	anchorage			
452	N053-029	CONS	RM1	NSTR	C	veneer	grouting	anchorage		
453	N053-030	INSP	GEB	NSTR	A	veneer				
454	N053-031	DES	GEB	NSTR	A	veneer	anchorage			
455	N053-032	DES	GEB	NSTR	A	veneer	connections			
456	N053-033	PLS	URM	RES	A	retrofit				
457	N053-034	PLS	URM	RES	A	retrofit				
458	N053-035	PLS	URM	RES	A					
459	N053-036	PLS	URM	MATL	C	collapse	retrofit			
460	N053-037	PLS	URM	RET	A	parapet	collapse			
461	N053-038	PLS	URM	RET	B	parapet	anchorage			

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
462	N053-039	CONS	URM	RET	C	crack repair				
463	N053-040	CONS	URM	QUAL	A	codes				
464	N053-041	CONS	GEB	NSTR	A	chimney				
465	N053-042	DES	GEB	NSTR	A	codes	reinforcement	reinforcement		
466	N053-043	DES	W1	NSTR	A	chimney	anchorage	codes		
467	N053-044	DES	RMW	STW	A	design	construction			
468	N053-045	DES	RMW	STW	A	construction	design			
469	N053-046	DES	RMW	STW	A	foundation	overturning			
470	N053-047	CONS	RMW	STW	C	grouting				
471	N053-048	CONS	RMW	MATL	D	site walls				
472	N053-049	DES	RMW	STW	C	isolation joint				
473	N053-050	PFNC	RM1	QUAL	A	serviceability	codes			
474	N053-051	PDAM	RM1	NSTR	A					
475	N053-052	PDAM	RM1	NSTR	A					
476	N053-053	PDAM	RM1	NSTR	A	glazing	relative movement			
477	N053-054	PDAM	RM1	CONF	A	nonstructural	glazing			
478	N053-055	PDAM	RM1	NSTR	A	equipment	stiff building			
479	N053-056	PLS	MH	FDN	A	piping				
480	N053-057	PLS	RM1	RES	A	essential building				
481	N053-058	DES	RM1	SW	A	codes	ductility	reinforcement	pier	
482	N053-059	DES	RM1	QUAL	A	pier	ductility	reinforcement		

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
483	N053-060	CONS	RM1	MATL	C	codes	ductility	reinforcement		
484	N053-061	DES	RM1	MATL	C	codes	reinforcement	ductility		
485	N053-062	DES	RM1	RES	A	codes				
486	N053-063	CONS	RM1	MATL	C	grouting				
487	N053-064	CONS	RM1	MATL	A	grouting	inspection			
488	N053-065	INSP	RM1	RES	A	codes	epicenter	essential buildings		
489	N053-066	INSP	RM1	RES	A	codes	hospitals			
490	N053-067	PLS	RM1	RES	A	nonstructural	schools			
491	N053-068	INSP	W1	NSTR	A	chimney	anchorage			
492	N053-069	INSP	RMW	STW	A					
493	N053-070	CONS	RM1	QUAL	D					
494	N053-071	DES	RM1	RES	A	codes				
495	N053-072	DES	RM1	NSTR	A	veneer	isolation joint			
496	N053-073	DES	RM1	NSTR	A	codes	veneer	anchorage		
497	N053-074	DES	RM1	NSTR	A	codes	veneer	anchorage		
498	N053-075	DES	URM	RET	A					
499	N053-076	DES	W1	NSTR	A	codes	chimney	anchorage		
500	N053-077	DES	RM1	SW	A	drift				
501	N053-078	DES	RM1	NSTR	A	fire				
502	N053-079	CONS	RM1	QUAL	A	grouting				
503	N053-080	INSP	RM1	QUAL	A					

No	Lesson ID	Category	Type	System	Class.	Keyword 1	Keyword 2	Keyword 3	Keyword 4	Keyword 5
504	N053-081	DES	RM1	SW	A	pier				
505	N053-082	CONS	RM1	NSTR	A	veneer	anchorage			
506	N053-083	DES	URM	RET	A	codes				
507	N053-084	DES	RMW	STW	A	codes				
508	N053-085	DES	RM1	SW	A	reinforcement				
509	N053-086	DES	RM1	SW	A	reinforcement				
510	N053-087	DES	RM1	SW	A	reinforcement				
511	N053-088	CONS	RM1	SW	A	reinforcement				
512	N053-089	CONS	RM1	SW	A	grouting				
513	N053-090	INSP	RMW	STW	A	chimney				
514	N053-091	DES	RM1	SW	A	codes	ductility			
515	N053-092	DES	RM1	SW	B	codes				
516	N053-093	DES	RM1	CONN	C	codes				
517	N053-094	DES	RM1	NSTR	A	codes	veneer	anchorage		
518	N053-095	DES	URM	RET	A	codes				
519	N053-096	DES	RM1	MATL	C	reinforcement				
520	N053-097	DES	GEB	NSTR	A	equipment	contents			
521	N053-098	DES	RM1	NSTR	A	veneer	relative movement			

Previous Earthquake Lessons

27-Mar-95

No	Lesson ID	Category	Type	System	Keyword 1	Keyword 2	Keyword 3	Keyword 4
1	P001-106	PLS	GEB	QUAL	occupancy			
2	P001-107	PFNC	GEB	RES				
3	P001-105	PLS	GEB	QUAL	occupancy			
4	P001-104	DES	GEB	NSTR	contents	flexibility		
5	P001-103	DES	GEB	NSTR	computers			
6	P001-102	DES	GEB	NSTR	elevators			
7	P001-101	DES	GEB	NSTR	gas	fire		
8	P001-100	DES	GEB	NSTR	sprinklers			
9	P001-099	DES	GEB	NSTR	equipment			
10	P001-098	DES	GEB	NSTR	piping			
11	P001-097	DES	GEB	NSTR	piping			
12	P001-096	DES	GEB	NSTR	electrical	emergency power		
13	P001-087	PDAM	GEB	NSTR	lights			
14	P001-108	DES	GEB	QUAL	exits			
15	P001-095	DES	GEB	NSTR	equipment	electrical		
16	P001-094	DES	GEB	NSTR	emergency power			
17	P001-093	DES	GEB	NSTR	electrical	conduit		
18	P001-092	DES	GEB	NSTR	electrical	conduit		
19	P001-091	DES	GEB	NSTR	electrical	equipment		
20	P001-090	DES	GEB	NSTR	emergency power			

No	Lesson ID	Category	Type	System	Keyword 1	Keyword 2	Keyword 3	Keyword 4
21	P001-089	DES	GEB	NSTR	emergency lighting			
22	P001-088	DES	GEB	NSTR	lights			
23	P001-086	DES	GEB	NSTR	piping			
24	P001-085	PDAM	GEB	NSTR	equipment	anchorage		
25	P001-084	DES	GEB	NSTR	HVAC	support		
26	P001-083	DES	GEB	NSTR	HVAC	equipment	anchorage	falling hazards
27	P001-082	PDAM	GEB	NSTR	equipment	suspended	unbraced	piping
28	P001-081	PDAM	GEB	NSTR	equipment			
29	P001-080	PDAM	GEB	NSTR	equipment	isolation mounts	piping	
30	P001-079	PDAM	GEB	NSTR	equipment	piping	anchorage	
31	P001-078	PLS	GEB	NSTR	canopies	collapse	falling hazards	
32	P001-077	PLS	GEB	NSTR	appendages	collapse	falling hazards	
33	P001-076	PLS	GEB	NSTR	apendages	anchorage	collapse	falling hazards
34	P001-075	PLS	GEB	NSTR	stairs	falling hazards		
35	P001-074	PLS	GEB	NSTR	UMR	stair enclosure	exiting	falling hazards
36	P001-073	DES	GEB	NSTR	stairs	diaphragm		
37	P001-072	DES	GEB	NSTR	stairs			
38	P001-071	PDAM	GEB	NSTR	exterior skin system	detailing		
39	P001-070	PDAM	GEB	NSTR	curtain wall	glass infill		
40	P001-069	PLS	GEB	NSTR	brick veneer	collapse	anchorage	falling hazards
41	P001-068	PDAM	GEB	NSTR	masonry infill			

No	Lesson ID	Category	Type	System	Keyword 1	Keyword 2	Keyword 3	Keyword 4
42	P001-067	DES	GEB	NSTR	precast	connections	falling hazards	panels
43	P001-066	DES	GEB	NSTR	suspended ceiling			
44	P001-065	PDAM	GEB	NSTR	suspended ceiling			
45	P001-064	PDAM	GEB	NSTR	suspended ceiling	falling hazards		
46	P001-063	PFNC	GEB	NSTR	controlled movemen	partition	cost benefit	
47	P001-062	PFNC	GEB	NSTR	partition	drywall		
48	P001-061	PFNC	GEB	NSTR	partition	tall building	flexible frame	
49	P001-060	PLS	GEB	NSTR	partition	URM		
50	P001-059	PLS	GEB	CONF				
51	P001-058	PFNC	GEB	QUAL				
52	P001-057	PLS	GEB	ADJ	pounding	separation	collapse	falling hazards
53	P001-056	PLS	GEB	CONF	short column			
54	P001-055	PLS	GEB	CONF	triangular shape	torsion		
55	P002-001	GM	GEB					
56	P001-054	PLS	GEB	SW	discontinuities			
57	P001-053	PLS	GEB	CONF				
58	P001-052	PLS	GEB	CONF	soft story	collapse		
59	P001-051	PFNC	GEB	NSTR	equipment	anchorage		
60	P001-050	PLS	GEB	ADJ				
61	P001-049	GM	GEB	RES	long-span	vertical motion	ground movement	
62	P001-048	PLS	GEB	NSTR	contents	equipment	anchorage	

No	Lesson ID	Category	Type	System	Keyword 1	Keyword 2	Keyword 3	Keyword 4
63	P001-047	PLS	URM	MATL	retrofit			
64	P001-046	DES	GEB	DPHM				
65	P001-045	PDAM	GEB	CONF	reentrant corner			
66	P001-044	DES	GEB	DPHM	continuity ties	connections	wood roof	
67	P001-043	DES	GEB	CONN	pilater	concrete	masonry	confinment
68	P001-042	DES	GEB	CONN	diaphragm	anchorage		
69	P001-041	PLS	GEB	NSTR	contents	anchorage	falling hazards	
70	P001-040	PFNC	GEB	NSTR	contents			
71	P001-039	INSP	GEB	QUAL	construction			
72	P001-038	PLS	GEB	CONN	detailing			
73	P001-037	PLS	GEB	ADJ	separation	pounding		
74	P001-036	DES	GEB	QUAL	nonstructural			
75	P001-035	DES	GEB	CONF	redundancy	collapse		
76	P001-034	DES	GEB	CONF	irregular			
77	P001-033	DES	GEB	QUAL	load path	collapse		
78	P001-032	DES	GEB	QUAL	detailing	configuration		
79	P001-031	DES	GEB	QUAL	codes	ductility	inelastic deformatio	
80	P001-030	PFNC	GEB	NSTR	contents	anchorage	lights	equipment
81	P001-029	PLS	GEB	NSTR	fire	gas	chemicals	electrical
82	P001-028	PLS	W1	MATL	wood	configuration	combined materials	
83	P001-027	PLS	S1	MATL	steel			

No	Lesson ID	Category	Type	System	Keyword 1	Keyword 2	Keyword 3	Keyword 4
84	P001-026	PLS	C1	MATL	detailing			
85	P001-025	DES	PC1	MATL	precast	prestressed	connections	
86	P001-024	PLS	RM1	MATL				
87	P001-023	PLS	URM	MATL				
88	P001-022	PLS	GEB	NSTR	panels	parapet	anchorage	falling hazards
89	P001-021	DES	GEB	MRF	column			
90	P001-020	PLS	GEB	QUAL	collapse	strength	ductility	
91	P001-019	DES	GEB	CONN	weak link			
92	P001-018	DES	GEB	QUAL	ductility	structural elements	connections	
93	P001-017	PLS	GEB	NSTR	elevator	stairway	evacuation	
94	P001-016	PDAM	GEB	ADJ	pounding			
95	P001-015	PLS	GEB	CONF	soft story			
96	P001-014	PLS	GEB	CONF	irregular	design		
97	P001-013	PDAM	GEB	QUAL	stiffness			
98	P001-012	PLS	GEB	DPHM	seismic response			
99	P001-011	PLS	GEB	QUAL	stiff element	seismic response		
100	P001-010	PLS	GEB	QUAL	weak link			
101	P001-009	PLS	GEB	QUAL	ductility	redundancy	collapse	
102	P001-008	PLS	GEB	FDN	ground movement			
103	P001-007	PLS	GEB	FDN	deep foundation	piles	settlement	
104	P001-006	PLS	GEB	FDN	faulting	liquefaction	ground failure	shallow foundation

No	Lesson ID	Category	Type	System	Keyword 1	Keyword 2	Keyword 3	Keyword 4
105	P001-005	PLS	GEB	MATL	repeated earthquake			
106	P001-004	PLS	GEB	FDN	ground movement	ground failure	collapse	
107	P001-003	CONS	GEB	QUAL	quality control	collapse	construction	
108	P001-002	PDAM	GEB	QUAL	design	detailing	construction	
109	P001-001	PDAM	GEB	QUAL	codes			

APPENDIX B

BUILDING DAMAGE OBSERVED AFTER THE NORTHRIDGE EARTHQUAKE

NORTHRIDGE BUILDING DATABASE

LOCATION Building No.: ______

Building Name: ______ Thomas Guide: ______
Address: ______ Cross Street: ______
City: ______

BUILDING DATA

Bldg. Type Category: ______ Year Built: ______ No. Stories: ______ Dwg.: ______
Occupancy: ______ Des. Code: ______ Basement: ______ Geot.: ______
X Dimension: ______ X Orientation: ______ Foundation: ______
Y Dimension: ______ Y Orientation: ______ Base Isol.: ______ Retrofit: ______
Vertical System: ______

______

Lateral System: ______

______

Special Features: ______

______

SITE

Soil Conditions: ______ Structural Fill: ______
UBC Soil Profile: ______ Depth to Groundwater: ______ Hillside Site: ______

GROUND MOTION

Instrumented by: ______ Station ID: ______

PERFORMANCE Posting: ______

Overall: ______ ______
Vertical: ______ ______
Lateral: ______ ______
Architectural: ______ ______
Egress: ______ ______
MEP Systems: ______ ______
Contents: ______ ______
Occupancy: ______ ______

Keywords: 1. ______ 2. ______ 3. ______ 4. ______
5. ______ 6. ______ 7. ______ 8. ______

Comments: ______

______

Ref. ID: Page: Ref. ID: Page: Ref. ID: Page: Unpublished Source:
______ ______ ______ ______ ______ ______ Firm: ______
______ ______ ______ ______ ______ ______ By: ______
______ ______ ______ ______ ______ ______ Date: ______

GENERAL DAMAGE DESCRIPTIONS BY PERFORMANCE LEVELS AND SYSTEMS

	Performance Level				
	10	9 8	7 6 5	4 3	2 1
System Description	**Fully Functional**	**Operational**	**Life Safety**	**Near Collapse**	**Partial/Total Collapse**
Overall Building	Negligible	Light	Moderate	Severe	Catastrophic
Vertical Load Carrying Elements	Negligible	Negligible	Light to moderate, but substantial capacity remains to carry gravity demands.	Moderate to heavy, but elements continue to support gravity loads.	Partial or total loss of gravity load support.
Lateral Load Carrying Elements	Negligible - generally elastic response; no significant loss of strength or stiffness; transient drift below damage level (<0.2%) no permanent drift.	Light - nearly elastic response; original strength and stiffness substantially retained. Minor cracking/yielding of structural elements; repair implemented at convenience; transient drift <0.5%, negligible permanent drift.	Moderate - reduced residual strength and stiffness, but lateral systems remains functional. Some permanent drift. Transient drift <1.5%	Negligible residual strength and stiffness. No story collapse mechanisms, but large permanent drifts. Secondary structural elements may completely fail. Transient drift <2.5%.	Partial or total collapse. Primary elements may require demolition.
Architectural Systems	Negligible damage to cladding, glazing, partitions, ceilings, finishes, etc. Isolated elements may require repair at users convenience.	Light to moderate damage to architectural systems. Select protected items undamaged. Hazardous materials contained.	Moderate to severe damage to architectural systems, but large falling hazards not created. Hazardous materials contained.	Severe damage to architectural systems. Some elements may dislodge and fall.	Highly dangerous falling hazards. Destruction of components.
Egress System	Not impaired.	No major obstruction in exit corridors. Elevators can be restarted following minor servicing.	No major obstructions in exit corridors. Elevators may be out-of-service.	Egress may be obstructed.	Highly obstructed or impossible.
Mechanical/Electrical Plumbing Systems	Functional. If utility is disrupted, local emergency service is available.	Equipment remains secured, but may require repair. Fire/life safety systems functional. Utility service may not be available.	Some equipment dislodged or overturned. Systems not functional. Piping, conduit ruptured.	Severe damage and disruption of systems.	Partial or total destruction of systems.
Contents	Some light to moderate damage. Critical contents and hazardous materials secured.	Light to moderate damage. Critical contents and hazardous materials secured.	Moderate to severe damage to contents. Hazardous materials contained.	Severe damage to contents. Hazardous materials may not be contained.	Partial or total loss of contents.
Occupancy	Continuous.	Immediate.	Indefinite loss of use.	Potential permanent loss of use.	Permanent loss of use.

LOCATION

Building Name: Building name, if applicable and known.
Building No.: Left blank for future use.
Address: Provide street and actual or approximate address.
City: City or community.
Cross Street: Nearest major street intersection.
Thomas Guide: Page and coordinates.

BUILDING DATA

Building Type Category:

29	W1	Wood, Light Frame
4	W2	Wood, Commercial and Industrial
5	S1	Steel Moment Frame
2	S2	Steel Braced Fame
	S3	Steel Light Frame
	S4	Steel Frame with Concrete Shear Masonry
2	C1	Concrete Moment Frame
2	C2	Concrete Shear Walls
4	C3	Concrete Frame with Infill Shear Walls
1	PC1	Precast/Tilt-up Concrete Walls with Lightweight Flexible Diaphragms
1	PC2	Precast Concrete Frames with Concrete Shear Walls
4	RM1	Reinforced Masonry Bearing Walls with Wood/Metal Deck Diaphragms
	RM2	Reinforced Masonry Bearing Walls with Precast Concrete Diaphragms
2	URM	Unreinforced Masonry Bearing Wall Buildings
	RMW	Reinforced Masonry Non-Bearing or Retaining Walls
	CW	Concrete Non-Bearing or Retaining Walls
	MH	Manufactured Housing

Occupancy:

30	APT	Apartment/Condominium	2	IND	Light Industrial/Warehouse
4	COM	Commercial/Retail		MAN	Manufacturing/Heavy Industrial
	DWL	Dwelling	8	OFF	Office
1	ESS	Essential (fire, police)	1	PRK	Parking
	GOV	Government	2	PUB	Public Assembly
	HIS	Historic		SCH	School
2	HOS	Hospital	4	UNV	University
2	HOT	Hotel			

No. Stories: Number of levels above grade.
Basement: Number of levels below grade.
Foundation:
50 SF Spread Footing
M Concrete Mat
5 P Piles or Caissons

Dwg.: Geot.:	Structural drawings and/or site-specific geotechnical reports with subsurface data known to be available (Y/N).
X Dimension: Y Dimension:	Longitudinal (X) and transverse (Y) overall dimensions in feet.
X Orientation: Y Orientation:	Orientation in degrees of longitudinal and transverse directions of the building with respect to north (e.g., 0 = north-south, 90 = east-west).
Base Isol.:	Base Isolated (Y/N).
Retrofit:	Previous earthquake damage repair or seismic strengthening.
Vertical System:	Summary of gravity system including roof and floor framing, columns, and longitudinal and transverse bearing walls.
Lateral System:	Summary of lateral system including roof and floor diaphragms, shear walls and moment or braced frames in each principle direction, and connections.
Special Features:	Irregularities, large openings, etc.

SITE

Soil Conditions:	Brief Summary (alluvium, stiff soil, shallow soil over rock, rock, etc.).
UBC Soil Profile:	1994 UBC (S1, S2, S3, S4).
Depth to Groundwater:	Depth to water table (feet).
Structural Fill: Hillside site:	Fill and/or hillside site (Y/N).

GROUND MOTION

Instrumented by: Station ID:	Station number and agency (CSMIP, USGS, etc.).

PERFORMANCE

Posting:	Northridge posting (R = red, Y = yellow, G = green).
Damage Description:	Use table of general damage descriptions by performance levels and systems.
Keywords:	Use keywords to capture additional building, site, ground motion or performance information for which an input field is not provided.
Comments:	Any additional comments on the special importance of this building as a case history with respect to "Vision 2000" objectives.
Ref. ID: Page:	List published documents (from Northridge References) that contain information on the building. If data is from unpublished consultant or public agency files, enter consultant or agency name, name of individual providing the information, and date provided.

Northridge Building Data

24-Mar-95

BUILDING DATA

BUILDING NAME	BLDG NO.	CITY	TYPE	OCC.	STORIES	BASE	YEAR	FDN	SITE	FILL	HILL	SOURCE
Cochran Ave. Apt.	1	Los Angeles	W1	APT	2	0	1947	SF	S2	NO	NO	MNH
Colbath Avenue Apt.	2	Sherman Oaks	W1	APT	2	0	1960	SF	S2	NO	NO	MNH
Baker-Taylor Software	3	Simi Valley	RM1	IND	1	0	1976	SF	S2	NO	NO	MNH
Cloverdale Avenue Apt.	4	Los Angeles	W1	APT	5	1	1988	SF	S2	NO	NO	MNH
Cedros Ave. Apt.	5	Panorama City	W1	APT	2	0	1977	SF	S2	NO	NO	MNH
Colfax Ave. Apt.	6	North Hollywood	W1	APT	2	0	1980	SF	S2	NO	NO	MNH
Denny Ave. Apt.	7	North Hollywood	W1	APT	2	0	1987	SF	S2	NO	NO	MNH
Hollywood Blvd. Apt.	8	Los Angeles	W1	APT	4	2	1986	SF	S2	UNK	YES	MNH
Hazeltine Ave. Apt.	9	Van Nuys	W1	APT	3	0	1986	SF	S2	NO	NO	MNH
Jolly Roger	10	Northridge	W2	COM	1	0	1970	SF	S2	NO	NO	MNH
Laurel Vista Apt.	11	Los Angeles	W1	APT	3	1	1987	SF	S2	NO	NO	MNH
Lanewood Ave. Apt.	12	Los Angeles	W1	APT	4	2	1991	SF	S2	NO	NO	MNH
Murietta Ave. Apt.	13	Sherman Oaks	W1	APT	2	0	1960	SF	S2	NO	NO	MNH
Mammoth Ave. Apt.	14	Sherman Oaks	W1	APT	2	0	1960	SF	S2	NO	NO	MNH
Palms Blvd. Apt.	15	Los Angeles	W1	APT	3	1	1988	SF	S2	NO	NO	MNH
Sherman Oaks Atrium	16	Sherman Oaks	S1	OFF	4	3	1985	SF	S2	NO	NO	MNH
Bank of America	17	Woodland Hills	RM1	COM	1	0	1960	SF	S2	UNK	NO	MNH
Vanowen St. Apt. B	18	Van Nuys	W1	APT	3	0	1986	SF	S2	NO	NO	MNH
Vanowen St. Apt. A	19	Van Nuys	W1	APT	3	0	1983	SF	S2	NO	NO	MNH
Ventura Blvd. Office	20	Encino	W2	OFF	2	0	1955	SF	S2	NO	NO	MNH
West Blvd. Apt.	21	Los Angeles	W1	APT	2	0	1925	SF	S2	NO	NO	MNH

							PERFORMANCE RATING							
BLDG NO	INST.	PHGA	DIST	DIR	MMI	POST.	OVERALL	VERT.	LAT.	ARCH.	EGRESS	MEP	CONTENT	OCC.
1	UNK	0.25	16	SE	7	NA	9	10	10	8	10	UNK	UNK	10
2	UNK	0.36	8	SE	9	R	4	4	4	5	5	6	5	4
3	UNK	0.3	10	N	8	Y	4	2	2	4	4	4	3	5
4	UNK	0.26	16	SE	7	NA	9	10	10	9	10	10	9	10
5	UNK	0.45	5	ENE	8	NA	8	9	9	6	9	UNK	UNK	10
6	UNK	0.37	9	ESE	8	NA	9	10	10	8	10	UNK	UNK	10
7	UNK	0.33	11	ESE	8	NA	9	9	8	8	10	UNK	UNK	10
8	UNK	0.3	14	SE	7	NA	9	9	9	8	10	9	9	10
9	UNK	0.38	7	ESE	8	NA	9	10	10	8	10	UNK	UNK	10
10	UNK	0.43	1	NW	9	NA	8	10	10	8	9	8	7	7
11	UNK	0.35	10	ESE	8	G	8	9	9	7	10	UNK	UNK	10
12	UNK	0.28	14	SE	7	G	10	10	10	9	10	UNK	UNK	10
13	UNK	0.36	8	SE	9	NA	9	10	10	8	10	10	9	10
14	UNK	0.36	8	SE	9	R	3	4	3	3	4	5	4	3
15	UNK	0.27	15	SSE	7	NA	10	9	9	8	10	10	9	10
16	UNK	0.38	6	SE	8	R	4	4	5	4	6	5	6	5
17	UNK	0.3	5	WSW	8	NA	8	10	10	8	6	9	9	8
18	UNK	0.39	3	SE	8	NA	9	9	9	7	10	UNK	UNK	10
19	UNK	0.38	7	E	7	NA	10	10	10	8	10	UNK	UNK	10
20	UNK	0.37	5	SE	8	G	8	10	9	6	7	8	8	10
21	UNK	0.23	18	SE	8	Y	5	4	5	6	8	UNK	UNK	5

BUILDING DATA

BUILDING NAME	BLDG NO.	CITY	TYPE	OCC.	STORIES	BASE	YEAR	FDN	SITE	FILL	HILL	SOURCE
Wilbur Medical Plaza	22	Tarzana	S1	OFF	3	3	1985	SF	S2	NO	NO	MNH
College Court	23	Northridge	W1	APT	3	0	1981	SF	S2	NO	NO	MNH
Develcore Center	24	Encino	S1	OFF	6	3	1981	P	S2	NO	NO	MNH
Foothill Village Apt.	25	Sylmar	W1	APT	2	0	1984	SF	S2	NO	NO	MNH
Venice Blvd. Apt	26	Venice	W1	APT	4	2	1990	SF	S2	NO	NO	MNH
Royce Hall	27	Los Angeles	C3	UNV	3	1	1930	P	S2	NO	YES	UCLA
Men's Gynasium	28	Los Angeles	C3	UNV	4	1	1932	P	S2	NO	NO	UCLA
Dickson Art Center	29	Los Angeles	C3	UNV	8	0	UNK	UNK	S2	NO	YES	UCLA
Stanford Medical Record	30	Santa Monica	C3	IND	2	0	1967	P	S2	NO	NO	UCLA
Plummer St. Church	31	Northridge	RM1	PUB	1	0	1979	SF	S2	NO	NO	MNH
Oakdale Ave. Church	32	Chatsworth	RM1	PUB	1	0	1971	SF	S2	NO	NO	MNH
Sherman Village	33	Sherman Oaks	W1	APT	4	0	1981	SF	S2	NO	NO	MNH
Kingsbury Court	34	Granada Hills	W1	APT	3	0	1979	SF	S2	NO	NO	MNH
L'Mirage	35	Sherman Oaks	W1	APT	4	0	1983	SF	S2	NO	NO	MNH
Van Nuys Holiday Inn	36	Van Nuys	C1	HOT	7	0	1966	P	S2	NO	NO	REF
USC University Hospital	37	Los Angeles	S2	HOS	6	1	1988	SF	S1	NO	NO	REF
Los Angeles County FCC	38	Los Angeles	S2	ESS	2	0	1990	SF	S1	YES	YES	FD
Best/Levitz Warehouse	39	Northridge	PC1	IND	1	0	UNK	SF	S2	NO	NO	REF
Edgemont St. Apt.	40	Los Angeles	W1	APT	4	2	1990	SF	S2	NO	YES	MNH
Jefferson Blvd. Office	41	Culver City	W2	OFF	2	0	1966	SF	S2	NO	NO	MNH
Santa Monica Blvd. Strip Mall	42	Los Angeles	W2	COM	1	0	1980	SF	S2	NO	NO	MNH
Shenandoah St. Apt.	43	Los Angeles	W1	APT	4	0	1988	SF	S2	NO	NO	MNH

							PERFORMANCE RATING							
BLDG NO	INST.	PHGA	DIST	DIR	MMI	POST.	OVERALL	VERT.	LAT.	ARCH.	EGRESS	MEP	CONTENT	OCC.
22	UNK	0.34	3	S	8	NA	9	9	9	8	10	UNK	UNK	10
23	UNK	0.46	2	NNE	9	G	7	7	7	8	8	8	8	7
24	UNK	0.37	5	SE	8	G	9	10	9	9	9	9	9	9
25	UNK	0.55	9	NE	8	G	9	9	8	10	9	7	10	9
26	UNK	0.27	16	SSE	6	G	10	10	9	9	9	9	10	10
27	UCLA	0.31	11	SE	7	NA	2	3	3	2	3	4	4	5
28	UCLA	0.31	11	SE	7	NA	4	4	5	4	6	6	7	8
29	UCLA	0.31	11	SE	7	NA	5	6	5	6	8	8	8	8
30	UNK	0.3	13	SSE	8	Y	3	3	3	4	4	4	2	4
31	UNK	0.48	3	NW	9	Y	9	9	8	6	10	10	9	9
32	UNK	0.45	3	NNW	8	R	9	9	8	4	9	9	8	7
33	UNK	0.36	8	SE	8	R	5	7	5	8	9	4	6	6
34	UNK	0.52	4	NW	9	R	7	8	5	8	9	8	7	6
35	UNK	0.36	8	SE	8	R	5	7	5	8	9	4	6	6
36	CSMIP	0.44	4	E	8	NA	6	7	7	6	6	UNK	4	6
37	CSMIP	0.23	22	SE	7	NA	10	10	10	10	10	10	10	10
38	CSMIP	0.22	24	SE	7	G	10	10	10	9	9	10	10	10
39	UNK	0.42	1	NW	9	R	2	2	2	2	3	2	3	3
40	UNK	0.26	17	SE	7	NA	10	10	10	9	10	UNK	UNK	10
41	UNK	0.23	18	SSE	6	NA	9	9	9	9	10	9	9	10
42	UNK	0.28	13	SSE	7	NA	9	10	10	8	10	UNK	UNK	9
43	UNK	0.27	14	SE	7	NA	9	10	8	8	10	UNK	UNK	9

BUILDING DATA

BUILDING NAME	BLDG NO.	CITY	TYPE	OCC.	STORIES	BASE	YEAR	FDN	SITE	FILL	HILL	SOURCE
Colonial House	44	West Hollywood	URM	APT	7	0	1925	SF	S1	NO	NO	DB
Pine Tree Terrace	45	Reseda	W1	APT	2	1	UNK	SF	S2	YES	NO	DB
Devonshire Arms	46	Northridge	W1	APT	3	0	1960	SF	S2	UNK	UNK	DB
Park Colfax	47	Studio City	W1	APT	3	0	UNK	SF	S2	YES	NO	DB
Grand Apt.	48	Sherman Oaks	W1	APT	5	1	1989	SF	S2	NO	NO	D M
Transamerica Bldg.	49	Woodland Hills	C2	OFF	4	0	1962	SF	S2	NO	NO	D M
CSUN Prkg. Strct. C	50	Northridge	PC2	PRK	4	1	1991	SF	S2	NO	NO	D M
Wilshire Blvd. Office	51	Santa Monica	S1	OFF	5	3	1993	SF	S2	NO	NO	REF
Union Rescue Mission	52	Los Angeles	C1	HOT	6	1	1993	SF	S2	NO	NO	REF
Stern Ave. Apt.	53	Sherman Oaks	W1	APT	3	1	1986	SF	S2	NO	NO	REF
Lassen Street Apt.	54	Northridge	W1	APT	4	0	1986	SF	S2	NO	NO	REF
Saint Johns Hospital	55	Santa Monica	C2	HOS	7	1	1966	SF	S2	NO	NO	REF
Hollywood Blvd. URM	56	Hollywood	URM	COM	2	0	1920	UNK	S2	NO	NO	REF
Sherman Oaks Centrium	57	Sherman Oaks	S1	OFF	3	3	1982	SF	S2	NO	NO	MNH
Simi Valley Plaza Home Base	58	Simi Valley	PC1	COM	1	0	1987	SF	S2	NO	NO	MNH
Sepulveda VA Bldg. 1	59	Sepulveda	RM2	HOS	2	1	1952	SF	S2	YES	NO	DEG
Sepulveda VA Bldg. 2	60	Sepulveda	C2	HOS	4	1	1952	SF	S2	YES	YES	DEG
Sepulveda VA Bldg. 3	61	Sepulveda	C2	HOS	6	1	1952	SF	S2	YES	NO	DEG
Sepulveda VA Bldg. 4	62	Sepulveda	RM2	HOS	2	1	1952	SF	S2	YES	NO	DEG
Sepulveda VA Bldg. 5	63	Sepulveda	RM2	HOS	2	1	1952	SF	S2	YES	NO	DEG
Sepulveda VA Bldg. 7	64	Sepulveda	RM2	HOS	1	1	1952	SF	S2	YES	NO	DEG
Sepulveda VA Bldg 10	65	Sepulveda	RM2	HOS	3	1	1952	SF	S2	YES	NO	DEG

							PERFORMANCE RATING							
BLDG NO	INST.	PHGA	DIST	DIR	MMI	POST.	OVERALL	VERT.	LAT.	ARCH.	EGRESS	MEP	CONTENT	OCC.
44	UNK	0.3	13	SE	7	Y	6	6	6	6	10	9	9	9
45	UNK	0.4	0.1	E	8	R	5	6	5	5	10	9	9	6
46	UNK	0.5	3	NNE	9	R	4	5	4	4	9	10	9	6
47	UNK	0.34	10	ESE	8	R	4	5	4	7	9	9	9	6
48	UNK	0.37	6	SE	9	Y	8	8	8	8	6	6	8	8
49	UNK	0.33	4	SW	8	R	3	3	3	4	4	6	6	4
50	UNK	0.47	2	NNE	9	R	1	1	1	1	1	1	UNK	1
51	UNK	0.31	14	S	8	NA	8	10	6	9	10	8	10	10
52	UNK	0.22	21	SE	7	NA	10	10	10	10	10	10	10	10
53	UNK	0.36	8	SE	9	NA	6	6	6	5	UNK	UNK	UNK	6
54	UNK	0.48	3	NE	9	R	3	4	3	2	2	3	3	3
55	UNK	0.31	14	SSE	8	R	5	5	5	5	5	6	6	6
56	UNK	0.27	16	SE	7	NA	4	6	5	4	4	UNK	UNK	5
57	UNK	0.38	6	SE	8	G	8	10	9	8	7	6	7	8
58	UNK	0.28	13	N	7	NA	7	9	9	9	7	7	6	6
59	UNK	0.5	4	ENE	8	G	9	10	9	7	9	6	6	9
60	UNK	0.5	4	ENE	8	G	7	9	9	5	9	7	5	7
61	UNK	0.5	4	ENE	8	R	4	4	4	4	4	4	3	4
62	UNK	0.5	4	ENE	8	G	8	9	9	8	9	8	6	9
63	UNK	0.5	4	ENE	8	G	9	10	9	9	10	8	9	9
64	UNK	0.5	4	ENE	8	G	9	10	9	9	10	9	10	9
65	UNK	0.5	4	ENE	8	G	7	8	8	6	8	7	6	7

BUILDING DATA												
BUILDING NAME	BLDG NO.	CITY	TYPE	OCC.	STORIES	BASE	YEAR	FDN	SITE	FILL	HILL	SOURCE
Sepulveda VA Bldg. 20	66	Sepulveda	RM2	HOS	2	0	1952	SF	S2	YES	NO	DEG
Sepulveda VA Bldg. 21	67	Sepulveda	C2	PUB	1	0	1952	SF	S2	YES	NO	DEG
Sepulveda VA Bldg. 22	68	Sepulveda	RM2	HOS	2	0	1952	SF	S2	YES	NO	DEG
Sepulveda VA Bldg. 23	69	Sepulveda	RM2	PUB	1	0	1952	SF	S2	YES	NO	DEG
Sepulveda VA Bldg. 24	70	Sepulveda	RM2	PUB	1	0	1952	SF	S2	YES	NO	DEG
Sepulveda VA Bldg. 25	71	Sepulveda	RM2	HOS	1	0	1952	SF	S2	YES	NO	DEG
Sepulveda VA Bldg. 40	72	Sepulveda	RM2	HOS	1	0	1952	SF	S2	YES	NO	DEG
Sepulveda VA Bldg. 99	73	Sepulveda	S4	HOS	2	0	UNK	SF	S2	YES	NO	DEG
Los Robles Medical Center	74	Thousand Oaks	C1	HOS	3	1	1966	SF	S1	YES	NO	MNH
Cedar-Sinai Main Bldg.	75	Los Angeles	S1	HOS	8	2	1971	P	S2	NO	NO	MNH
Cedar-Sinai Thalians	76	Los Angeles	C1	HOS	3	1	1971	P	S2	NO	NO	MNH
Bank of America Glendale	77	Glendale	S1	COM	14	2	1971	SF	S2	UNK	NO	NYA
Century Woods	78	Century City	W1	APT	4	0	1981	SF	S2	YES	YES	NYA
Walnut Ave. Office	79	Pasadena	W1	OFF	3	0	1974	SF	S3	NO	NO	NYA
Northridge Fash. Cntr. Bullock	80	Northridge	C2	COM	2	1	1970	SF	S3	NO	NO	NYA
Saint Monica's Church	81	Santa Monica	C3	PUB	4	1	1924	SF	S2	NO	NO	NYA
CSUN Admin. Bldg.	82	Northridge	C2	UNV	5	0	1963	SF	S2	UNK	NO	NYA
LA Memorial Coliseum	83	Los Angeles	C1	PUB	2	0	1930	P	S2	YES	NO	NYA
Olive View Lab	84	Sylmar	RM1	HOS	2	0	1954	SF	S2	UNK	NO	NYA
Blue Cross	85	Woodland Hills	S1	OFF	14	1	1976	P	S2	UNK	NO	NYA
Nordhoff St. Office A	86	Chatsworth	C2	OFF	2	0	1984	SF	S3	YES	NO	NA
Canoga Ave. Factory A	87	Canoga Park	C2	IND	2	0	1958	SF	S4	NO	NO	NA

							PERFORMANCE RATING							
BLDG NO	INST.	PHGA	DIST	DIR	MMI	POST.	OVERALL	VERT.	LAT.	ARCH.	EGRESS	MEP	CONTENT	OCC.
66	UNK	0.5	4	ENE	8	G	7	7	7	7	7	7	7	7
67	UNK	0.5	4	ENE	8	G	7	9	9	6	6	9	6	7
68	UNK	0.5	4	ENE	8	G	7	9	9	7	8	8	8	7
69	UNK	0.5	4	ENE	8	Y	6	7	6	6	7	6	5	6
70	UNK	0.5	4	ENE	8	G	10	10	10	8	9	9	10	10
71	UNK	0.5	4	ENE	8	G	10	10	10	9	9	UNK	UNK	9
72	USGS	0.5	4	ENE	8	R	4	6	3	4	4	6	6	5
73	UNK	0.5	4	ENE	8	G	6	9	8	7	8	7	5	6
74	UNK	0.2	19	W	6	NA	7	6	6	9	9	7	9	10
75	CODE	0.28	13	SE	7	NA	8	10	10	5	7	5	8	10
76	UNK	0.28	13	SE	7	NA	8	10	9	6	8	8	8	10
77	UNK	0.25	17	E	7	NA	10	10	10	10	10	10	10	10
78	UNK	0.28	15	SE	7	G	10	10	10	8	10	10	8	10
79	UNK	0.2	23	E	6	NA	10	10	10	8	10	9	10	10
80	UNK	0.43	2	NNW	9	R	2	2	6	3	2	UNK	UNK	2
81	UNK	0.31	14	S	8	R	2	3	2	4	4	UNK	UNK	5
82	UNK	0.45	2	NNE	9	R	5	6	5	5	5	7	6	5
83	UNK	0.2	20	SE	7	R	4	4	3	4	4	6	6	4
84	UNK	0.6	9	NE	8	G	9	10	9	9	10	9	9	10
85	UNK	0.32	4	SW	8	NA	9	9	8	9	10	9	9	9
86	UNK	0.39	2	NW	8	NA	4	7	6	6	7	6	6	5
87	UNK	0.33	4	SW	8	G	7	9	9	4	5	5	5	7

BUILDING DATA												
BUILDING NAME	BLDG NO.	CITY	TYPE	OCC.	STORIES	BASE	YEAR	FDN	SITE	FILL	HILL	SOURCE
Owensmouth Ave. Office	88	Canoga Park	RM1	OFF	1	0	1958	SF	S3	NO	NO	NA
Canoga Ave. Office A	89	Canoga Park	C2	OFF	1	0	1958	SF	S3	NO	NO	NA
Canoga Ave. Office B	90	Canoga Park	C2	OFF	1	0	1956	SF	S3	NO	NO	NA
Canoga Ave. Office C	91	Canoga Park	C2	OFF	1	0	1956	SF	S3	NO	NO	NA
Canoga Ave. Office D	92	Canoga Park	PC1	OFF	2	0	1960	SF	S3	NO	NO	NA
Canoga Ave. Factory B	93	Canoga Park	S2	IND	1	0	1956	SF	S3	NO	NO	NA
Nordhoff St. Office B	94	Chatsworth	C3	OFF	2	0	1958	SF	S3	NO	NO	NA
Nordhoff St. Office C	95	Chatsworth	C2	OFF	2	0	1960	P	S3	NO	NO	NA
Northridge Fashion Cntr. Store	96	Northridge	C3	COM	3	1	1971	P	S3	NO	NO	EQE
Ventura Blvd. Office A	97	Sherman Oaks	C2	OFF	12	0	1965	P	S3	NO	NO	EQE
Van Nuys Office Bldg.	98	Van Nuys	PC1	OFF	2	0	1984	SF	S2	UNK	NO	REF
Valencia Office Bldg. A	99	Valencia	S1	OFF	4	0	1992	SF	S1	YES	NO	EQE
Valencia Office Bldg. B	100	Valencia	S1	OFF	1	0	1992	SF	S1	YES	NO	EQE
Cedar-Sinai Halper Bldg.	101	Los Angeles	C3	HOS	6	1	1959	SF	S2	NO	NO	MNH
Middle School Bldg. S	102	San Fernando	C2	SCH	2	1	1916	SF	S2	UNK	UNK	MNH
Middle School Bldg. A	103	San Fernando	S1	SCH	2	0	1975	SF	S2	UNK	UNK	MNH
Middle School Auditorium	104	San Fernando	C2	SCH	1	1	1916	SF	S2	UNK	NO	MNH
Derring Ave. Commerical	105	Canoga Park	W2	COM	1	0	UNK	SF	S2	UNK	NO	DEG
N. Prairie Prkg. Structure	106	Inglewood	C3	PRK	2	0	UNK	SF	S2	UNK	NO	DEG
Westhills Plaza	107	Woodland Hills	S1	OFF	3	2	1988	P	S1	UNK	UNK	MNH
Lurline Gardens Bldg. B	108	Chatsworth	W1	APT	2	0	1971	SF	S2	UNK	UNK	MNH
Lruline Gardens Bldg. A	109	Chatsworth	W1	APT	2	0	1971	SF	S2	UNK	UNK	MNH

							PERFORMANCE RATING							
BLDG NO	INST.	PHGA	DIST	DIR	MMI	POST.	OVERALL	VERT.	LAT.	ARCH.	EGRESS	MEP	CONTENT	OCC.
88	UNK	0.33	4	SW	8	G	7	7	8	8	10	7	8	5
89	UNK	0.33	4	SW	8	Y	4	8	4	3	4	4	6	5
90	UNK	0.33	4	SW	8	G	10	10	10	10	10	10	8	10
91	UNK	0.33	4	SW	8	R	4	6	3	5	4	7	6	6
92	UNK	0.33	4	SW	8	R	5	8	5	3	6	7	6	6
93	UNK	0.33	4	SW	8	G	6	7	8	3	4	3	5	6
94	UNK	0.38	3	NW	8	Y	5	7	6	4	3	6	5	6
95	UNK	0.39	2	NW	8	NA	8	9	9	8	8	8	6	7
96	UNK	0.43	2	NNW	9	Y	3	3	4	3	5	5	6	5
97	UNK	0.37	6	SE	9	Y	6	8	6	6	UNK	UNK	UNK	8
98	UNK	0.42	3	E	8	Y	6	6	5	6	UNK	UNK	UNK	6
99	UNK	0.65	13	N	8	Y	6	4	4	9	8	8	6	6
100	UNK	0.65	13	N	8	Y	4	2	3	4	8	6	3	6
101	UNK	0.28	13	SE	7	R	5	5	5	7	4	3	3	2
102	UNK	0.5	8	NE	8	R	9	7	9	8	UNK	9	9	6
103	UNK	0.5	8	NE	8	R	7	10	9	3	4	8	4	7
104	UNK	0.5	8	NE	8	NA	8	8	8	8	10	10	9	9
105	UNK	0.33	4	WSW	8	NA	9	9	10	9	10	UNK	UNK	9
106	UNK	0.2	10	SSE	6	NA	10	10	10	10	10	UNK	UNK	10
107	UNK	0.3	5	SW	8	Y	5	8	5	7	9	9	7	6
108	UNK	0.4	3	NW	8	G	7	8	6	6	8	9	UNK	9
109	UNK	0.4	3	NW	8	Y	4	7	4	4	6	9	UNK	6

BUILDING DATA												
BUILDING NAME	BLDG NO.	CITY	TYPE	OCC.	STORIES	BASE	YEAR	FDN	SITE	FILL	HILL	SOURCE
N. Prairie Office Complex 1	110	Inglewood	S4	OFF	4	0	1971	P	S2	UNK	UNK	DEG
N. Prairie Office Complex 2	111	Inglewood	S1	OFF	4	0	1991	SF	S2	UNK	NO	DEG
Canoga Ave. Complex	112	Canoga Park	RM1	COM	1	0	UNK	M	S2	UNK	NO	DEG
Gains Residence	113	Reseda	W1	DWL	1	0	UNK	SF	S2	UNK	NO	DEG
Hyman Residence	114	Los Angeles	W1	DWL	1	1	UNK	SF	S1	UNK	YES	DEG
Bledsoe St. Industrial	115	Sylmar	PC1	IND	1	0	1978	SF	S2	NO	NO	REF
Rancho Santa Susana Com Ctr	116	Simi Valley	RM1	PUB	1	0	1994	SF	S3	YES	NO	DEG
Chatsworth Commerical Bldg. A	117	Chatsworth	PC1	COM	1	0	1969	SF	S2	UNK	NO	REF
Chatsworth Commerical Bldg. B	118	Chatsworth	PC1	COM	1	0	1969	SF	S2	UNK	NO	REF
Chatsworth Commerical Bldg. C	119	Chatsworth	PC1	COM	1	0	1977	SF	S2	UNK	NO	REF
Clifford Electronics	120	Chatsworth	PC1	IND	1	0	UNK	SF	S2	UNK	NO	DEG
Alfa Corporation	121	Chatsworth	PC1	COM	1	0	1980	SF	S2	UNK	NO	DEG
Gordon Residence	122	Northridge	W1	DWL	2	0	UNK	SF	S2	NO	NO	DEG
Northridge Fashion Cntr Broadw	123	Northridge	C2	COM	3	1	1970	SF	S3	NO	NO	REF
CSUN-Library Wing Addtn.	124	Northridge	S2	UNV	4	1	1990	P	S2	UNK	NO	REF
CSUN-Library Main Bldg.	125	Northridge	C3	UNV	4	1	1971	P	S2	UNK	NO	REF
JFK High School - Adm. A	126	Granada Hills	C3	SCH	3	0	1969	P	S2	UNK	NO	REF
Bronow Residence	127	Northridge	W1	DWL	1	0	1958	M	S2	UNK	NO	DEG
Rinaldi St. Commerical	128	San Fernando	PC1	COM	1	0	UNK	SF	S2	UNK	NO	DEG
Owensmouth Office Bldg. A	129	Woodland Hills	W2	OFF	2	0	UNK	M	S2	UNK	NO	DEG
Topanga Plaza Broadway	130	Canoga Park	C2	COM	3	0	1964	P	S2	UNK	NO	REF
Balboa Apt. Bldg. 1	131	Northridge	W1	APT	3	0	UNK	SF	S2	UNK	NO	DEG

							PERFORMANCE RATING							
BLDG NO	INST.	PHGA	DIST	DIR	MMI	POST.	OVERALL	VERT.	LAT.	ARCH.	EGRESS	MEP	CONTENT	OCC.
110	UNK	0.2	10	SSE	6	NA	10	10	10	9	UNK	UNK	UNK	10
111	UNK	0.2	10	SSE	6	NA	10	10	10	10	UNK	UNK	10	10
112	UNK	0.36	4	W	8	NA	7	9	9	5	UNK	7	UNK	UNK
113	UNK	0.4	1	E	8	NA	9	9	9	9	UNK	UNK	9	9
114	UNK	0.29	15	ESE	7	NA	7	9	9	6	UNK	UNK	UNK	8
115	UNK	0.6	7	NE	8	NA	2	3	2	2	3	UNK	2	6
116	UNK	0.35	8	NW	8	NA	10	10	10	9	10	10	10	10
117	UNK	0.37	4	N	8	NA	8	9	9	8	UNK	7	8	8
118	UNK	0.37	4	N	8	R	2	2	2	2	2	UNK	2	2
119	UNK	0.37	4	N	8	R	2	2	2	2	2	UNK	2	2
120	UNK	0.43	3	NW	8	G	9	9	9	7	4	9	8	8
121	UNK	0.4	3	NW	8	R	2	2	2	2	3	3	3	3
122	UNK	0.45	3	NNW	8	NA	9	9	8	8	UNK	UNK	7	9
123	UNK	0.43	2	NNW	9	Y	5	6	6	5	4	UNK	5	6
124	UNK	0.46	2	NNE	9	NA	6	6	5	4	UNK	UNK	7	5
125	UNK	0.46	2	NNE	9	NA	9	9	9	9	9	UNK	9	8
126	UNK	0.56	4	NE	9	NA	5	6	4	4	4	6	4	5
127	UNK	0.47	2	NE	9	NA	7	8	8	5	UNK	UNK	UNK	9
128	UNK	0.52	4	SW	9	Y	3	2	2	3	4	UNK	3	3
129	UNK	0.31	4	SW	8	NA	7	8	8	7	6	UNK	UNK	UNK
130	UNK	0.31	4	SW	8	Y	5	6	5	6	UNK	5	6	5
131	UNK	0.42	2	E	8	G	10	10	10	9	UNK	UNK	UNK	9

BUILDING DATA

BUILDING NAME	BLDG NO.	CITY	TYPE	OCC.	STORIES	BASE	YEAR	FDN	SITE	FILL	HILL	SOURCE
Balboa Apt Bldg. 2	132	Northridge	W1	APT	4	0	UNK	SF	S2	UNK	NO	DEG
Balboa Apt Bldg. 3	133	Northridge	W1	APT	4	0	UNK	SF	S2	UNK	NO	DEG
Balboa Apt Bldg. 4	134	Northridge	W1	APT	4	0	UNK	SF	S2	UNK	NO	DEG
Balboa Apt Bldg. 5	135	Northridge	W1	APT	3	0	UNK	SF	S2	UNK	NO	DEG
Roscoe St Industrial Bldg.	136	Van Nuys	C2	IND	8	0	1953	P	S2	UNK	NO	REF
Mullholand Apt's.	137	Woodland Hills	W1	APT	3	0	1988	SF	S2	UNK	YES	DEG
Encino Medical Center	138	Encino	C1	OFF	3	1	1971	P	S2	UNK	NO	DEG
Ottenstein Residence	139	Encino	W1	DWL	2	0	1992	SF	S2	UNK	NO	DEG
Sepulveda Office Bldg.	140	Sherman Oaks	C1	OFF	4	1	1977	P	S2	UNK	NO	REF
Sherman Oaks Tower Office	141	Sherman Oaks	C2	OFF	13	0	1965	P	S3	NO	NO	REF
Riverside Dr. Bldg. Office	142	Sherman Oaks	S1	OFF	6	1	1982	SF	S3	NO	NO	REF
Bloch Residence	143	Sherman Oaks	W1	DWL	2	0	UNK	SF	S2	UNK	NO	DEG
Graydon Residence	144	Sherman Oaks	W1	DWL	2	0	UNK	M	S2	UNK	YES	DEG
Graydon Recording Studios	145	Sherman Oaks	W1	IND	1	0	UNK	M	S2	UNK	NO	DEG
Tabaz Residence	146	Beverly Hills	W1	DWL	1	0	UNK	P	S1	YES	YES	DEG
Lankershim Blvd. Office	147	North Hollywood	S2	OFF	4	1	1986	SF	S2	UNK	YES	REF
Broderson Residence	148	Hollywood	W1	DWL	2	0	UNK	SF	S2	UNK	YES	DEG
Cresent Dr. Condominiums	149	Beverly Hills	W1	DWL	3	1	1986	SF	S2	UNK	NO	DEG
Hollywood Commercial	150	Hollywood	URM	COM	2	0	1920	UNK	S2	UNK	NO	REF
Pacheco Residence	151	Silverlake	W1	DWL	2	0	UNK	SF	S1	UNK	YES	DEG
Hazeltine Apartments	152	Sherman Oaks	W1	APT	2	0	UNK	SF	S2	UNK	UNK	MNH
Carl Thorpe School-Main Bldg	153	Santa Monica	W1	SCH	2	0	UNK	SF	S2	UNK	NO	DEG

							PERFORMANCE RATING							
BLDG NO	INST.	PHGA	DIST	DIR	MMI	POST.	OVERALL	VERT.	LAT.	ARCH.	EGRESS	MEP	CONTENT	OCC.
132	UNK	0.42	2	E	8	Y	6	6	6	6	UNK	UNK	UNK	6
133	UNK	0.42	2	E	8	G	9	9	9	8	UNK	UNK	UNK	9
134	UNK	0.42	2	E	8	G	9	9	9	9	UNK	UNK	UNK	9
135	UNK	0.42	2	E	8	G	9	9	9	9	UNK	UNK	UNK	9
136	UNK	0.45	4	E	8	NA	10	10	9	UNK	UNK	9	9	9
137	UNK	0.27	7	SW	7	NA	8	9	8	7	9	8	9	9
138	UNK	0.36	4	SE	8	R	6	5	8	5	3	6	8	6
139	UNK	0.36	4	SE	8	NA	9	10	10	8	UNK	UNK	UNK	9
140	UNK	0.38	6	SE	9	R	4	4	4	UNK	UNK	UNK	UNK	4
141	UNK	0.37	7	SE	8	Y	6	8	6	UNK	UNK	UNK	UNK	6
142	UNK	0.36	8	ESE	8	Y	6	7	6	6	UNK	UNK	UNK	6
143	UNK	0.36	8	SE	8	NA	8	8	7	7	UNK	7	UNK	9
144	UNK	0.36	8	SE	7	NA	8	7	8	4	UNK	6	UNK	9
145	UNK	0.36	8	SE	8	NA	8	7	7	8	UNK	UNK	UNK	UNK
146	UNK	0.34	10	SE	7	Y	6	6	6	6	UNK	6	6	6
147	UNK	0.34	10	ESE	7	Y	6	8	6	6	UNK	6	6	6
148	UNK	0.33	12	SE	8	NA	5	7	6	6	7	7	5	5
149	UNK	0.3	10	SE	8	G	9	9	9	9	UNK	UNK	UNK	9
150	UNK	0.27	16	ESE	7	NA	4	6	5	4	4	UNK	UNK	5
151	UNK	0.24	18	ESE	7	NA	6	5	5	7	UNK	7	4	6
152	UNK	0.36	8	SE	8	Y	9	10	8	9	10	10	9	10
153	UNK	0.31	13	SSE	8	NA	8	8	8	7	UNK	UNK	10	9

BUILDING DATA

BUILDING NAME	BLDG NO.	CITY	TYPE	OCC.	STORIES	BASE	YEAR	FDN	SITE	FILL	HILL	SOURCE
Carl Thorpe School-Classroom	154	Santa Monica	W1	SCH	1	0	UNK	SF	S2	UNK	NO	DEG
Werner Residence	155	Los Angeles	W1	DWL	2	1	1920	SF	UNK	UNK	YES	DEG
West Olympic Complex	156	Santa Monica	S1	OFF	5	4	UNK	SF	S2	UNK	NO	DEG
Skywalker Sound	157	Santa Monica	RM1	OFF	2	1	UNK	SF	S2	UNK	NO	DEG
Overland Office	158	Los Angeles	W1	OFF	3	0	UNK	UNK	S2	UNK	UNK	DEG
Wilshire Doheny Garage	159	Beverly Hills	C3	PRK	5	1	UNK	SF	S2	UNK	NO	DEG
Jones Residence	160	Los Angeles	W1	DWL	1	0	1930	P	S2	NO	NO	DEG
Holtzman Furniture	161	Los Angeles	C1	IND	4	4	1920	SF	S2	UNK	UNK	MNH
Pacific Palisades Residence	162	Pacific Palisades	W1	DWL	2	0	1990	SF	S2	UNK	UNK	MNH
St Monicas Parish Elementry	163	Santa Monica	URM	SCH	2	1	1930	SF	S2	UNK	NO	REF
Roberts Residence	164	Santa Monica	W1	DWL	1	0	UNK	UNK	S2	UNK	NO	DEG
Fifth St Office Bldg.	165	Santa Monica	S2	OFF	5	2	UNK	SF	S2	UNK	NO	DEG
Buug Alley	166	Santa Monica	URM	IND	1	0	UNK	M	S2	UNK	NO	DEG
Ocean West Condominium	167	Santa Monica	W1	APT	3	1	1982	SF	S2	UNK	NO	DEG
Santa Monica College Prkg	168	Santa Monica	C2	PRK	4	0	1979	SF	S2	UNK	NO	REF
Douglas Parking Garage	169	El Segundo	PC2	PRK	3	0	UNK	SF	S2	UNK	NO	DEG
Douglas Office Bldg.	170	El Segundo	PC1	OFF	2	0	UNK	SF	S2	UNK	NO	DEG
Ampex Bldg.	171	El Segundo	PC1	IND	1	0	UNK	SF	S2	UNK	NO	DEG
Western Laser Graphics	172	Valencia	PC1	IND	1	0	UNK	SF	S2	UNK	NO	DEG
Automobile Club S.C.	173	Santa Clarita	S1	COM	2	0	1991	SF	S3	YES	NO	REF
Newhall Land & Farm Bldg.	174	Santa Clarita	C1	OFF	2	0	1975	P	S3	UNK	UNK	REF
Spilio's Residence	175	Valencia	W1	DWL	2	0	UNK	SF	S3	UNK	NO	DEG

							PERFORMANCE RATING							
BLDG NO	INST.	PHGA	DIST	DIR	MMI	POST.	OVERALL	VERT.	LAT.	ARCH.	EGRESS	MEP	CONTENT	OCC.
154	UNK	0.31	13	SSE	8	NA	9	9	10	10	UNK	UNK	10	9
155	UNK	0.31	12	SSE	7	NA	9	10	10	9	10	10	9	10
156	UNK	0.3	14	SSE	8	NA	7	9	8	4	UNK	6	UNK	UNK
157	UNK	0.3	14	SSE	8	NA	10	10	9	9	UNK	UNK	10	9
158	UNK	0.27	13	SSE	8	NA	8	9	8	8	9	9	8	8
159	UNK	0.27	13	SSE	7	R	7	7	8	7	6	UNK	UNK	6
160	UNK	0.25	15	SSE	8	NA	8	9	8	8	9	9	9	9
161	UNK	0.23	22	SSE	7	NA	9	10	9	9	8	9	9	10
162	UNK	0.31	11	SSE	8	G	9	10	9	8	10	10	UNK	9
163	UNK	0.31	13	SSE	8	NA	5	6	5	4	4	7	6	6
164	UNK	0.31	13	SSE	8	NA	9	9	8	8	UNK	7	UNK	9
165	UNK	0.31	13	SSE	8	NA	7	8	6	7	UNK	7	UNK	8
166	UNK	0.31	14	SSE	8	NA	9	9	10	9	UNK	UNK	UNK	UNK
167	UNK	0.31	14	SSE	8	NA	9	9	9	8	UNK	UNK	UNK	9
168	UNK	0.31	14	SSE	8	NA	5	4	4	UNK	UNK	UNK	UNK	UNK
169	UNK	0.2	23	SSE	6	N	10	10	10	UNK	UNK	UNK	UNK	10
170	UNK	0.2	23	SSE	6	N	8	7	8	9	UNK	UNK	UNK	9
171	UNK	0.2	22	SSE	6	N	10	10	10	10	10	10	10	9
172	UNK	0.6	16	NNW	8	Y	8	8	9	6	UNK	UNK	6	6
173	UNK	0.63	13	N	8	R	4	5	3	3	3	7	3	3
174	UNK	0.63	13	N	8	G	9	10	10	UNK	UNK	UNK	8	9
175	UNK	0.63	13	N	8	Y	8	8	7	8	UNK	UNK	7	7

BUILDING DATA

BUILDING NAME	BLDG NO.	CITY	TYPE	OCC.	STORIES	BASE	YEAR	FDN	SITE	FILL	HILL	SOURCE
Sierra Commerical Bldg. A	176	Santa Clarita	RM1	COM	1	0	UNK	SF	S2	UNK	YES	DEG
Sierra Commercial Bldg. B	177	Santa Clarita	RM1	COM	1	1	UNK	SF	S2	UNK	YES	DEG
Sierra Commerical Bldg. C	178	Santa Clarita	S1	COM	2	1	UNK	SF	S2	UNK	YES	DEG
Sierra Commerical Bldg. D	179	Santa Clarita	RM1	COM	1	1	UNK	SF	S2	UNK	YES	DEG
Sierra Commerical Bldg. E	180	Santa Clarita	RM1	COM	1	0	UNK	M	S2	UNK	YES	DEG
Scholz Residence	181	Valenica	W1	DWL	2	0	1970	P	S2	NO	NO	DEG
Nacol Residence	182	Valencia	W1	DWL	1	0	1979	SF	S2	YES	YES	DEG
Track Auto	183	Santa Monica	URM	COM	1	0	UNK	SF	S2	UNK	NO	DEG
Bagdassariam Residence	184	Los Angeles	W1	DWL	1	0	UNK	SF	S2	UNK	NO	DEG
Guttman Residence	185	Los Angeles	W1	DWL	1	0	UNK	SF	S2	UNK	NO	DEG

							PERFORMANCE RATING							
BLDG NO	INST.	PHGA	DIST	DIR	MMI	POST.	OVERALL	VERT.	LAT.	ARCH.	EGRESS	MEP	CONTENT	OCC.
176	UNK	0.55	13	NNE	8	NA	9	9	9	9	UNK	UNK	UNK	9
177	UNK	0.55	13	NNE	8	N	7	9	9	4	4	UNK	UNK	9
178	UNK	0.55	13	NNE	8	R	3	2	2	9	UNK	3	UNK	3
179	UNK	0.55	13	NNE	8	N	9	10	10	9	UNK	UNK	UNK	9
180	UNK	0.55	13	NNE	8	N	9	10	10	9	UNK	UNK	UNK	9
181	UNK	0.7	11	N	9	N	9	9	8	8	UNK	UNK	8	9
182	UNK	0.7	11	N	9	N	9	9	8	8	UNK	UNK	6	9
183	UNK	0.31	14	SSE	8	R	3	2	4	2	4	UNK	3	3
184	UNK	0.25	17	SE	7	N	6	6	6	5	UNK	UNK	UNK	6
185	UNK	0.25	17	SE	7	N	7	6	8	8	UNK	7	UNK	9

Structural Engineers Association of California

VISION 2000

Performance Based Seismic Engineering of Buildings

Part 4: Moving the Blue Book Toward Performance Based Engineering

Prepared for:

California Office of Emergency Services

Prepared by:

Structural Engineers Association of California
Vision 2000 Committee

April 3, 1995
Final Report

Performance Based Seismic Engineering of Buildings

Part 4: Moving the Blue Book Toward Performance Based Engineering

prepared by

Structural Engineers Association of California

Vision 2000 Committee

Chris Poland, Chair
James Hill, Vice Chair
Roland Sharpe, Vice Chair

delegates:

Vitelmo Bertero
Joseph Nicoletti
Ronald Hamburger
Nabih Youssef
Anthony Court
Scott Stedman
Ajit Virdee
Robert Chittenden
Richard Phillips

editor:

Jeffrey Soulages

ad hoc members:

Robert Thacker
Vilas Mujumdar
Charles Thiel, Jr.

ex-officio members:

Ronald Nelson
Arthur Ross
Jane Andersen

contractors:

Richard Fallgren

555 University Avenue, Suite 126
Sacramento, CA

April 3, 1995

SEAOC Vision 2000 Committee

In recognition of the need for a new generation of design procedures, the Structural Engineers Association of California (SEAOC) has formed a special committee to look into the future and develop a framework for procedures that will yield structures of predictable seismic performance. This committee, called Vision 2000, first started their deliberations in 1990 and continues today with a multifaceted effort that is aimed at both the final goal and at the process that will deliver the profession to that end. While Vision 2000 intends to identify the framework, others are expected to translate it into design guidelines and codes as appropriate. Within SEAOC, the Seismology Committee will incorporate the new concepts, as they are available and appropriate, into future editions of the SEAOC Blue Book.

The first step of the Vision 2000 Committee was to establish the project parameters. These project parameters were incorporated into a Goal/Mission Statement. The goal of the Vision 2000 is as follows:

The goal of the Vision 2000 Committee is to develop the framework for procedures that yield structures of predictable seismic performance.

This framework will explicitly address life-safety, damageability, and functionality issues. The intent will be to produce structures whose seismic performance is predictable with a reasonable degree of confidence provided the actual materials and construction accurately reflect the design.

The second step was to establish a work plan and timeline for both the short-term phase of the project and for the mid-term and long-term phases of the project. A copy of this Workplan is available from the SEAOC Office.

The third step was to create the Vision 2000 Report which includes the results from all phases of the short-term work of the committee. This report includes: performance based engineering definitions, interim recommendations on how to apply performance based engineering concepts using today's available standards, a conceptual framework for a future performance based design code, preliminary lessons from the Northridge Earthquake, and a road map on moving the SEAOC Blue Book towards performance based design concepts. This report is divided into four parts as follows:

Part 1: Interim Recommendations
Part 2: Conceptual framework
Part 3: Preliminary Northridge Lessons
Part 4: Moving the Blue Book Towards Performance Based Engineering

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1. INTRODUCTION

1.1 Background

In order to transition from our current state of practice towards new performance based engineering concepts, the ideas of Vision 2000 need to be incorporated into future editions of the building code. The *Recommended Lateral Force Requirements and Commentary*, published by the Seismology Committee of the Structural Engineers Association of California (SEAOC) and known as the "Blue Book" is the most likely avenue to make a smooth and orderly transition. Through the Blue Book, information about performance based engineering can be disseminated to practicing engineers, and our current design process can be gradually transformed to incorporate performance based engineering concepts. This process will likely take many years and multiple editions of the Blue Book to complete.

The members of the Task D and E Subcommittees of Vision 2000 have been directed to prepare recommendations on how this transition can take place. Task D members concentrated their efforts on short-term "post-Northridge" goals and aided the SEAOC Seismology Committee on a number of important projects. These projects included: the 1994 UBC Emergency Code Change, the 1995 Blue Book Change, and the 1996 UBC Supplement. Task E members were directed to look farther into the future to create a road map of how performance based engineering can be incorporated into our future design practice using the SEAOC Blue Book. The results of both of these efforts, as well as those of the liaison efforts of Vision 2000 are included in Part 4 of the Vision 2000 Report.

All of the performance based engineering concepts introduced in this report as items to be added to the SEAOC Blue Book have been described in Parts 1 and 2 of the Vision 2000 Report. The reader is directed to these other parts of the report for clarification and elaboration of any of these specific performance based concepts.

1.2 Overview

It seems logical to divide performance based engineering modifications to the Blue Book into roughly three steps: short-term, mid-term, and long-term. The short-term modifications could take place during the 1995 and 1997 Blue Book phases. These modifications could include the following: the addition of Performance Definitions, matrix of Performance Objectives, Performance Based Engineering Framework, updated force/strength approach, and new nonstructural standards.

Mid-term modifications could take place during the Year 2000 Blue Book phase. These modifications could include the following: the addition of a robust Prescriptive Design Approach, complete Simplified Force/Strength Design Approach, Displacement Based Approach, and perhaps some advanced analysis approaches for acceptability checks such as nonlinear pushover methods or nonlinear dynamic analysis.

Long-term modifications could take place during the Year 2003 and beyond Blue Book phases. These modifications might include the following: the addition of an energy or comprehensive design approach, and additional advanced analysis approaches for acceptability checks. This would fully complete the conversion of the Blue Book to performance based engineering.

2. EFFORTS TOWARD PERFORMANCE BASED DESIGN CODES

2.1 SEAOC Seismology Committee

The Seismology Committee of the Structural Engineers Association of California has been developing recommended provisions for the earthquake-resistant design of structures, commonly known as the "Blue Book" since 1959. Since that first edition, the Blue Book has continually been at the forefront of the development of seismic design criteria for use in building codes.

It has always been the intention of SEAOC that the *Recommended Lateral Force Requirements and Commentary* be a "living" document. That is, the Blue Book should always change to reflect current advances in research and lessons learned from recent earthquakes. The last major revision effort to the SEAOC Blue Book was in 1988. It is expected that the addition of performance based concepts to the Blue Book will be a similar effort in time and labor to the 1988 effort.

It is anticipated that the Seismology Committee will continue to take the lead role in the development of performance based model design codes. Aided by resource documents produced by other SEAOC Committees such as Vision 2000, the Seismology Committee is well equipped to produce this next generation code.

2.2 SEAOC Vision 2000

In 1992, the Structural Engineers Association saw the need for a new generation of design procedures. SEAOC formed a special committee to look into the future and develop a framework for procedures that will yield structures of predictable performance. This committee, called Vision 2000, has embarked upon a multifaceted effort that is aimed at both the final goal and the process that will deliver the profession to that end.

While Vision 2000 intends to identify the framework, others are expected to translate it into design guidelines and code provisions as appropriate. Within SEAOC, the Seismology Committee will incorporate these new concepts, as they are available and appropriate, into future editions of the SEAOC Blue Book as described previously. However, the efforts of Vision 2000 are not intended to just stay within the SEAOC community. It is intended that the Vision 2000 ideas be distributed to as many different groups as possible to foster good communication between parallel efforts.

2.3 Other Efforts Toward Performance Based Design Codes

A. *ATC-33*

The ATC-33 project is a multi-year effort to provide technically sound, nationally applicable and consensus guidelines for the seismic rehabilitation of existing buildings. The project is a joint effort of the Building Seismic Safety Council (BSSC), the American Society of Civil Engineers (ASCE) and the Applied Technology Council (ATC). The effort is being funded by the Federal Emergency Management Agency (FEMA).

The work products of the ATC-33 effort will be a Guidelines document that provides detailed requirements and procedures, a Commentary volume which explains these requirements and procedures, and a Model Buildings volume that contains information on typical deficiencies, rehabilitation costs, and other background information.

The ATC-33 team has used performance based engineering concepts as the model for the Guidelines document. Inherent in the procedure is the selection of a Performance Level to be achieved by a particular building's rehabilitation, termed a "Rehabilitation Objective". Although specific objectives for certain situations are not given in the Guidelines, the Commentary provides guidance on typically appropriate choices of risk and Performance Levels for generic cases.

The ATC-33 effort is currently at the 50% level. The Vision 2000 Committee has incorporated some of the language of ATC-33 to develop the performance levels and objectives for Vision 2000. However, because the ATC-33 project deals only with existing buildings whereas Vision 2000 deals with both new and existing structures, the language is similar, but not identical.

B. *ATC-34*

The ATC-34 project is a five-year study into response modification factors and other critical seismic code issues. It is a joint effort by the National Center for Earthquake Engineering Research (NCEER) and the Applied Technology Council (ATC). The effort is being funded by the Federal Emergency Management Agency (FEMA).

The objectives of the study are to investigate the bases for the modification factors used in current seismic codes such as the Uniform Building Code (UBC) and the NEHRP Provisions, to build on the results of the ATC-19 project, "Evaluation of Structural Response Factors," and to provide a critical review of, and to recommend conceptual changes to current seismic design practice in the United States.

The first work product of ATC-34, "A Critical Review of Current Approaches to Earthquake-Resistant Design" has been completed. This report reviews the evolution of seismic design practice in the United States, critiques current seismic design practice, presents an action plan for improving reliability of current design practice, and makes recommendations for the studies and research needed to support the development of the next generation of seismic codes.

As many of the objectives of Vision 2000 and ATC-34 overlap, it is natural to compare the two. This, however, has not yet been done. The Vision 2000 committee recommends that the next set of tasks include such a comparison.

C. *FEMA PFS/Performance Based Design of Buildings*

The FEMA Problem Focused Studies project on "Seismic Design of Buildings: An Action Plan for Future Studies," is a multi-year effort towards the establishment of performance based seismic design standards for the design of new buildings. The project is an effort of the Earthquake Engineering Research Center which is affiliated with the University of California, Berkeley. The first stage of the project has been funded by the Federal Emergency Management Agency (FEMA).

The work product of the effort will be to provide seismic design procedures in a guidelines and commentary format that can form the basis of a new generation of seismic codes, and enable designers to design and construct structures of predictable seismic performance.

As the objective of this project and the Vision 2000 project is the same, Vision 2000 will be closely observing the results of the FEMA PFS project. However, since this effort has just started, no work products are yet available.

D. *NSF Study at University of Texas at Austin*

The NSF Study at the University of Texas at Austin is intended to bring together a panel of specialists in earthquake design and research to outline the actions that will be needed to produce and implement a new generation of codes. The project is an effort of the University of Texas at Austin working in concert with other NEHRP agencies such as USGS, NIST and FEMA. The work is being funded by the National Science Foundation.

The aim of the work is to define the role of the National Science Foundation in nurturing the code development process so that new generation codes can be developed in a timely, systematic manner. The panel is concerned not with the philosophy, details, or specifics of a new code, but only the process for achieving the goal.

As many of the objectives of the Vision 2000 effort and this NSF effort are the same, Vision 2000 will be closely observing the results of the NSF Study. However, since this effort has just started, no work products are yet available.

2.4 USGS Ground motion Efforts

Most of the future efforts of USGS with regard to code development center around the Ground Motion Initiative. The Ground Motion Initiative is a multi-year project to develop ground motion input for the next, safer generation of building and seismic design codes and professional practices. The project is an effort of the United States Geological Survey aided by private contractors and other organizations such as ATC. The work is being funded by USGS.

The goal of the Ground Motion Initiative is to make the USGS and the earth sciences partner in the development of the next generation of seismic design practices and regulations. Ground motion includes all aspects of earthquake occurrence, including source mechanics, propagation, and site response. The work products of the Initiative will be new maps with better ground motion information than is currently available, and new ground motion characterizations to be used in the next generation of building codes.

A sub-project of the Ground Motion Initiative is Project '97 which is a joint effort of FEMA/BSSC and USGS. The goal of Project '97 is to develop new maps for the 1997 NEHRP Provisions. The work products of the effort will be new seismic hazard maps and new seismic design procedures that utilize these new maps.

As many of the objectives of the Vision 2000 effort and these USGS efforts are the same, Vision 2000 will be closely observing the results of the USGS Ground Motion Initiative and Project '97. However, since these efforts have just started, no work products are yet available. Vision 2000 members have attended the first meetings of the Ground Motion Initiative and will continue to do so as part of its ongoing liaison effort.

3. MODIFICATIONS TO THE BLUE BOOK AND UNIFORM BUILDING CODE

3.1 Short-term Modifications

The SEAOC Seismology's *Recommended Lateral Force Recommendations and Commentary* forms the basis for the seismic design provisions in the International Conference of Building Official's Uniform Building Code (ICBO-1994). The SEAOC Seismology Committee is constantly assessing the effects of earthquakes on structures designed in accordance with the provisions of the Blue Book and UBC. While small scale structures and full scale structural subassemblies can be tested in the laboratory for the effects of earthquake ground shaking, tests of full scale structures is not feasible; hence, the only way real structures are tested are by earthquakes themselves. The proving ground of earthquakes becomes the test for the seismic provisions of the SEAOC Blue Book and the UBC. After each damaging earthquake in California, as well as after earthquakes all over the world, SEAOC assesses the performance of the Blue Book provisions. Changes are made to the Blue Book and are also submitted to the International Conference of Building Officials (ICBO) to update the UBC as part of the ICBO Code Change process. The Northridge Earthquake has been the most recent test of structures built to the UBC Code.

A. *1994 UBC Emergency Code Change*

The Northridge earthquake demonstrated that the current prescriptive provisions for the steel Special Moment Resisting Frame (SMRF) connection were flawed. The problem is complex and not fully understood at this time. While the behavior of the connection is being investigated and solutions being developed, SEAOC Seismology undertook three steps in the interim: 1) development of an emergency code change for the UBC which eliminates the current prescriptive provisions; 2) development of an interim guidance document for the design of new and the repair of existing steel SMRF connections; and 3) development of an interim recommendation which supplements and interprets the emergency code change.

The documents incorporated the lessons learned from the Northridge earthquake, as well as the most recent research conducted at the University of Texas. The interim guidance document gives an engineer guidance on one method of reinforcing the connection and on the development of improved welding procedures. The documents are oriented to the practicing engineer. These documents will be updated as more data becomes available. The interim guidance document is attached as Appendix B. The interim recommendation and emergency code change is included in the recently published SAC Advisory #3 (SAC-1995).

B. *1995 Blue Book Changes*

The Blue Book is currently in the process of being updated to include SEAOC Seismology sanctioned code changes which reflect lessons learned from the recent Northridge earthquake. These SEAOC code changes are also being submitted to ICBO as proposed changes for the 1996 UBC Update and the 1997 UBC. These code changes are based on lessons learned from Northridge as well as other data. However, the UBC code changes will not be effective until May 1996 at the earliest for the 1996 Update and March 1997 for the 1997 UBC. Because of the long lead time for code changes, the next edition can only take the first small but important steps towards performance based engineering.

The Blue Book is being updated to include previous code changes made to the UBC and acceptable to SEAOC from 1991 up to and including the 1994 UBC. Extensive revision and development work will be done on the Commentary. The Commentary is as important as the Recommendations as it explains the reasoning behind the provisions in the Requirements. A special chapter in the Commentary is devoted to lessons learned from the Northridge earthquake. This chapter is included as Appendix C. The Blue Book update effort is anticipated to be completed in the fall of 1995.

It is proposed that the 1995 edition of the Blue Book incorporate the Performance Definitions developed in Chapter 2 of Part 1 of the Vision 2000 Report. Part 1 defines performance based engineering, the four earthquake performance levels, the four earthquake hazard levels and the three performance objectives to be chosen by the owner. This information can be incorporated as an additional appendix to the current 1995 Blue Book proposal, perhaps using a different color page to differentiate the appendix from the current Blue Book Commentary.

C. *1996 UBC Supplement*

As noted above, an emergency code change deleting the provisions for the prescribed SMRF connection and requiring that the connection be shown by testing or calculation to have sufficient inelastic rotational capacity and adequate strength considering steel overstrength and strain hardening was successfully inserted in the 1994 UBC. As more data becomes available a code change for the connection will be filed for the 1997 UBC if appropriate.

Code changes based on lessons learned from the Northridge earthquake were drafted and submitted for the 1996 UBC supplement. Topics included:

a) purpose of earthquake provisions,
b) seismic criteria for inverted pendulum building systems and elements,
c) deformation compatibility enhancements,
d) anchorage forces for flexible diaphragms,
e) structural observation criteria for wood shear walls and diaphragms,
f) special inspection requirements for concrete SMRF's,

g) provisions for shear strength of round columns,
h) provisions for the steel SMRF connection,
i) increased b/t ratio's for braces in steel braced frames and the prohibition of "ordinary" braced frames in Seismic Zones 3 and 4,
j) provisions for positive shear transfer of diaphragm forces in Type V buildings,
k) reduction in hold-down values and diaphragm ratios for wood shear walls, and
l) a reduction in the shear values for gypsum board and plaster shear walls.

Copies of the code changes are included as Appendix D.

D. *1997 Blue Book and UBC Changes*

The next full edition of the Blue Book is slated to be published in 1997. This edition will probably be very similar in format to the old edition with some amendments and a much larger appendix. By 1997, many parts of the PBE Framework as well as a consensus version of the matrix of Performance Objectives as discussed in Part 1 should be complete. In addition, much of the work towards new provisions for the seismic design of nonstructural components should be complete. All three of these items should be included as appendix items in the 1997 Blue Book, perhaps using a different color page to differentiate the appendix from the current Blue Book Commentary.

Another step to be completed by 1997 is to modify the current code force/strength design procedure to incorporate performance based engineering features. In other words, our current equivalent static lateral force procedures could be modified to yield structures of more predictable seismic performance by changing the "R_w" factors to more meaningful values for different building types. These modifications should be included as code items in the 1997 Blue Book.

3.2 Mid-term Modifications

Mid-term modifications are those changes to the Blue Book which need a major effort to complete before they could be incorporated. Much of this effort will be undertaken by subcommittees of the SEAOC Seismology Committee and other similar groups over the next five years. A complete rewrite of the SEAOC Blue Book is anticipated to occur around the year 2000, hence the "Vision 2000" name. At this stage, a large portion of the basic description of performance based engineering will have been written, and the first parts incorporated in the code section of the Blue Book.

The first step in the development of the Year 2000 Blue Book is to move the Performance Definitions, matrix of Performance Objectives, and PBE Framework out of the appendix and into the code pages. This will require a major rewrite of the current Section 1 of the Blue Book. In addition, the nonstructural provisions should also be moved from the appendix and into Section 1 or into a new section devoted to the seismic design of nonstructural components.

The second step in the development of the Year 2000 Blue Book is to incorporate some of the other design approaches and acceptability checks that are contained as part of the PBE Framework into the appendix of the Year 2000 Blue Book. In an ideal world, all the approaches and acceptability checks should be developed before the PBE Framework is incorporated into the code pages of the Blue Book. However, these approaches, which include the Prescriptive Design Approach, Simplified Force/Strength Approach, Displacement Based Approach, Energy Approach and Comprehensive Design Approach, and the many acceptability checks such as nonlinear pushover analysis and nonlinear dynamic analysis will be complete to varying degrees by the year 2000. Those that are complete should be included in an appendix to the 1997 Blue Book, perhaps using a different color page to differentiate the appendix from the current Blue Book Commentary. All other approaches and checks that are not complete should have place-holders in the appendix so that they may be incorporated into later interim editions of the Blue Book.

3.3 Long-term Modifications

Long-term modifications are those changes to the Blue Book which need a major effort to complete before they could be incorporated. Much of this effort will be undertaken by university researchers, subcommittees of the SEAOC Seismology Committee and other similar groups over the next five to ten years. The editions of the Blue Book after the major rewrite in the year 2000 to incorporate the PBE Framework and performance based engineering are anticipated to add design and analysis procedures and checks as they are developed. At the end of this stage, the conversion of the Blue Book to performance based engineering will be complete.

Between the Year 2000 Blue Book and the next two Blue Book rounds, presumably, sometime in 2003 and sometime in 2006, the design approaches and acceptability checks included in the appendix would go through a consensus review and trial design process to fine tune the code language and procedures. Those approaches and checks which have completed their review by the time of each new edition should be moved from the appendix to the code pages of the Blue Book. Those that have not yet completed review should remain in the appendix pages and those newly developed procedures should be added to the appendix pages. By the Year 2006 Blue Book, it is anticipated that all design approaches and acceptability checks will have been included in the code pages of the Blue Book.

4. CONCLUSIONS

The modifications to the Blue Book towards performance based engineering concepts will come in roughly three steps: short-term, mid-term, and long-term. These modifications encompass at least five editions of future SEAOC Blue Books. These modifications might include the following:

1995 Blue Book

a. Performance Definitions added to appendix.

1997 Blue Book

a. Matrix of Performance Objectives added to appendix.
b. PBE Framework added to appendix.
c. Nonstructural provisions added to appendix.
d. Current force/strength procedures modified in code pages and commentary.

Year 2000 Blue Book

a. Performance Definitions moved from appendix to code pages.
b. Matrix of Performance Objectives moved from appendix to code pages.
c. PBE Framework moved from appendix to code pages.
d. Nonstructural provisions moved from appendix to code pages.
e. Prescriptive Design Approach added to appendix.
f. Simplified Force/Strength Approach added to appendix.
g. Some advanced acceptability checks added to appendix.

Year 2003 Blue Book

a. Move design approaches that have completed consensus process from appendix to appropriate code pages.
b. Move acceptability checks that have completed consensus process from appendix to appropriate code pages.
c. Displacement Based Approach added to appendix.
d. Energy Based Approach added to appendix.
e. Comprehensive Design Approach added to appendix.
f. Remainder of advanced acceptability checks added to appendix.

Year 2006 Blue Book

a. Move the remaining design approaches from appendix to appropriate code pages.
b. Move the remaining acceptability checks from appendix to appropriate code pages.

5. REFERENCES

Structural Engineers Association of California, SEAOC Seismology Committee, *Recommended Lateral Force Requirements and Commentary*, Sacramento, California, 1990.

International Conference of Building Officials, *Uniform Building Code*, Whittier, California, 1994.

SAC Joint Venture, *Steel Moment Frame Connection, SAC Advisory #3*, SAC 95-01, Sacramento, California, 1995.

APPENDICES

APPENDIX A

VISION 2000 LIAISON EFFORTS

A.1 Introduction

The Liaison Committee was established to seek information about any similar on-going activity by other professional organizations for the development of performance based seismic design criteria. The intent was to solicit technical information that may be of use to the SEAOC Vision 2000 Committee. In addition, it was felt necessary that the public and other organizations be kept informed about the Vision 2000 efforts.

A.2 Short-term Goals and Accomplishments

A list of professional organizations was compiled in September 1994. This list contained the names of 53 organizations (see Table A-2). Letters of introduction were mailed out to these organizations. In these letters, brief background information about the SEAOC and the formation of Vision 2000 Committee was provided. The mission statement for the Vision 2000 project along with a questionnaire were also included. The questionnaire was tailored to seek information about their interest in the project, availability of information about any similar on-going activity, possibility of technical or financial support, interest in attending seminars and receiving future publications (see attached).

To date, the Liaison Committee has received nineteen responses. Most of these responses are positive, except that they do not offer much technical information about any parallel on-going activity (see Table A-1). These responses indicate interest in receiving future publications and a copy of the final document. They show interest in attending seminars and workshops.

A.3 Long-term Goals

It is anticipated that the final document will be distributed widely to all interested organizations. Communication is to be maintained with all interested organizations. Dates for future seminars or workshops are to be advertised well in advance for larger attendance and participation.

A.4 Conclusions

The efforts to establish liaison with other organizations have been successful. Nineteen organizations have responded very positively and are eager to receive the Vision 2000 Report. Although no technical assistance has been offered by any organization, it is probably due to the fact that not many advances in the performance based criteria have been made elsewhere.

A.5 Recommendations

The nineteen organizations and agencies who have responded to the SEAOC Vision 2000 project questionnaire should be put on the mailing list for the project report. It is further recommended that a serious effort be made in presenting the performance based seismic design criteria to structural design community. This may be done by presenting seminars or short courses at the local SEA levels as well as at national structural engineering forums.

Organization	Interests	Current Projects with PBE	Primary Concerns	Provide Support?	Information Dissemination			
					Seminar	Publications	Workshop	Other
American Concrete Institute	Safety and serviceability performance of concrete structures	Workshop re: new initiatives in seismic research	Safety and Serviceability after EQ	No		X	X	
American Consulting Engineers Council		None		No	X	X	X	
American Institute of Architects			Public Safety	No	X	X	X	
American Institute of Steel Construction		Resolve Northridge steel moment frame problems (with SAC)		Comm. participation				
Steel Joist Institute	Development of resource document for ASCE standard use	ASCE Standard 7-93; also with ATC, BSSC and FEMA document: Seismic Rehab of Buildings		Yes		X		
Brick Institute of America	Stated objectives of earthquake performance		Performance of masonry struct.	Review of info.	X	X	X	
Building Officials and Code Administration		NEHRP provisions		No				
Canadian National Committee on Earthquake	All aspects	Reasses Canadian Code requirements for year 2000 edition	Life-safety; cost; accuracy	Tech. cooperation	X	X		
Interagency Committee on Seismic Safety	Receiving information	ICSSC RP-4	Life-safety	Tech. dialogue				
Los Angeles County Building Department	Life-safety and property preservation	Task Force to make recommendations for strengthening Building Codes after Northridge EQ	Seismic performance of buildings with minimum damage	Review of materials	X	X	X	
Masonry Institute of America		Masonry shear walls and wall frames	Ease of construction, reliable perf.	Some	X	X	X	X - News Release

Table A-1a: Summary of Organization Responses

Organization	Interests	Current Projects with PBE	Primary Concerns	Provide Support?	Information Dissemination			
					Seminar	Publications	Workshop	Other
Metal Building Manufacturers Association	Damage concept	May start in 1996	Maintain structural integrity, minimize secondary damage	Tech. support via liaison activities and info exchange	X	X		
National Ready Mixed Concrete Association				Maybe	X	X		
Permanent Committe for Structural Safety	Item 1, 7 & 8 of Mission Statement	Update of their seismic code	Life-safety and maintain function	Tech. support	X	X		
Southern Building Code	Building seismic performance	SBCCI Seismic Prescriptive Standards		Limited Staff time for review			X	
Steel Joist Institute	Light steel framed structures	Recommendation for low-rise structures using steel joists for seismic design	Use of steel joists for seismic design	Advisory capacity	X	X	X	
Structural Engineers Association of Illinois	Coordinating level of seismicity with predictions for nonstructural damage	None	Structural safety	No			X	
Structural Engineers Association of Oregon				No	X	X		
The Masonry Society	Structural and material requirements for masonry structures at each level	Has investigating team to look at the performance of masonry structures in EQs and make recommendations to improve performance		Possibly technical or staff support	X	X	X	X - Internet

Table A-1b: Summary of Organization Responses

AMERICAN CONCRETE INSTITUTE
Mr. George S. Leyh
22400 West Seven Mile
Detroit, MI 48219

AMERICAN CONSULTING ENGINEERS COUNCIL
Mr. Howard Messner
1015 15th Street NW, Suite 802
Washington, DC 20006

AMERICAN INSTITUTE OF ARCHITECTS
Mr. Terrace M. McDermott
1735 New York Avenue, N.W.
Washington, DC 20006

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
Mr. Nestor R. Iwankiw
One East Wacker Drive, Suite 3100
Chicago, IL 60601-2001

AMERICAN INSURANCE SERVICES GROUP, INC.
Mr. John Mineo
85 John Street
New York, NY 10038

AMERICAN IRON AND STEEL INSTITUTE
Mr. Andy Sharkey
1101 17th Street NW, Suite 1300
Washington, DC 20036-4700

AMERICAN FOREST AND PAPER ASSOCIATION
Mr. David Tyree
P.O. Box 419
Georgetown, CA 95634

AMERICAN SOCIETY OF CIVIL ENGINEERS
Mr. James E. Davis
345 E. 47th Street
New York, NY 10017-2398

APPLIED TECHNOLOGY COUNCIL
Mr. Chris Rojahn
555 Twin Dolphin Drive, #550
Redwood City, CA 94065

ASSOCIATED GENERAL CONTRACTORS OF AMERICA
Mr. James W. Lail
1957 E Street, NW
Washington, DC 20006

ASSOCIATION OF MAJOR CITY BUILDING OFFICIALS
Mr. Arthur Johnson
City of Los Angeles
Department of Building and Safety
200 N. Spring Street, #405
Los Angeles, CA 90012

BSSC
Mr. Jim Beavers
1201 L Street, NW Suite 400
Washington, DC 20005

BRICK INSTITUTE OF AMERICA
Mr. Mark Nunn
Senior Building Codes Engineers
11490 Commerce Park Drive, #300
Reston, VA 22091

BUILDING OFFICIALS AND CODE ADMINISTRATION
Mr. Paul Heilstedt
BOCA, International
4051 West Flossmoor Road
Country Club Hills, IL 60477

CALIFORNIA GEOTECHNICAL ENGINEERS ASSOCIATION
Mr. Alan Kropp
c/o Read Jones Christofferson Ltd.
2140 Shattuck Avenue
Berkeley , CA 94704

CANADIAN NATIONAL COMMITTEE ON EARTHQUAKE
Dr. R H Devall
210 West Broadway
Vancouver, B.C.
CANADA V5Y 3W2

CITY OF LOS ANGELES, BUILDING DEPARTMENT
Mr. Tom Remillard
Superintendent of Building
900 South Fremont Avenue
Alhambra, CA 91803

CITY OF SAN JOSE, BUILDING DEPARTMENT
Mr. Dennis Day
801 North First Street, #200
San Jose, CA 94583

CITY OF SAN FRANCISCO, BUILDING DEPARTMENT
Mrs. Joan MacQuarrie
1515 Mission Street
San Francisco, CA 94103

CITY OF SACRAMENTO, BUILDING DEPARTMENT
Tim Sullivan
Manager of Building Inspections
1231 I Street, Room 200
Sacramento, CA 95814

CITY OF SAN DIEGO, BUILDING DEPARTMENT
Mr. Tom Trainor
4995 Murphy Canon Road, #200
Mail Stop 74
San Diego, CA 92123

CONCRETE MASONRY ASSOCIATION OF CALIFORNIA AND NEVADA
Mr. Stuart Beavers
6060 Sunrise Vista Drive, Suite 1875
Citrus Heights, CA 95610

CONCRETE REINFORCING STEEL INSTITUTE
Mr. James Nevin
1110 East Alcosta Avenue
Glenora, CA 91740

CORPS OF ENGINEERS
Col. John Reese
1325 J Street
Sacramento, CA 95814

Figure A-2a: List of Organizations

DSA/ORS, STATE OF CALIFORNIA
Mr. Vilas Mujumdar
Department of General Services
Office of Regulation Services
400 P Street, Fifth Floor
Sacramento, CA 95814

EARTHQUAKE ENGINEERING RESEARCH INSTITUTE
Mr. Susan Tubbesing
499 14th Street, Suite 320
Oakland, California 94612-1902

GENERAL REINSURANCE CORPORATION
Ms. Cynthia Bordelon
Assistant Vice President
300 S. Riverside Plaza, Suite 200 North
Chicago, IL 60606

INTERAGENCY COMMITTEE ON SEISMIC SAFETY
Dr. Richard Wright
Building and Fire Research Lab
Building 226/Room B218
Gaithersburg, MD 20899

KLEINFELDER, INC.
Mr. Gerald Salontai
9555 Chesapeake Drive, #101
San Diego, CA 92123

MASONRY INSTITUTE OF AMERICA
Mr. James Amrhein
2550 Beverly Boulevard
Los Angeles, CA 90057

METAL BUILDING MANUFACTURERS ASSOCIATION
Mr. Lee Shoemaker
1300 Summer Avenue
Cleveland, OH 44115-2850

NATIONAL CONFERENCE OF STATES ON CODES
Mr. Robert Wible
505 Huntmar Park Drive, #210
Hemdon, VA 22070

NATIONAL ELEVATOR INDUSTRY, INC.
Mr. George Kappenhagen
c/o Schindler Elevator Corporation
20 Whippany Road
Morristown, NJ 07962-1935

NATIONAL FIRE SPRINKLER ASSOCIATION
Mr. Jack Wood
c/o The Viking Corporation
Hastings, MI 49058

NATIONAL FOREST PRODUCTS
Mr. Bob Kirshner
1250 Connecticut Avenue, NW, Suite 200
Washington, DC 20036

NATIONAL INSTITUTE OF BUILDING SCIENCES
Mr. David A. Harris
1201 L Street, NW, Suite 400
Washington, DC 20005

NATIONAL INSTITUTE OF STANDARD AND TECHNOLOGY
Mr. H.S. Lew
NIST
Building 226, Room B168
Gaithersburg, MD 20899

NATIONAL READY MIXED CONCRETE ASSOCIATION
Mr. Robert Garbini
900 Spring Street
Silver Spring, MD 20910

OSHPD
Mr. Sharad Pandya
1600 9th Street, NW, Suite 400
Sacramento, CA 95814

PERMANENT COMMITTEE FOR STRUCTURAL SAFETY
Civil Engineering/Chairman
Conj. Las Flores Edif.
Clavel Apt. 113, Avenue Paez El Paraiso
Carcas, Venezuela 1021

PORTLAND CEMENT ASSOCIATION
Mr. Mark Kluver
4105 Terra Alta Drive
San Ramon, CA 94583

PRECAST/PRESTRESSED CONCRETE INSTITUTE
Mr. Douglas Mooradian
100 North Brand Boulevard, #200
Glendale, CA 91203

RACK MANUFACTURERS INSTITUTE
Dr. Victor Azzi
1100 Old Ocean Boulevard
Rye, NH 03870

SEISMIC SAFETY COMMISSION
Mr. Thomas Tobin
1900 K Street, #100
Sacramento, CA 95814

SOUTHERN BUILDING CODE CONGRESS
Mr. Bill Tangye
900 Montclair Road
Birmingham, AL 32513

STEEL DECK INSTITUTE
Mr. Bernard Cromi
Steel Deck Institute
P.O. Box 9506
Canton, OH 44711

STEEL JOIST INSTITUTE
Mr. Donald Murphy
1205 48th Avenue, North #A
Myrtle Beach, SC 29577

STRUCTURAL ENGINEERS ASSOCIATION OF ARIZONA
Dr. Ehsami
206 Civil Engineering Building
University of Arizona
Tucson, AZ 85721

Figure A-2b: List of Organizations

STRUCTURAL ENGINEERS ASSOCIATION OF ILLINOIS
Ms. Jeanne Vogelzang
203 North Wabash Avenue, Room 805
Chicago, IL 60601

STRUCTURAL ENGINEERS ASSOCIATION OF OREGON
Mr. David Bugni
P.O. Box 4801
Portland, OR 97208-4801

STRUCTURAL ENGINEERS ASSOCIATION OF WASHINGTON
Mr. Fulton Desler
P.O. Box 11700
Tacoma, WA 98411

THE MASONRY SOCIETY
Mr. Williams Palmer
2619 Spruce Street, Suite B
Boulder, CO 80302

WESTERN STATES CLAY PRODUCTS ASSOCIATION
Mr. Donald Wakefield
Western States Clay Products Association
2171 E. Lorita Way
Sandy, UT 84092

Figure A-2c: List of Organizations

STRUCTURAL ENGINEERS ASSOCIATION
OF CALIFORNIA
555 UNIVERSITY AVENUE, SUITE 126
SACRAMENTO, CALIFORNIA 95825
(916) 427-3647 FAX (916) 568-0677

January 23, 1994

Company

Dear Mr./Mrs.:

Several months ago the Structural Engineers Association of California (SEAOC) began a major project to develop a framework for procedures that would yield structures of predictable seismic performance. The mission statement for the project, Vision 2000, is attached for your information.

Over the years, and especially following a major seismic event, such as the recent Northridge Earthquake of January 17, 1994 in Southern California, the public, including owners of industrial facilities, commercial buildings, residential structures, and others have expressed concern about the performance of buildings during earthquakes. Though it is understood by engineers and others that the building codes provide for life/safety as a prime consideration, the extent of damage during the last earthquake left people somewhat confused and upset. With expected damage from a single seismic event reaching into the $30-50 billion range, the public is likely to now focus on the dollar loss as well as life/safety considerations.

Before the advent of modern codes, engineers were forced to think about the performance of buildings during earthquakes based on their observations and experience. They developed concepts and detailing that attempted to address the anticipated effects of the major earthquake ground motions. Their goal was to provide buildings that would perform in a safe manner. Today, in order for the public to form an opinion on "acceptable levels of damage", they must know the cost and options for reducing the losses of the various expected earthquakes.

Vision 2000 will develop a framework for seismic design that will consist of the selection of appropriate systems, layout, proportioning, and detailing for a structure, including non-structural components. Thus, for specified levels of ground motion and within defined levels of reliability the structure's response and performance will be predictable.

When the Vision 2000 framework is fully embodied into structural engineering practice, building owners, for example, would be able to choose the desired seismic performance, including a minimum level that assured life safety, an intermediate level that provides a predictable protection against damage, and a higher level that is aimed at ensuring continued occupancy and functionality of the building.

The purpose of this letter is to solicit your interest in, and support for Vision 2000. We intend for the development of the Vision 2000 guidelines to be an open process that is coordinated with other related efforts.

We would appreciate it if you would complete the attached questionnaire and return it to Ajit S. Virdee, Chairman of the Vision 2000 Liaison Task Group. This can be done by either mail or FAX (the address and phone number are listed on bottom of the form). We intend to compile information, produce a synopsis of other efforts, and develop an active liaison with all interested groups. We will share this information with all those who are involved with and who have an expressed interest in the Vision 2000 project.

Thank you for your assistance and cooperation.

Sincerely,

STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA

Allen Paul Goldstein
Executive Director

VISION 2000 QUESTIONNAIRE

WOULD YOU LIKE TO CONTINUE TO RECEIVE INFORMATION ON SEAOC's VISION 2000 PROGRAM?

YES ________ NO, THANK YOU ________

I AM PARTICULARLY INTERESTED IN THE FOLLOWING ASPECTS OF THE ISSUE:

ARE YOU OR YOUR ORGANIZATION CURRENTLY INVOLVED WITH ANY PROJECTS RELATING TO THE SEISMIC PERFORMANCE OF BUILDINGS. IF SO, WOULD YOU PLEASE DESCRIBE THE PARAMETERS AND PURPOSE OF YOUR PROJECT.

IF YOU HAVE RECENTLY COMPLETED SUCH A PROJECT, WOULD YOU PLEASE FORWARD A COPY OF YOUR WORK PRODUCT OR AN ABSTRACT OF YOUR CONCLUSIONS AND RESULTS.

WHEN CONSIDERING THE SEISMIC PERFORMANCE OF BUILDINGS, FROM YOUR PERSPECTIVE, WHAT ARE YOUR PRIMARY CONCERNS?

IS YOUR ORGANIZATION IN A POSITION TO PROVIDE STAFF, TECHNICAL OR FINANCIAL SUPPORT FOR THE VISION 2000 PROJECT?

DO YOU HAVE ANY SUGGESTIONS AS TO THE MOST EFFECTIVE WAY TO COMMUNICATE THE RESULTS OF OUR PROJECT TO THOSE WHO ARE "END USERS"

SEMINARS ______ WORKSHOPS ______
PUBLICATIONS ______ OTHERS ______

THANK YOU FOR TAKING THE TIME TO RESPOND TO THIS QUESTIONNAIRE

PLEASE RETURN THIS QUESTIONNAIRE BY MAIL OR FAX TO:

AJIT S. VIRDEE
BUEHLER & BUEHLER ASSOCIATES
7300 FOLSOM BOULEVARD, SUITE 103
SACRAMENTO, CA 95826
FAX 916/381-8673

APPENDIX B

DUCTILE STEEL BEAM-COLUMN JOINTS: A DISCUSSION OF PRELIMINARY OBSERVATIONS, CONCLUSIONS, AND RECOMMENDATIONS

DUCTILE STEEL FRAME BEAM-COLUMN JOINTS
A DISCUSSION OF PRELIMINARY OBSERVATIONS, CONCLUSIONS AND RECOMMENDATIONS

Seismology Committee
Structural Engineers Assocation of California

INTRODUCTION

Over 100 steel buildings sustained damage to beam-column joints in ductile steel frames during the 1994 Northridge Earthquake. The precise extent of damage and its causes are not known with certainty, and there is little published technical material discussing the essential issues surrounding this serious problem. The structural engineering profession is being asked to evaluate methods of repairing damaged joints and to evaluate methods of designing reliable joints for ductile steel frames in on-going and future projects. This paper presents a brief summary of the current state of knowledge regarding beam-column joints in ductile steel frames. It includes a discussion of potential mechanisms that might have caused the observed damage, methods being considered to obtain acceptable levels of performance, results of current testing, and suggested policies to guide structural engineers while definitive recommendations are developed.

Ductile steel frames have long been considered one of the premier lateral force resisting systems because they were believed to be ductile, reliable, not overly sensitive to subtle detailing problems, economical, and adaptable to a variety of programmatic uses. The Northridge Earthquake, however, suggests that there are a number of areas in which the performance of these frames is unacceptable.

A large number of steel frame buildings have reported failures in the bolted web- welded flange connections in the seismic moment frames. These failures did not result in any building collapses and in a number of instances, the amount of accompanying minor nonstructural damage was small enough to make the subsequent discovery of extensive connection failure a genuine surprise. Nevertheless, the poor performance of these connections invites careful scrutiny regarding the repair of damaged joints and careful consideration regarding the continued use of bolted web-welded flange connections.

ISSUES

A number of important issues are suggested by the observed damage. This paper attempts to address these issues and indicate what information is available, how reliable that information is, and what kinds of information must be gathered to permit structural engineers to make informed judgements regarding the assessment and solution of the beam/column joint problem.

Observed Damage

1. What type of damage did ductile steel frame beam/column joints sustain?
2. How did the observed damage relate to total building performance?
3. Were there patterns in the observed damage?
4. Was the amount and type of collateral damage (i.e. nonstructural) consistent with the amount of structural damage?
5. Were the damaged buildings in the state of incipient collapse?

Potential Causes

1. What was the cause of the damage?
2. Was there evidence of inferior workmanship or negligent structural design that contributed to the observed damage?
3. What does prior research suggest about the performance of these joints?
4. Did beam/column or welding material properties affect performance?

Repair and Joint Design Recommendations

1. Should existing steel frame buildings be inspected, and if so, how?
2. Can or should the damaged joints be repaired?
3. How can the repairs be effectively accomplished?
4. What are recommendations for buildings currently under construction or in design?
5. What changes need to be made in inspection and structural observation to increase reliability of this connection?
6. What changes should be made to the building code?

Testing

1. What is an acceptable level of performance?
2. Is the current bolted web-welded flange joint capable of achieving these performance levels?
3. How can tests investigate the relevant issues?
4. What are long-term testing issues?

DESCRIPTION OF CONNECTION FAILURES

Connection Description. Accounts of steel frame connection failures have not been widely reported following other earthquakes. The Northridge Earthquake was one of the first earthquakes with high accelerations to affect a heavily urbanized area with a large number of steel frame buildings. The basic requirement for the joint in question is that it be capable of developing the strength of the frame girder, and engineers used a variety of methods to satisfy this objective. One method of accomplishing this is to use fully welded flange and web connections; however, most of the damaged connections appeared to have been designed to comply with prescriptive provisions similar to those found in Section 2211.7.1 of the *1994 Uniform Building Code* (*UBC*) [ICBO]. The prescriptive requirements use complete joint penetration welds between the beam and column flanges and shear resistance based on gravity loads and seismic forces to satisfy the design objective. This bolted web-welded flange connection was already in widespread use by the time it first codified in the *1988 UBC* and *1988 Recommended Lateral Force Requirements and Commentary* [SEAOC].

The original study comparing bolted web-welded flange joints to fully welded joints was reported in the early 1970's [Popov (1972)]. This original research, using W18x50 and W24x76 beams which are quite a bit smaller than those found in most ductile steel frames, showed that while the bolted web-welded flange joints did not perform as well as the fully welded joints, they did develop substantial inelastic deformation. In spite of the erratic performance suggested by this research and subsequent testing programs [Popov (1986), Tsai (1988), Anderson (1991), and Engelhardt (1993)], the conclusions drawn from the initial research, the improved economy of the bolted web-welded flange connection, and the structural engineering profession's increasing reliance on welding lead to widespread use of this connection.

Given the relatively recent discovery of the connection damage and the lack of relevant research and the broad range of joint design issues illuminated by the Northridge Earthquake, there are widely differing opinions regarding the causes, significance, and solutions to the problem posed by these joint failures. Because the data collection, damage evaluation, and joint testing are on-going, any conclusions and recommendations presented herein must be considered tentative. It is recommended that structural engineers endeavor to keep abreast of this developing topic.

Failure Attributes. Based upon a review of available information, it appears that typical joint failures possess one or more of the following general attributes, roughly in the following order of occurrence:

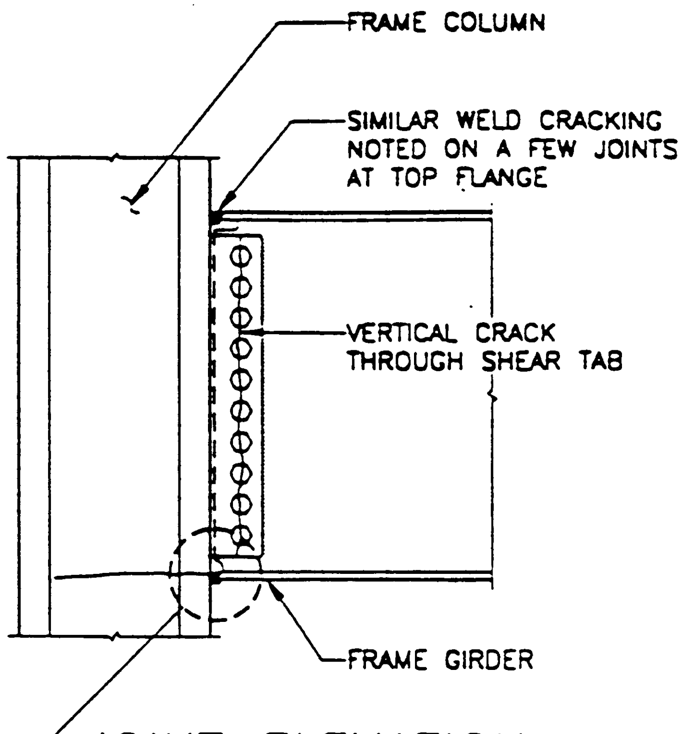
1. Failures at the frame girder bottom flange welds;
2. Divots of steel removed from the face of the column flange;
3. Cracks in the frame column;
4. Cracks in the shear tab or shearing of web bolts;
5. Failures at the frame girder top flange welds; and
6. Horizontal cracks in the beam bottom flange.

Figure 1 schematically illustrates the six basic failure attributes; however, by far the most common problem is failure at the frame girder bottom flange welds. There is evidence that suggests that these failures initiated from the root of the weld and propagated into the filler metal or parent metal. The next most common failure attributes are divots torn from the column flange and horizontal crack across the column flange and, in many instances, across the column web. Failures at the shear tab have included vertical cracking of the tab leaving the bolts intact as well as shear tab connections that have lost over 50 percent of the bolts in shear.

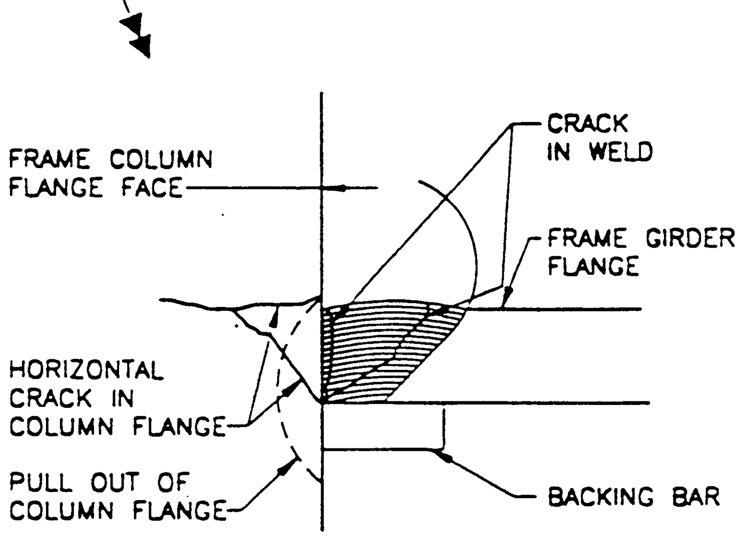
The range of building characteristics in structures reporting connection failures has been extremely broad and it has been difficult to develop recurring patterns that predict where one will find damaged connections. This makes it difficult to establish where to inspect buildings based on engineering analysis or intuition. Building location has ranged widely, from near the epicentral area in Northridge, Woodland Hills, and Sherman Oaks, north to the Santa Clarita Valley, east to North Hollywood, Burbank and the Cahuenga Pass, and to the south and west in Santa Monica and West Los Angeles. Most of the damaged buildings are found in the epicentral region These frame connection failures have been noted in buildings of as little as one story to as many as 27 stories. In the shorter buildings, the joint failures have been evenly distributed throughout the building height. Some investigators have reported evidence suggesting that in the taller buildings most of the failures have occurred in the upper half to two-thirds of the building; however, there has not been a systematic investigation of this behavior.

Nearly identical structures located adjacent to each other have reported widely varying rates of connection failure. Of those buildings reporting connection failures, some buildings have reported failure rates of less than 10% while others have reported failure rates as high as almost 100% over a large number of floors and/or in specific compass directions. Many of the reported failures, however, appear to consist of small cracks at the root of the weld between the beam and column flange. Failures have occurred in connections with and without column flange stiffeners as well as connections with and without return welds on the shear tabs. Both wide flange columns and built-up box sections appear to have been affected. Failures have been noted in buildings with a relatively small number of frame bays in each direction as well as in highly redundant buildings where nearly every girder line is part of a moment frame. It is interesting to note that the age of the affected buildings appears to be fairly low, with most buildings having been completed in the 1980's. This observation, however, coincides with the era during which a majority of steel frame buildings in the affected areas were constructed. Work is ongoing to establish the relative frequency of these characteristics and to determine if there is significant correlation between any of them.

Welding and Materials. From the perspective of welding materials and technology, it appears that most, if not all, of these frames were theoretically constructed using welding procedures and materials sanctioned by Structural Welding Code of the American Welding Society (ANSI/AWS D1.1). As will be discussed in greater detail later, deviations from the requirements of AWS D1.1 have been noted in the buildings examined, although not always in conjunction with failed connections. Most of the recently completed buildings or the buildings currently under construction were field welded using the Flux Cored Arc Welding (FCAW) process; it is not known if this was the case in the older



JOINT ELEVATION



DIFFERENT OBSERVED JOINT FAILURES

CONNECTION FAILURE ATTRIBUTES FIGURE 1

Northridge Earthquake - Damage to Ductile Steel Frames 1994 buildings affected. Some fabricators used Shielded Metal Arc Welding for shop welds; however, it is not likely that any impact of the different welding methods can be determined by examining the damaged welds without careful review of filler metal metallurgy or physical properties. Electrode diameters of up to 0.120 in. were in common use and limited metallurgical testing indicates that the filler metal did not possess particularly high impact toughness; however, it is believed that the filler metal used in these joints was not required to meet minimum toughness requirements. AWS D1.1 does not contain minimum toughness requirements for filler metal.

Steel complying with ASTM A36 and, infrequently, ASTM A572 Grade 50 was used for frame beams, while many, but not all, columns appear to have been of Grade 50 stock. It has been argued that accidental overstrengths have resulted in beams with actual flexural strengths greater than those of the columns; however, there is no systematic evidence to suggest that this undesirable strength imbalance was present in a significant number of damaged connections. Nevertheless, it should be noted that the mean yield strength of A36 steel has been increasing in recent years and now averages approximately 49 ksi. This increased strength can significantly increase the demand on the weld as well as columns in which the yield strength is only slightly above 50 ksi. Limited toughness testing of steel from damaged connections suggests that Charpy V-Notch toughness, although not a requirement for most of the sections examined, seldom exceeded values considered the lower limits of acceptable toughness.

Possible Causes of Connection Failures

Initial responses to reports of ductile frame connection failures ranged from genuine surprise to an intuitive belief that these failures were simply examples of poor welding quality being exposed by the earthquake. As more and more cases were reported, a systematic search for possible causes was begun. The preliminary results of this effort suggest a number of possible contributors to beam-column connection failure, none of which has emerged as a completely reliable predictor of poor performance.

Recent tests of welded flange-bolted web ductile frame connections [Engelhardt (1993)] and a re-examination of similar tests stretching as far back as 1972, suggest that these welded connections show a large variability in performance, maximum attainable plastic rotation, and weld failure rates. The observed post-earthquake condition of the failed connections appears to mirror the results of the least reliable connections in almost every aspect except for the horizontal cracks in the columns, which do not appear to have been widely reported in the literature.

There appear to have been few unequivocal concerns voiced prior to the Northridge Earthquake about the reliability of the connection. Since the earthquake, a number of potential problems inherent in the connection detailing, the welding materials and procedures used to execute the connection, and the advisability of a current design practice favoring a smaller number of high capacity frames have been expressed. These suggested causes can be summarized as follows:

1. Excessive stresses and high strains in the complete joint penetration (CJP) groove welds between the beam and column flanges;

2. Poor workmanship and inspection in executing the CJP welds;

3. Welds of inadequate strength because of:
   a. Welding performed contrary to an appropriate Welding Procedure Specification (e.g. voltage, amperage, and wire feed speeds beyond the ranges recommended by the manufacturer or poor techniques in violation of specific AWS provisions)
   b. The use of high deposition rate, large diameter flux cored electrodes;
   c. Insufficient fusion between the filler metal and parent metal, particularly at the face of the column flange.

4. Difficulties in executing the bottom flange CJP weld through the restriction created by the web access hole;

5. Presence of a vertical stress riser caused by leaving the partially fused back-up bar in place after welding (although removal of back-up bars does not appear to have been a requirement in any of the projects examined);

6. Lack of column stiffeners (continuity plates);

7. Misalignment of column stiffeners and use of fillet welds in lieu of CJP welds at the column flange;

8. Material related deficiencies including lack of toughness of the filler and/or parent metal and through-thickness (Z-axis) strength of the column flange;

9. Residual stress concentrations caused by the welding sequence of highly restrained joints;

10. Earthquake loading rate effects;

11. A lack of redundancy because of the trend toward lateral systems consisting of relatively few frames with deep girders and stiff columns.

Demand on the CJP Weld. There is strong evidence, based on recent tests [Engelhardt (1993, 1994)], that high stresses and strains in the area of the CJP weld contribute to the vulnerability of the joint. Since it is generally assumed that the beam flanges transfer the majority of the moment to the column, although M_p is based on the full plastic section modulus including the beam web, developing the full value of M_p can generate stresses in the flange welds that approach or exceed F_u. Although it is unclear as to the precise distribution of stress and strain in a flexural cross section and the participation of the web at all stages of loading, calculations based on a simple tension-compression couple acting about the centroid of the flanges give some indication of the added demand placed upon the flanges in developing the entire plastic moment. These stresses can easily exceed 1.5 to 2.0 times the specified nominal yield stress and will undoubtedly be even higher as the ultimate tensile strength of the section is achieved through strain hardening. Since it is assumed that the filler metal is stronger than the specified strength of the parent metal, it is of interest when high stresses are developed in the regions adjacent to the CJP welds.

Workmanship. Poor workmanship is often cited as a significant contributor to poor joint performance noted following the Northridge Earthquake. Ongoing testing suggests that strict adherence to an approved Welding Procedure Specification (WPS) is one essential ingredient required to producing a reliable weld. A WPS is required by AWS D1.1, but it is clear from examination of *in situ* welds and discussions with fabricators, erectors, and inspectors that even cursory consideration of the precise requirements of the WPS was an extremely rare occurrence. In addition, violations of specific AWS code provisions and poor welding technique were encountered in many cases.

As discussed in the preceding section, examination of some failed welds revealed the presence of slag inclusions all along the root of the weld, which reduces the effective area of the weld. The slag inclusions increased in size from the flange tip inward towards the web cope, possibly because of the relative degree of difficulty the welder experienced in continuing the weld to the other side of the member through the web. Where the welding was done from the edge of the flange toward the web, the welder was forced to turn the electrode and push filler metal through the cope hole trapping slag and reducing fusion in this region. In some instances, large inclusions were found at the flange edges where improper run-off tabs were used. These welds are highly stressed and demand particular care during execution to ensure that the welds have no material flaws.

Welding Parameters. The debate regarding the role played by high deposition rate, large diameter flux cored electrodes has continued since the connection failures were first discovered. Some have argued that the large diameter electrodes, approximately 0.120 in. in diameter, encourage the placement of filler metal at rates that prevent adequate fusion of the filler and parent metals. In addition, the large electrode diameter makes it difficult to penetrate to the root of weld as well as causing difficulties in executing the weld in the vicinity of the beam web if the access hole is limited. As discussed in the next section, both excellent and unsatisfactory performance has been achieved with large diameter electrodes. There does not yet appear to be a convincing body of knowledge regarding the influence of these electrodes.

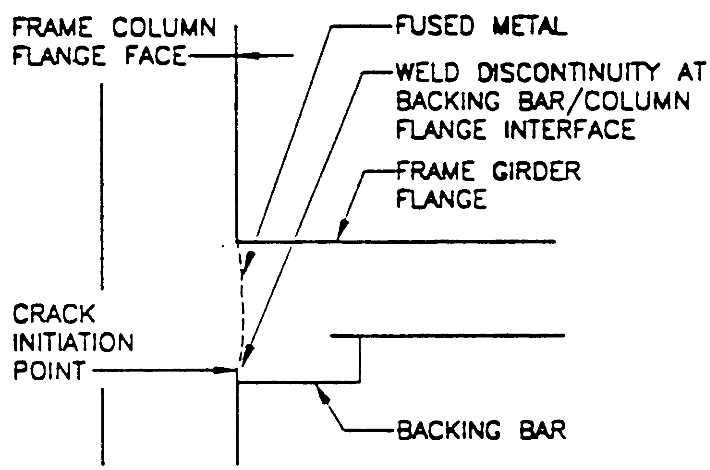
A similar debate has centered on the lack of toughness in the filler metal. Although not typically required of projects constructed using AWS D1.1, there are advocates suggesting that filler metal meeting a minimum standard of Charpy V-notch toughness will resist crack propagation more effectively than filler metal not meeting this minimum standard. To date, it is not known if tougher filler metals would have prevented these failures, although engineering judgement would suggest that toughness rated filler metals in welds with inherent discontinuities might improve performance and overall joint ductility. Current testing is unlikely to demonstrate the advantages of tougher filler metal because the loading rates are so low. Nevertheless, there appears to be no known disadvantage to using filler metal with better toughness properties than typically provided other than a currently unknown economic penalty.

Welding Inspection. Special inspection of the welds using visual and ultrasonic testing was commonplace on most, if not all, of the projects experiencing connection failures. This does not suggest, however, that all welds were ultrasonically inspected because commonly applied inspection requirements specify that only 25 percent of the welds be inspected ultrasonically once a given number of welds have been executed. There were instances of questionable welding practices related to the run-off tabs (e.g. use of run-off tabs to create a "dam" perpendicular to the direction of the weld bead) and elimination of shear tab return welds that were shown on the construction documents. Nevertheless, weld failures were reported on projects that are known to have been constructed in accordance with the general standard of practice, and widespread, faulty fabrication does not appear to be a primary cause of the joint failures. There is also no evidence suggesting that negligent structural design was a significant factor in the widely distributed joint damage.

Preheating and Residual Stresses. Another contributing factor that has been suggested relates to the difficulty in executing the bottom flange weld while maintaining proper weld interpass temperatures and evenly distributed cooling. The bottom flange welds are usually executed on one side of the web and then the other, giving rise to the possibility that differential cooling may lock in stresses. Examination of a few damaged welds revealed that only half of the bottom flange weld has cracked. In addition, some welds appear to have been cracked prior to the earthquake. These cracks have been identified through the presence of rust in the weld crack. The highly restrained character of the joint and the associated residual stresses have also been suggested as a contributing factor. On-going tests have used joints that are far less restrained than found in most buildings while still reproducing failures similar to those observed following the earthquake.

Column Stiffeners. Other factors such as flexible column flanges or flange plates in welded box columns have also been suggested as contributing causes to the poor performance of the welded joints. The limited testing conducted to date has shown improved joint ductility for smaller, thinner sections when column stiffeners were used. The most recent post-Northridge Earthquake tests have not substantiated that a lack of column stiffeners was or was not a factor in the observed joint failure, and a number of damaged connections using column stiffeners were found. It should be noted that the columns involved in this testing possessed very thick flanges that did not appear to need column stiffeners. Column flange flexibility could initiate premature failure of the weld due to unanticipated deformation of the column flange and concentration of stresses at the stiffer section of the weld opposite the column web.

Backing Bar. At least at the bottom flange, the presence of the backing bar in the final connection may encourage the initiation of a crack at levels of stress and strain that, taken for the section as a whole, are less than those required to develop significant plastic rotation in the section. Figure 2 illustrates the areas of fused metal when the backing bar is left in place and one can see the presence of a potential stress riser at the face of the backing bar and column flange below the area that is fused by the initial welding passes. Fractographic examination of some failed welds suggests that crack



SCHEMATIC OF FUSED METAL
AT FULL PENETRATION WELD

FUSION AND STRESS RISER AT BACKUP BAR

FIGURE 2

Northridge Earthquake - Damage to Ductile Steel Frames

1994

propagation initiated at this point. The tendency of the stress riser to initiate the propagation of a crack may be aggravated because of lack of fusion at the weld root and the presence of the included slag, weld execution problems caused by interference at the web, and the presence of built-in stresses due to the two step process used to weld the bottom flange. At this time, there has been no thorough examination of these aspects of the weld execution.

It has been noted that far fewer failures at the top flange weld have been observed. At the top flange, it can be seen that the most highly stressed area in tension is located on the opposite side of the flange from the backing bar and the stress riser is not significant. Further, the presence of a slab may assist the top flange by increasing the effective compression area through composite action and thereby exacerbate the demand bottom flange. Other explanations for the smaller number of top flange connection failures may also be related to higher quality welds because of easier welding condition. There has not yet been a systematic examination of these potential causes.

Despite the preceding observations, however, performance of different test specimens in a recently completed testing program [Engelhardt (1994)] does not resolve the issue of whether the backup bar is a significant factor in the observed performance. The fact that both bottom and top flange failures have been noted, even when the backup bars have been removed, further clouds the issue.

Column Cracks. The various manifestations of the crack (e.g. horizontal cracks through the column, vertical cracks in the weld, or divots removed from the column flange) may result from the set of unique conditions present in each connection, such as the presence of column tension, additional inclusions within the weld, and the like. Past and current research does not clearly describe the parameters and fracture mechanics that have lead to the different observed failures. Future research should consider the effects of column size, cyclic axial stress, and the like.

TESTING PROGRAM TO EXAMINE REPAIR AND NEW DESIGN CONCEPTS

The joints that were damaged by the earthquake must be repaired at some point, and details leading to improved performance of joints in new steel frame construction must be developed. In light of the fact that the cause of the connection failures is not known with precision, the most effective repair strategy also is uncertain. For this reason, a modest testing program sponsored by the American Institute of Steel Construction and a private-sector client was developed [Engelhardt (1994)]. The program replicates beam and column geometries found in one particular project and may not be representative of members found on other projects; nevertheless, the test results from this program represent the bulk of careful post-earthquake research available at this time.

In the discussion that follows, and despite the AISC testing program, it will be seen that there is limited data upon which one can confidently base a decision. Nevertheless, structural engineers are being called upon to recommend both repair details and revisions to moment frame connection details for new building construction. It is hoped that the information from this program will serve these short-term needs while a more comprehensive testing program is developed and the results published.

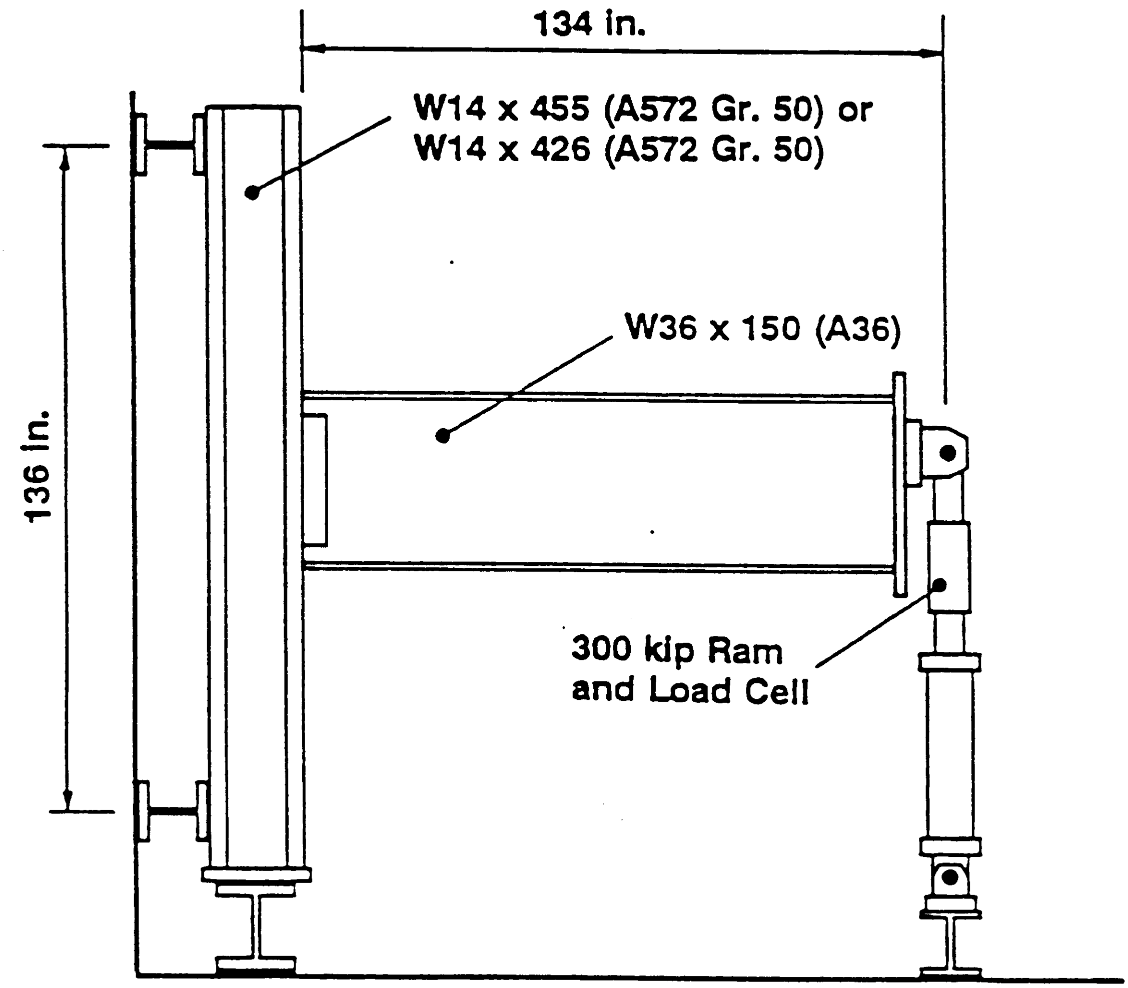
Testing Setup. The goal of the AISC testing program was to conduct an *immediate* series of large scale tests to develop preliminary guidelines for improved beam-column connections. The test setup, shown in Figure 3, uses some of the largest beam-column sections ever tested to better simulate the geometry of frames found in buildings. Design of the specimens is intended to limit the participation of the column, thereby concentrating deformation in the beam. A series of slowly applied cyclic loads is used to simulate the dynamic effects of an earthquake. Each loading step is repeated three times. The use of this loading protocol and the very stiff column which concentrates the plastic deformation in the beam may place greater demands on the beam-column connection than would occur in most earthquakes assuming the connection has the ability to achieve some measure of plastic rotation. On the other hand, the slowly applied load and the lack of column axial load ignores the effects of loading rate present in an actual earthquake and the lack of restraint in the test setup may result in test conditions more forgiving than those found in actual buildings. On the whole, it is hoped that the tests are representative of *in situ* conditions.

Test Series I Characteristics. The test specimens consist of two test series, Series I and Series II. Series I, consisting of four pairs of different connection geometries, represented the first attempts to investigate connection behavior. Series II, consisting of an additional five pairs of connections, reflected some of the lessons learned in Series I. There were two replicates of each specimen, each executed by one of two fabricators. Figures 4 through 9 present the different specimens.

Each specimen consisted of a W36 x 150, ASTM A36 beam and a W14x455, ASTM A572 Grade 50 column. Supplemental shear plate welds were used to transfer 20 percent of the web moment because $Z_f/Z < 0.7$. Field welders were used to execute the "field" welds and shop welders executed the "shop" welds. Visual and ultrasonic inspection was performed in accordance with AWS D1.1 The CJP welds were executed using flux cored arc welding and 0.120 diameter E70T-4 electrodes for flat position welds and 0.072 in. diameter E71T-8 electrodes for all other positions. The E70T-4 electrode does not have to satisfy impact toughness requirements.

One set of specimens in Series I was executed in strict accordance with an approved welding procedure specification (Test Series I "B" specimens) while in the other series the welders were permitted perform their welds in a manner that they believed would produce the best quality weld (Test Series I "A" Specimens). It will be seen that there is a significantly higher level of performance in the Test Series I "B" specimens over the Test Series I "A" specimens. In Test Series II, both fabricators followed the same welding procedure specification and there was no discernable difference in performance.

All of the backup bars were removed, except for at the top flange of Specimen 3, which involved gouging out of the weld root and reinforcement of the weld. This action removes the discontinuity in the weld that many believe leads to the premature failure of the connection. This practice is recommended in the Bridge Welding Code (AWS D1.5), although the basis for this recommendation is founded in concerns related to fatigue. There has been little or no testing to confirm if the backing bar is indeed a problem under the large deformations associated with seismic loads nor has there been any testing to show that the removal of the backing bar yields significant improvements in seismic



TEST SETUP - FIGURE

3

performance. All run-off tabs also were removed. No column stiffeners were used. Table 1 summarizes the characteristics of the specimens used in Test Series I.

Table 1
Test Series I Specimen Characteristics

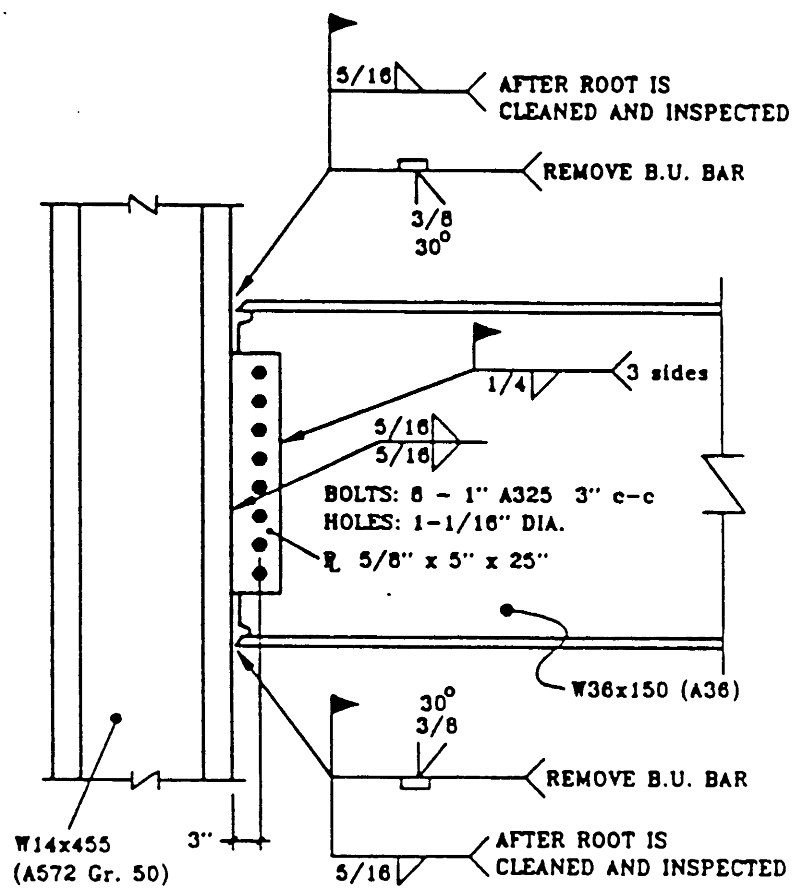
Specimen Number	Figure Number	Top Flange Reinforcement	Bottom Flange Reinforcement	Web
1	4	None	None	Bolted
2	5	None	None	Welded
3	6	Triangular	Rectangular	Bolted
4	Not Shown	None	Flat	Bolted

Specimen 1 modeled the typical frame connection found in most of the damaged buildings and consisted of a bolted web and welded flanges as shown in Figure 4.

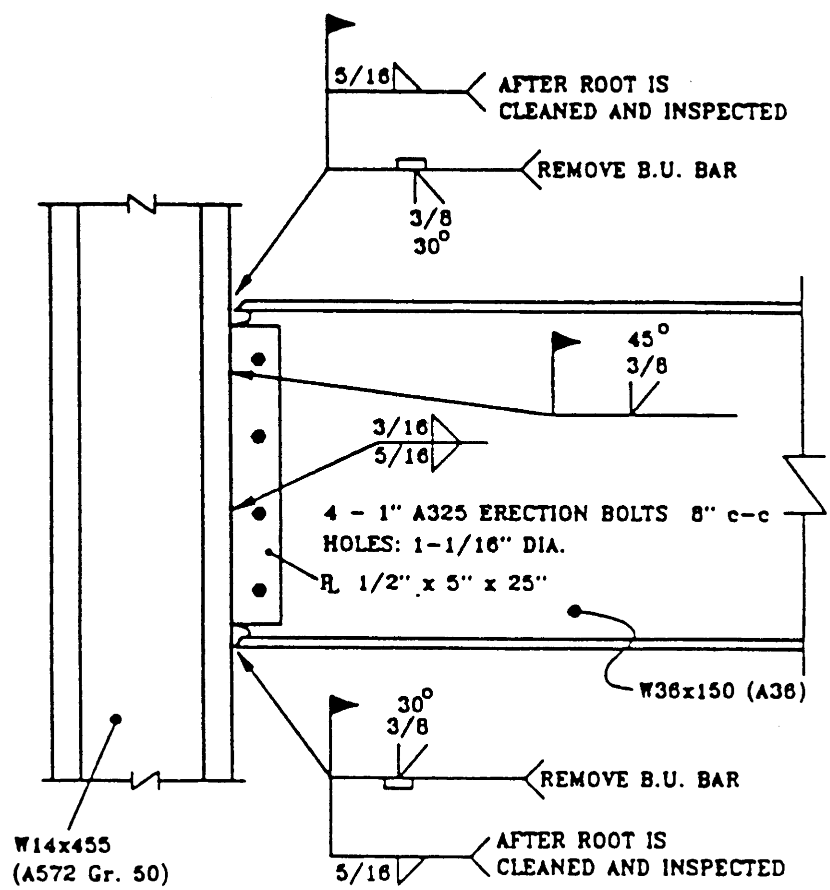
One suggestion to solve the problem of welded flange-bolted web connections is to return to a fully welded connection (welded flange-welded web). The steel fabrication industry originally encouraged use of the bolted web connection to lower fabrication costs by reducing the amount of field welding. The fully welded web solution offers the potential to reduce the effective stress in the flange by permitting transfer of moment to occur through the web as the section fully plastifies. Specimen 2 was intended to test this assumption and is described in Figure 5.

Specimen 3 (Figure 6), the first of the reinforced connections, a tapered top coverplate and rectangular bottom coverplate were used to shift the location of the plastic hinge from the face of the column inward into the span and to reduce the stress in the CJP weld. The tapered shape is to avoid stress concentrations at the termination of the plate. The top and bottom plate geometries permitted all of the welding to be performed downhand. The bottom plate weld was performed in the "shop," and this plate served as the backup bar for the bottom flange weld performed in the "field." The top plate was welded to the beam flange in the "shop" and the beam flange/top coverplate weld was executed as one weld in the "field." The coverplates were designed to increase the effective weld area by approximately 75%.

Similar to Specimen 3 in basic geometry, the top plate was omitted on Specimen 4 in the belief that it might be possible to simply reinforce the bottom flange because that was where a majority of the observed failures occurred. Specimen 4 was not tested because some of the top flanges of Specimens 1, 2, and 3 failed in the earlier tests. Others are currently testing details similar to this concept but with a haunch at the bottom flange built from a WT section.



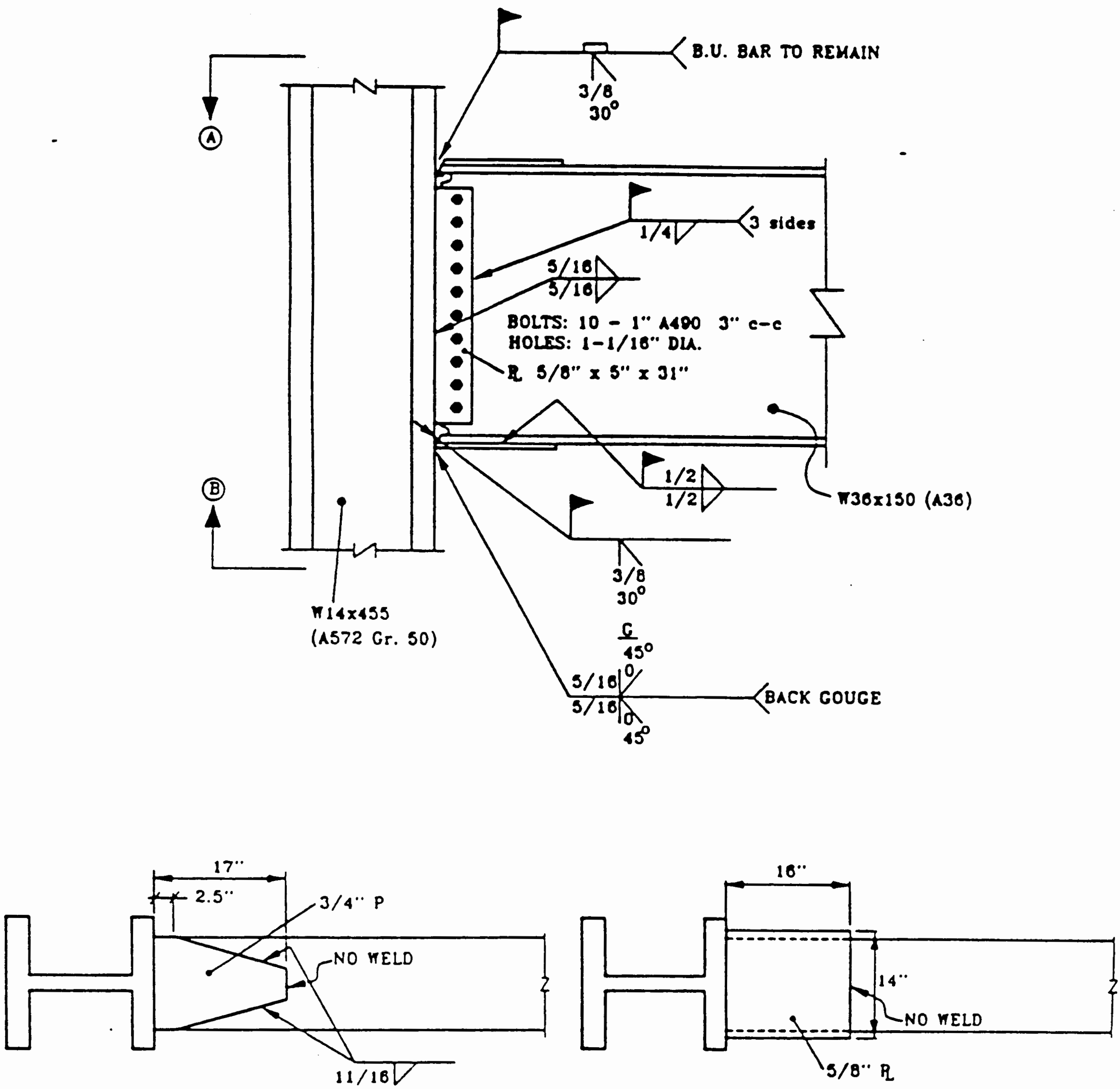
SPECIMEN 1 - FIGURE 4



SPECIMEN 2

FIGURE

5



SECTION A

SECTION B

SPECIMEN 3

FIGURE

6

Test Series II Characteristics. Test Series II was similar in most respects to Test Series I except that W14x426 columns and column stiffeners were used. In addition, more rigorous efforts were undertaken to develop and enforce an approved welding procedure specification. The other significant difference is that Specimens 5 and 6 were welded using 7/64 in. diameter E70TG-K2 electrodes and Specimens 7 and 8 were welded using 7/64 in. diameter E70T-7 electrodes for the flat position welds (CJP welds). These filler metals are capable of delivering a minimum of 20 ft.-lbs. at -20°F as measured by a standard Charpy V-notch test. Table 2 summarizes the characteristics of the specimens used in Test Series II.

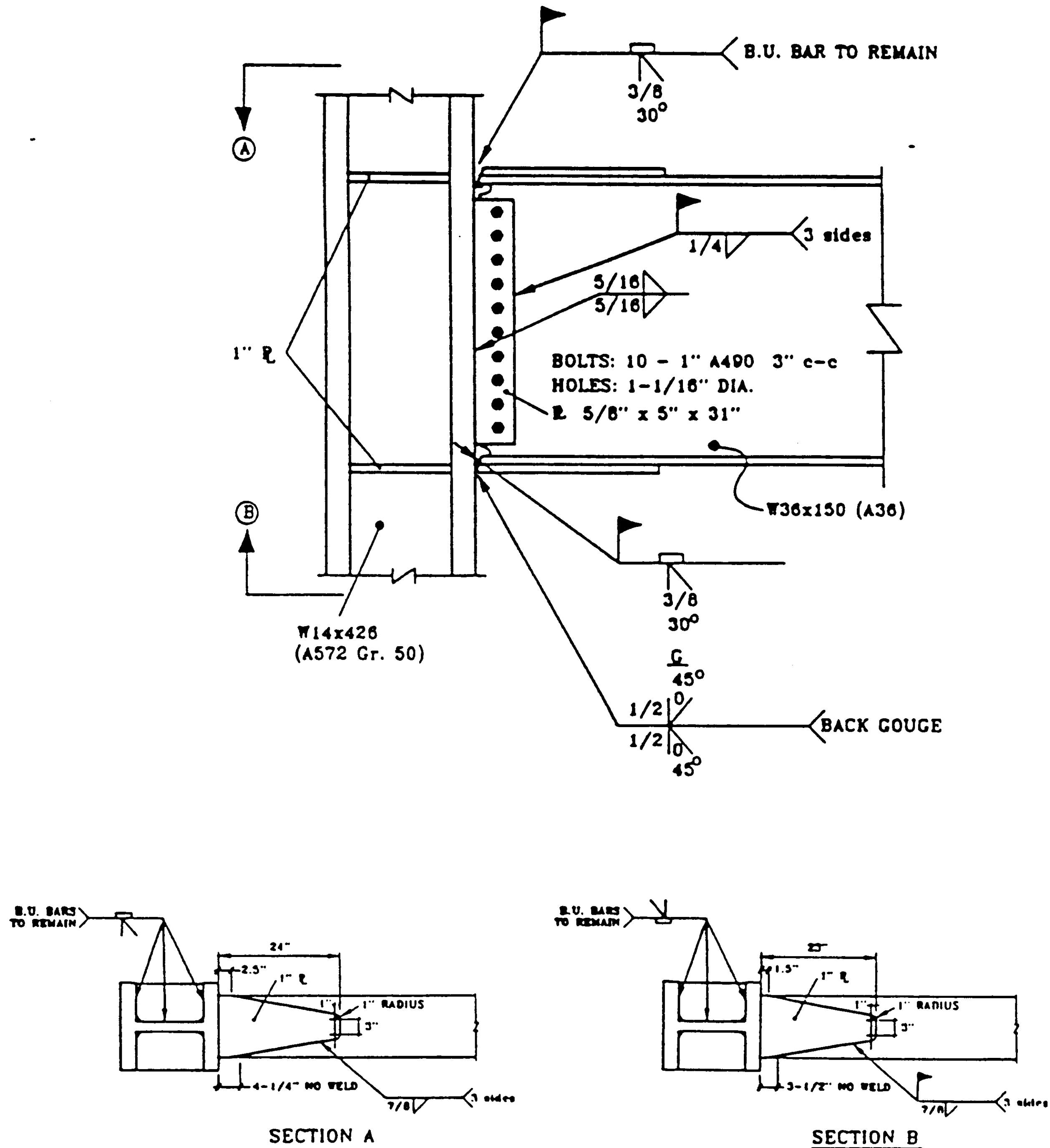
Table 2
Test Series II Specimen Characteristics

Specimen Number	Figure Number	Top Flange Reinforcement	Bottom Flange Reinforcement	Web
5	7	Triangular	Triangular	Bolted
6	8	Upstanding Rib	Upstanding Rib	Bolted
7	9	Triangular	Rectangular	Bolted
8	9	Triangular	Rectangular	Bolted

Specimen 5, shown in Figure 7, is similar to Specimen 3 except that the bottom flange is reinforced with a triangular coverplate. The plates have increased the effective flange area by approximately 100%. These plates were approximately 50% longer than those used in Specimens 3, 7, and 8 and were sized to enable the surrounding fillet welds to fully develop the plastic capacity of the coverplate. In addition, the welds at the tip of the coverplate were returned to assist in reducing the effects of shear lag. Based on observations from Specimen 3, the fillet weld between the coverplates and the CJP flange welds were not allowed to intersect.

Upstanding triangular ribs were used on the top and bottom flanges of Specimen 6, as shown in Figure 8, and were sized to increase the effective area of the flange by approximately 50%. Two rib plates were used to spread out the applied load and to provide some measure of redundancy in the reinforcement should the CJP welds of one rib fail. Sufficient room must be allowed between the ribs to permit the fillet welds to be executed. A stress relief hole was provided on the bottom corner of the top ribs to prevent the CJP welds between the ribs and the beam flange from intersecting.

Specimens 7 and 8, as shown in Figure 9, are identical to Specimen 3 except for the elimination of the intersecting welds at the column flange face and the use of filler metal supplying a minimum level of toughness.

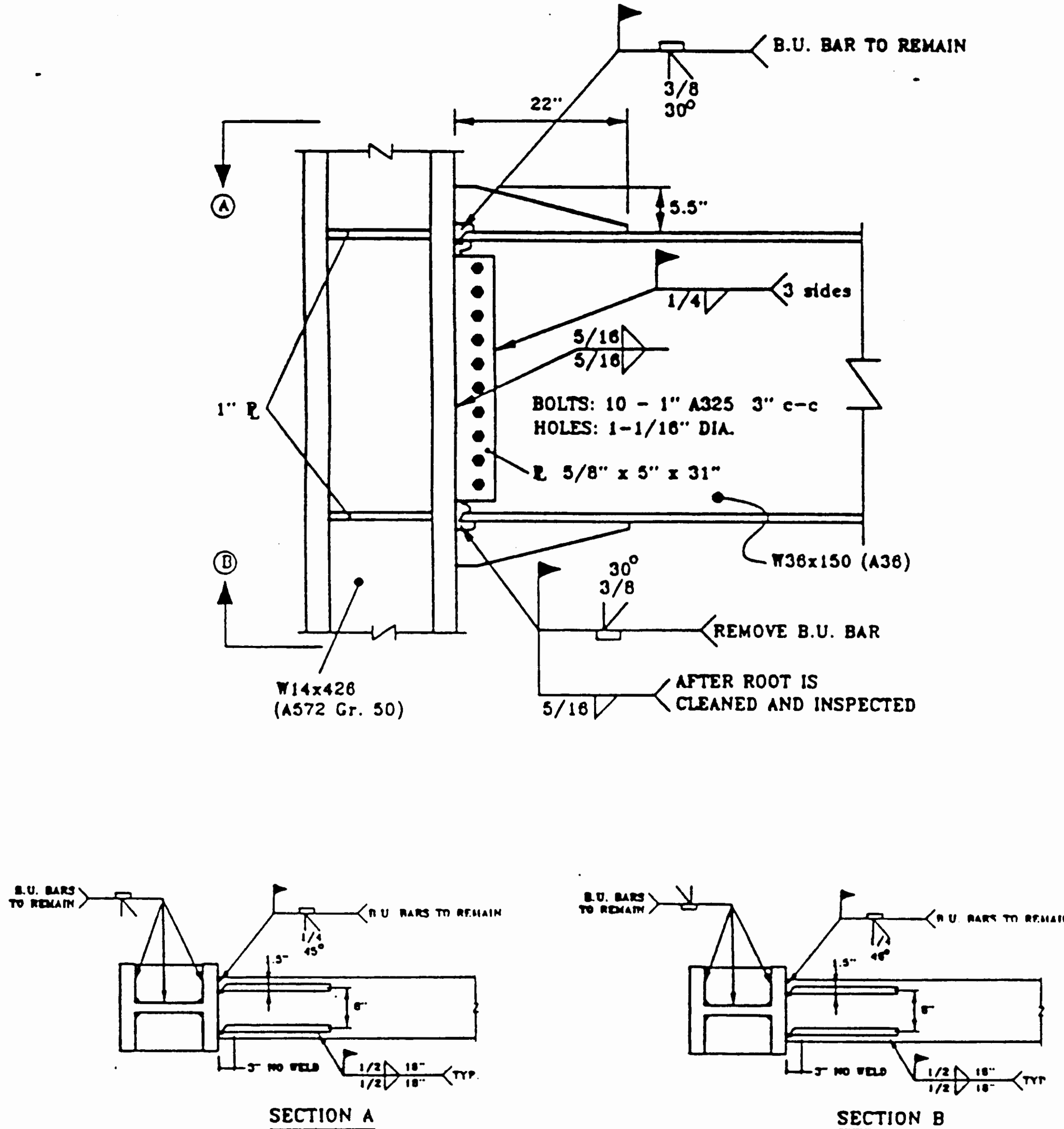


SPECIMEN 5

FIGURE 7

Northridge Earthquake - Damage to Ductile Steel Frames

1994



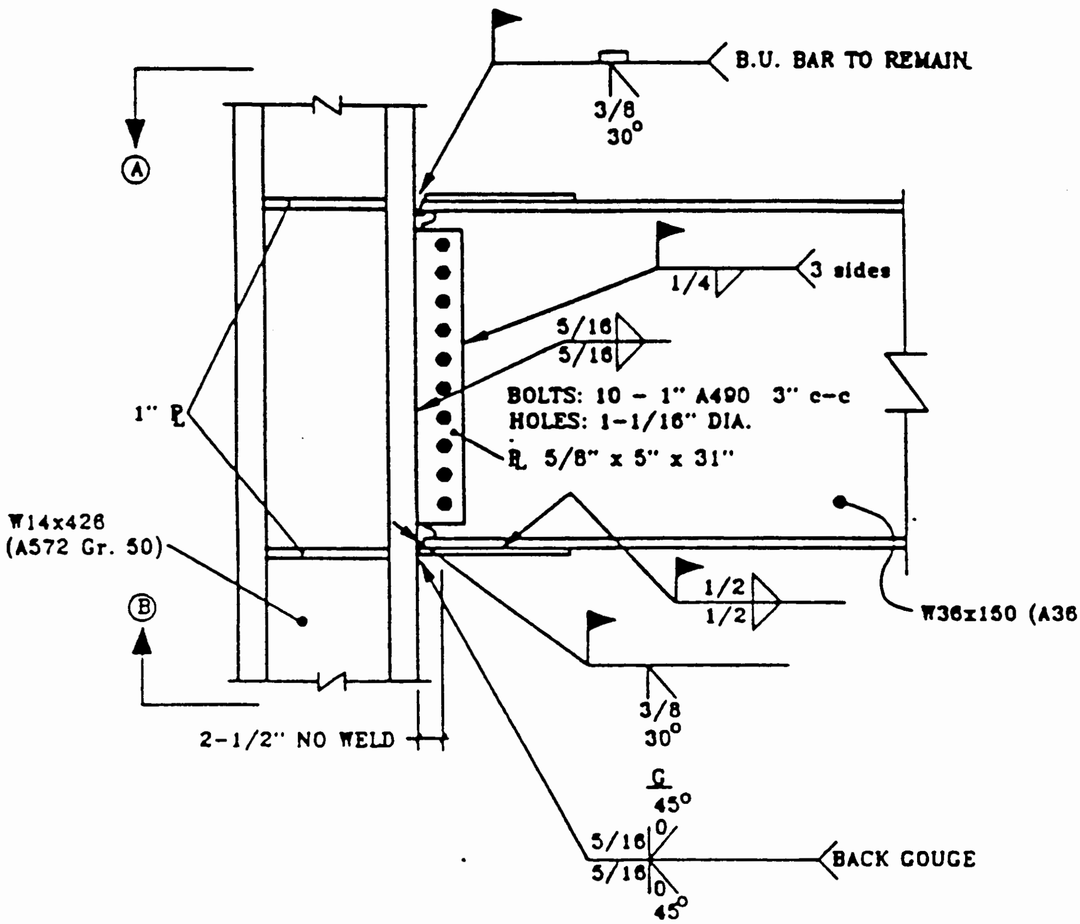
SPECIMEN 6

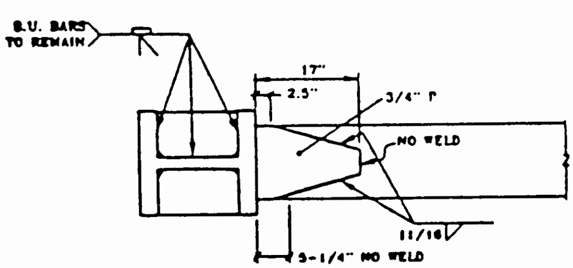
FIGURE

8

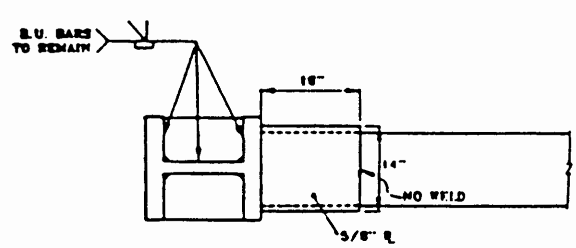
Northridge Earthquake - Damage to Ductile Steel Frames

1994





SECTION A



SECTION B

SPECIMENS 7 AND 8

FIGURE

9

Northridge Earthquake - Damage to Ductile Steel Frames

1994

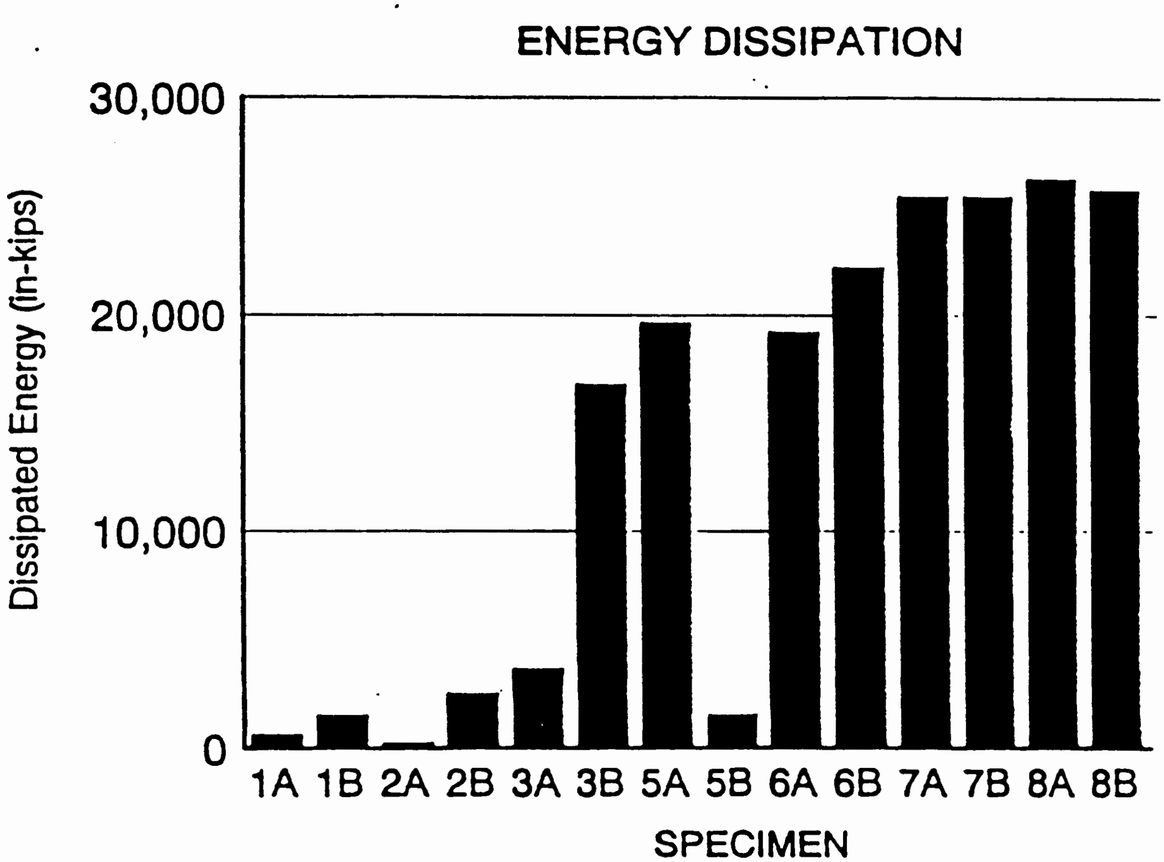
Test Results. The number of inelastic cycles was determined by counting the number of cycles at each displacement level that the specimen was able to achieve without failure. Three complete cycles of loading were performed at each displacement level. There is a significant difference in the amount energy absorbed as additional loading steps are achieved. Figure 10 plots the approximate amount of energy absorbed during the inelastic deformations to more clearly illustrate the additional energy absorbed by the more effective beam/column connections. The maximum plastic rotation describes that achieved by the specimen before failure even if the full three cycles at that displacement level were not completed. Overall performance assessment was assigned based on the general criteria summarized in Table 3.

Table 3
Approximate Assessment of Overall Performance

Assessment	Ductile Failure	Maximum Plastic Rotation
Very Poor	No	Less than one inelastic cycle at 0.0075 radians
Poor	No	Any amount greater than 0.0075 radians
Satisfactory	Yes	Approximately 75% of maximum strength maintained for at least one inelastic cycle at 0.02 radians
Very Good	Yes	Approximately 75% of maximum strength maintained for at least than one inelastic cycle at 0.025 radians
Excellent	Yes	Approximately 75% of maximum strength maintained for more than one inelastic cycle at 0.03 radians

A review of available test results indicates that the conventional and fully welded joints, Specimens 1 and 2, shown in Figure 11, are not capable of achieving the required level of plastic rotation without brittle fracture. Their performance was judged to be poor to unsatisfactory. This suggests that the prescriptive provisions in Section 2711.7.2 of the first and second printing of the *1994 UBC* and Section 8.2c of *AISC Seismic Provisions for Structural Steel Buildings* do not produce a joint capable of achieving acceptable performance based on the tested beam and column sizes as well as the many similar failures observed following the Northridge Earthquake.

Specimens 3A, shown in Figure 12, suffered an early fracture of the bottom beam flange but the bottom coverplate did not fail. The crack from the fractured bottom weld intersected the fillet weld between the bottom flange and the bottom coverplate. Little strength was lost due to the fracture of the bottom flange weld and significant additional plastic rotation was developed while the crack



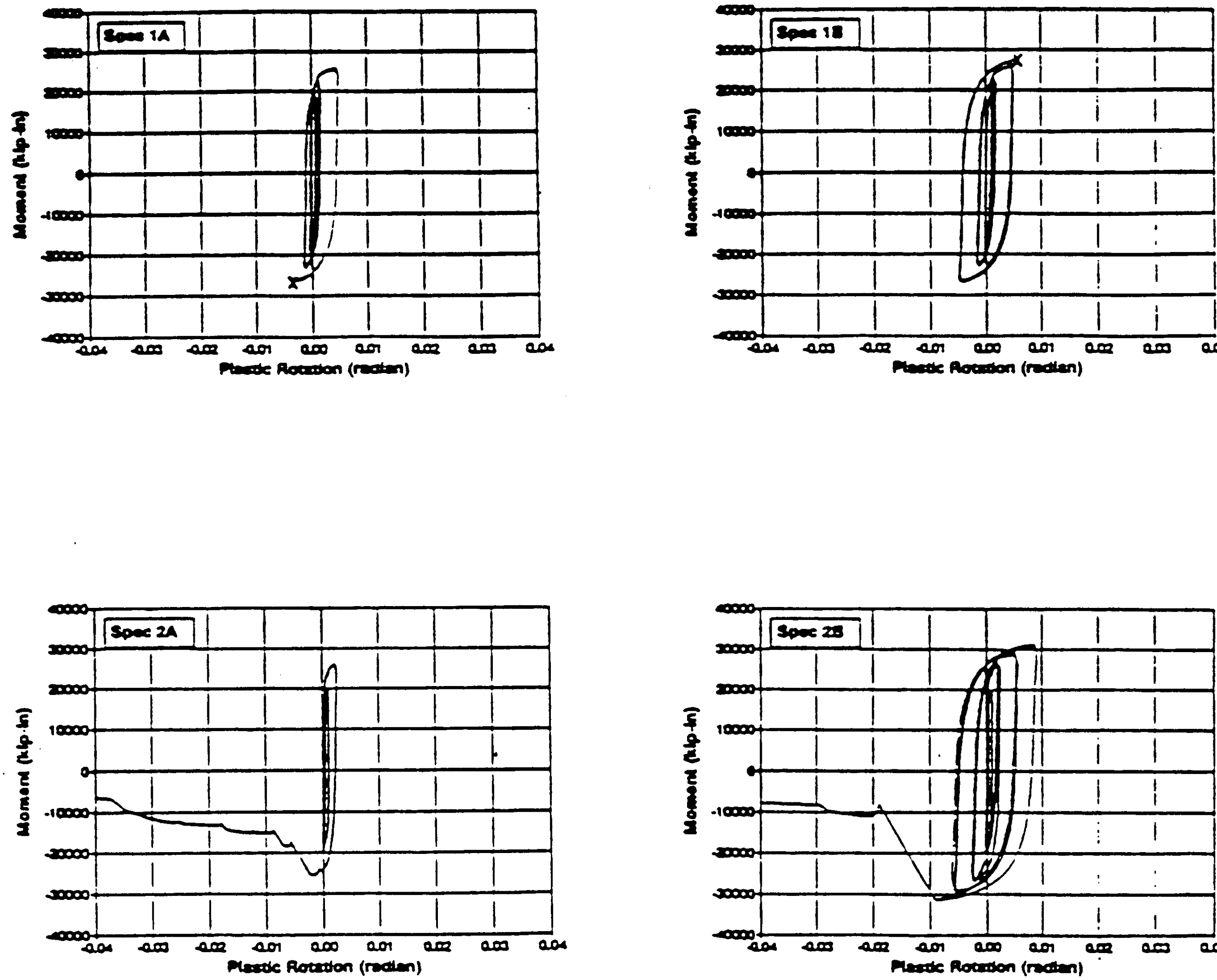
ENERGY DISSIPATION PLOT -

FIGURE

10

Northridge Earthquake -
Damage to Ductile Steel Frames

1994



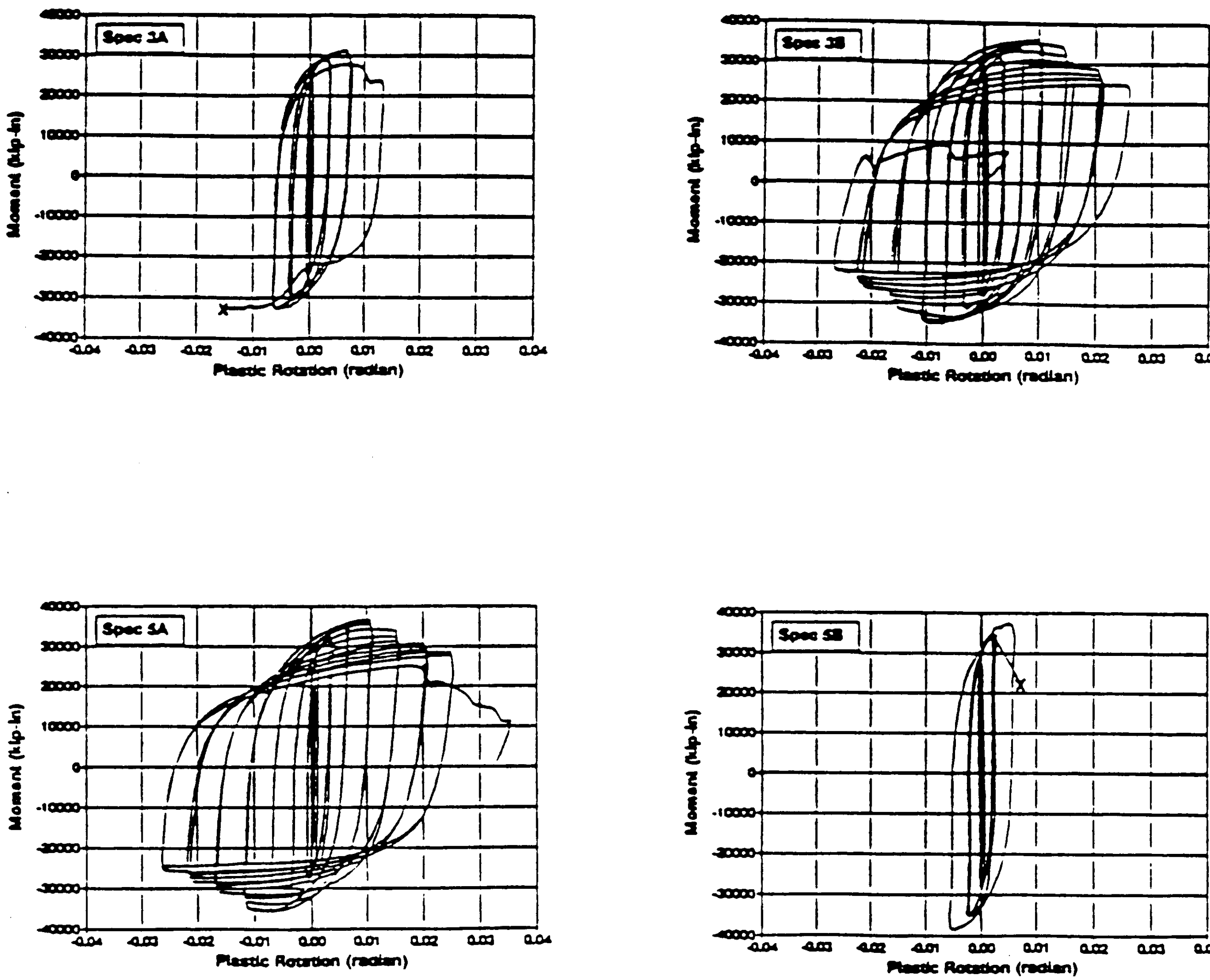
MOMENT ROTATION CURVES-
SPECIMENS 1 AND 2

FIGURE

11

Northridge Earthquake -
Damage to Ductile Steel Frames

1994



MOMENT ROTATION CURVES - SPECIMENS 3 AND 5

FIGURE 12

propagated along the length of the coverplate weld. The plastic hinge appeared to form near the end of the coverplate as hoped, thereby shifting the highly strained area away from the face of the column flange and the CJP weld. The section was performing very satisfactorily until a brittle fracture occurred at the top flange resulting in the overall performance assessment of "unsatisfactory."

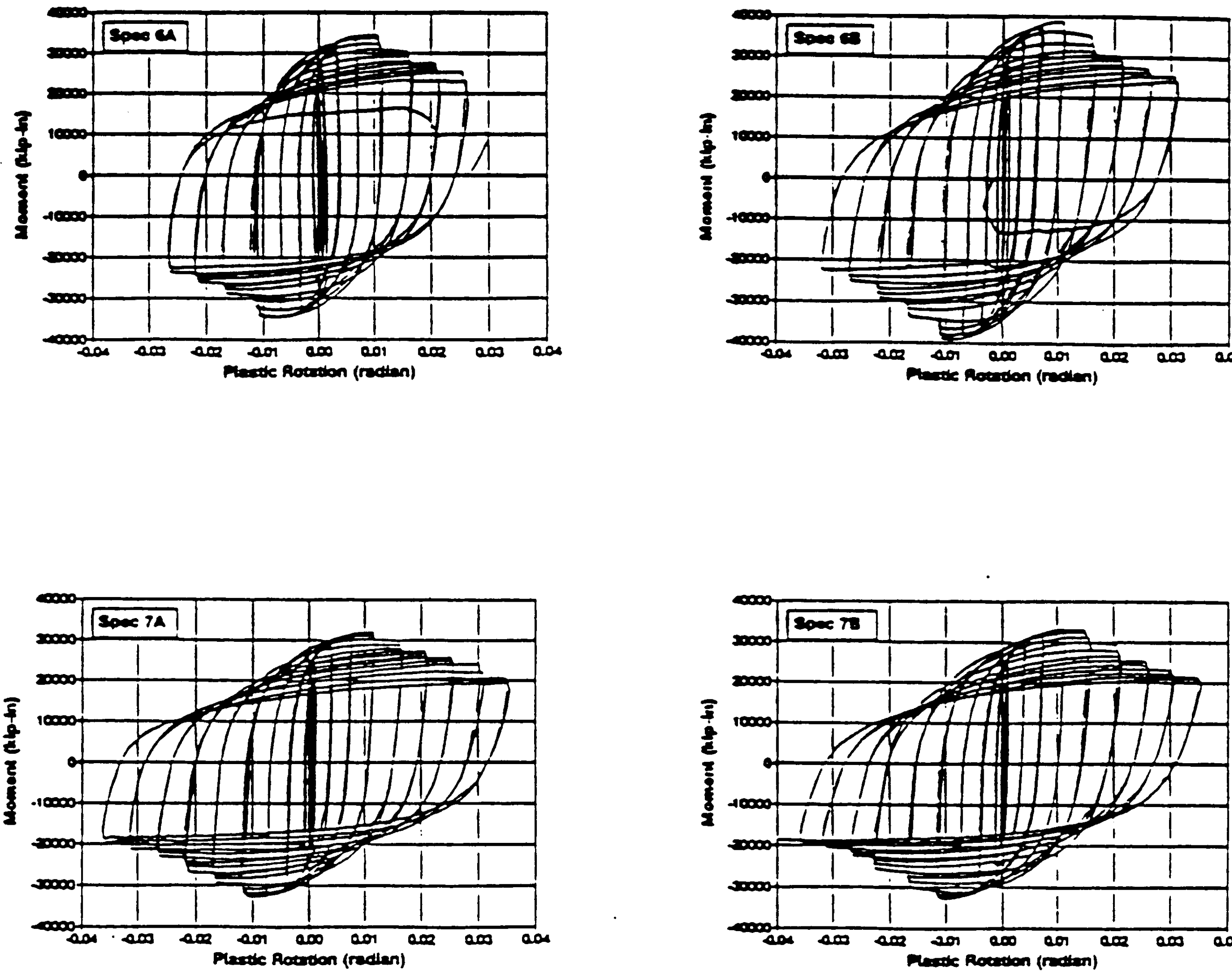
Specimen 3B, shown in Figure 12, was the first test judged to be successful. Significant inelastic deformation was developed by the plastic hinge accompanied by significant flange buckling. Ultimate failure of the section was a gradual tearing of the bottom flange at the end of the bottom flange coverplate. The relatively high level of plastic rotation, 0.025 radians and the ductile failure resulted in an overall performance assessment of "very good." Approximately two-thirds of the maximum flexural strength was maintained throughout the loading history. The gradual loss of flexural strength at increasing levels of plastic deformation is believed to have resulted from the buckling of the beam flanges. This buckling acted as a limit on the strength of the connection, and, hence, limited the stress on the CJP weld by concentrating the plastic hinge at the end of the coverplate.

It is interesting to note that the welder for Specimen 3B was forced to use settings on his welding machine, based on the electrode manufacturer's recommendations, that resulted in welds that he believed were going down "too cold." In spite of this, the performance of this specimens, and, in fact, all of the "B" specimens in Test Series I (Specimens 1B, 2B, and 3B), was significantly better than the corresponding "A" specimen where the welder was permitted to set the welding machine at settings he believed would result in the "best" weld.

Specimen 5B, shown in Figure 12, was tested next and failed when a divot of steel abruptly was pulled from the column flange at the top flange of the beam. It appears that the fracture occurred completely within the parent metal of the column at a very low level of plastic rotation. This sudden, brittle failure resulted in the overall performance assessment of "unsatisfactory." Specimen 5A achieved a high level of plastic rotation ; the test was terminated because of a gradual tearing of the bottom flange at the end of the coverplate. The performance of this specimen was judged to be "very good."

Both replicates of Specimen 6, shown in Figure 13, utilizing upstanding ribs, ultimately failed due to a gradual tearing of the bottom flange at the end of the coverplate at a high level of plastic rotation. Overall performance was assessed as "very good."

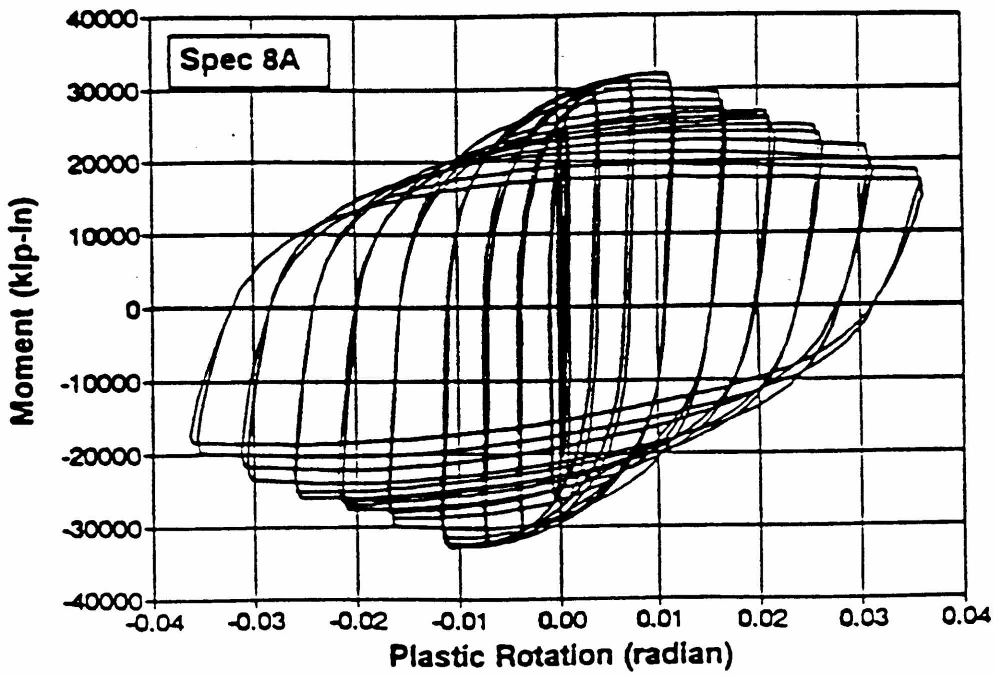
Specimen 7B, shown in Figure 13, achieved the highest level of plastic rotation of all of the specimens tested to date. The test was terminated when torsion of the beam threatened to damage the loading ram. The beam flanges were severely buckled but the connection maintained at least two-thirds of its maximum strength. Its performance was judged to have been excellent. Specimens 7A, shown in Figure 13, 8A, and 8B, shown in Figure 14, reported similarly excellent results.

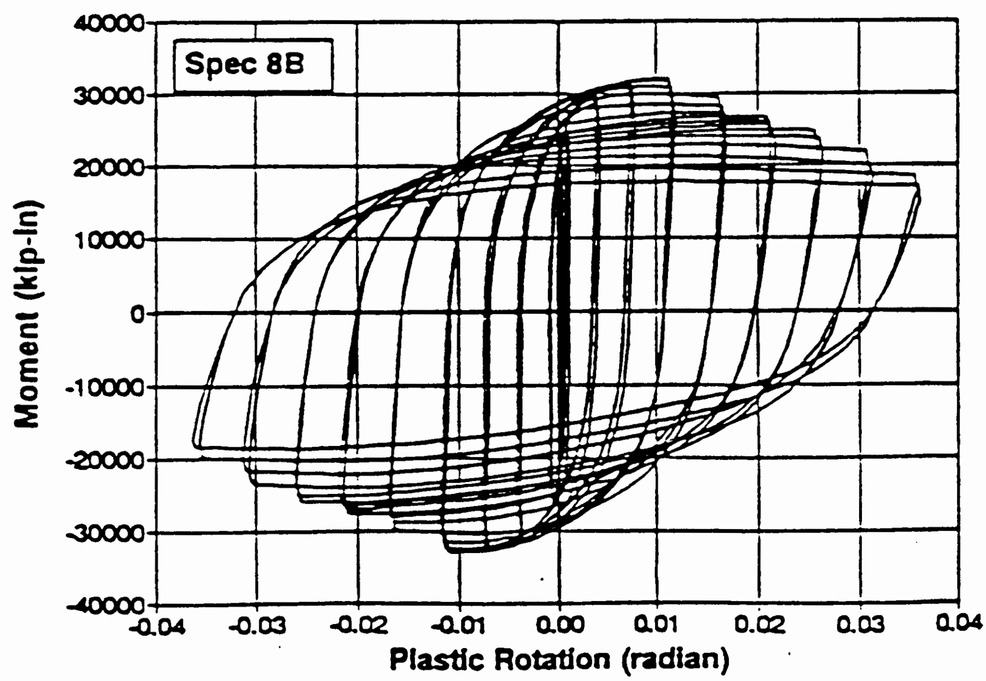


MOMENT ROTATION CURVES.
SPECIMENS 6 AND 7

FIGURE

13





MOMENT ROTATION CURVES - SPECIMEN 8

FIGURE 14

a. Crack type classification/description
b. AWS Discontinuity Severity Class of crack/defect per Table 8.2 of AWS D1.1-94
c. Depths of crack/defect
d. Length of crack/defect
e. Location of crack/defect

The field inspection report form shall, as a minimum, objectively identify the following for each shear connection plate inspected:

a. Bolt damage
b. Bolt slip
c. Bolt elongation
d. Ovaling of bolt hole in shear plate or beam
e. Plate yielding/stretch marks in shear plate
f. Cracking in shear plate
g. Cracking in beam web

4. If significant damage by quantity and/or severity is found, as determined by the Structural Engineer, an additional 10% but not less than 6 of the steel frame moment connections shall be inspected. If additional significant damage is found, the remaining steel frame moment connections shall be inspected.

High Rise Buildings

1. Utilize the inspection recommendations for low-rise buildings excepts as modified below.

2. The larger of a minimum of 10% of all steel frame moment connections in the building or 12 steel frame moment connections, in proportion to the number of connections in each direction, shall be inspected both visually and by ultrasonic testing. The minimum requirement of the number of steel frame moment connections for the initial sampling may be reduced if justified by a <u>combination</u> of a more elaborate analysis and positive structural attributes (i.e., uniform building configuration, structural regularity, moment frame redundancy, and embedded column anchorage). In no case shall the reduced sampling be less than the larger of 5% or 12 of all steel frame moment connections.

3. If significant damage by quantity and/or severity is found, as determined by the Structural Engineer, an additional 10% but not less than 12 of the steel frame moment connections shall be inspected. If additional significant damage is found, the remaining steel frame moment connections shall be inspected.

CONCLUSIONS AND RECOMMENDATIONS

Careful consideration will have to be given to the extent to which joints are repaired in buildings with weld connection damage. Should one simply repair the damaged joints or should one also upgrade all of the undamaged joints? It probably depends upon the status of the building (e.g. if under construction, it might be reasonable to consider upgrading all of the undamaged joints), the likelihood that the frames as designed will experience seismic loads that will begin to develop plastic hinges in the frame beams (e.g. frames based on design earthquakes with very long return periods might suggest lower rates of joint retrofit), the presence of unusual configurations, and the like.

The issue of what to do with damaged joints remains an open question. The AISC testing program as well as the poor performance of the typical beam/column frame joint provide convincing evidence that this joint is not capable of producing acceptable behavior. There is, therefore, reason to question the efficacy of only repairing the joint by rewelding and/or only removing the backup bars. In the short-term, it is not probable that the damaged buildings will be subjected to another earthquake likely put the building in further significant danger of collapse. Nevertheless, there is a risk in delaying repair work until more comprehensive testing is completed and repair design recommendations are developed. The structural engineer will have to carefully consider the following:

1. The extent and severity of damage to the building;
2. The extent to which the existing lateral force resisting system has been compromised;
3. The risk posed by waiting to repair or strengthen the damaged connections until a consensus is reached regarding effective repair methods.
4. The practical repair alternatives available;
5. The benefits gained by each repair alternative, both in the short-term and long-term, relative to the costs and disruption associated with the repair work.

In summary, the following recommendations are offered to assist structural engineers while more comprehensive testing is undertaken.

Evaluation of Existing Joints: The structural engineer should undertake a rational program to inspect the joints in steel frame buildings suspected of sustaining damage to ductile frame beam/column joints. An inspection program similar to that outlined in the "Inspection" section of this report may be used as a starting point .

Repairs of Damaged Joints: The structural engineer should establish the expected demand on the joints in a ductile frame to identify critical joints in the seismic system. Based on the results of this analysis and the engineer's professional judgement, a repair program should be developed that provides for reliable performance of the lateral force resisting system. That repair program may consist of a combination of damaged and undamaged joint repair and reinforcing.

Delay in Repairs or New Steel Frame Construction: If possible in the opinion of the structural engineer, with due consideration of potential immediate life-safety risks, repair of damaged connections should be delayed until a consensus is reached regarding effective repair methods. Construction of steel buildings utilizing ductile steel frames should be postponed until a consensus is reached regarding effective methods of producing an acceptable joint. If construction must continue, it is recommended that strengthened joints be utilized.

Use of Welded Flange/Bolted Web Connections: Use of details similar to those described in *1994 UBC* Section 2711.7.1.2, even those relying on complete penetration web welds in lieu of bolted shear connections, should be discontinued immediately. The International Conference of Building Officials has passed an emergency code change deleting the prescriptive joint design requirements from the *1994 UBC.*

Table 4
Test Results

Specimen Number	Failure Type and Location	Maximum Energy Dissipated	Plastic Rotation (radians)	M_{max}/ZF_y	Nominal CJP Weld Stress (ksi)	Overall Performance Assessment
1A	Brittle fracture of top CJP weld	680 in-kips	0.005	1.20	63.6	Very Poor
1B	Brittle fracture of bottom CJP weld	1,600 in-kips	0.005	1.29	68.7	Poor
2A	Brittle fracture of top CJP weld	290 in-kips	0.0025	1.24	66.2	Very Poor
2B	Brittle fracture of bottom CJP weld	2,620 in-kips	0.009	1.48	78.9	Poor
3A	Brittle fracture of top CJP weld	3,740 in-kips	0.015	1.48	43.3	Poor
3B	Ductile tearing of bottom flange	16,800 in-kips	0.025	1.67	48.9	Very good
5A	Ductile tearing of bottom flange	19,700 in-kips	0.025	1.77	43.3	Very good
5B	Brittle failure of at top flange (divot removed from col.)	1,600 in-kips	0.005	1.82	44.4	Very Poor
6A	Ductile tearing of bottom flange	19,250 in-kips	0.025	1.63	52.4	Very good
6B	Ductile tearing of bottom flange	22,250 in-kips	0.03	1.86	60.2	Very good
7A	Ductile tearing of bottom flange	25,500 in-kips	0.035	1.53	44.7	Excellent
7B	Test terminated to protect loading frame	25,500 in-kips	0.05	1.63	47.5	Excellent
8A	Ductile tearing of bottom flange	26,300 in-kips	0.035	1.53	44.7	Excellent
8B	Portion of bottom flange fractured	25,800 in-kips	0.035	1.58	46.1	Excellent

Table 4 summarizes the results of the tests. "Maximum Energy Dissipated" recorded the area inscribed by the load-deflection curve and corresponds to the graphical representation shown in Figure 10. The column labeled "Plastic Rotation" recorded the highest level of plastic rotation achieved, irrespective of the number of cycles at that maximum level. "M_{max}/ZF_y" gives some indication of the maximum moment achieved by the tested section compared to the nominal plastic moment, calculated using the plastic section modulus, Z, and the specified yield strength, F_y. The column labeled "Nominal CJP Weld Stress" represents the approximate stress at the weld at Mmax, calculated using the tension-compression couple of the unplated or plated section, as appropriate, and assuming that the web did not resist any of the moment. Ignoring the web contribution will tend to overstate the stress in the weld, but the amount of web participation is not known with certainty.

In addition to the use of flat coverplates or upstanding ribs, other alternatives have been suggested. These include concepts such as reducing the plastic section modulus of the girder by smoothly notching the beam flange or drilling a pattern of holes in the beam flange and adding a haunch from a split "tee" section to the bottom flange. These approach may also have some appeal for repairs where disruption of the existing joint is reduced. However, these alternatives were not tested in the current research program because this work was directed toward developing design concepts for buildings currently under design or under construction. Other approaches such as the use of semi-rigid connections, either intentionally designed into the structure or as the result of damage during an earthquake, may also prove to be of value. While the ultimate decision as to which path to take in this matter will have to be based on engineering judgement, research and economic evaluation, there is currently no physical research to document the effectiveness of these alternative approaches. It is expected that this situation will change in the near future as other research programs get underway.

Preliminary Test Conclusions. The preliminary conclusions of the AISC testing program, coupled with observations made of the large inventory of damaged connections suggests the following:

1. Beam/column connections with configurations corresponding to the basic requirements of UBC Section 2711.7.1.2 (bolted web and welded flanges) did not develop adequate amounts of plastic rotation to insure acceptable performance in a large earthquake. This occurred despite efforts to insure a high level of workmanship and better than normal welding techniques, removal of back-up bars, additional reinforcing fillets, fully welded web connections, and the like were used.

2. Reinforced connections that shifted the plastic hinge away from the face of the column and the CJP weld performed better than connections that did not shift the plastic hinge. Specimens 3, 5, 6, and 7 developed significantly more plastic rotation than did Specimens 1 and 2.

3. Strict adherence to an approved Welding Procedure Specification (WPS) and proper welding technique appears to offer the best chance of reliable performance. The AISC testing program suggests that it is difficult for even an experienced welder to produce reliable welds when they deviate from an approved WPS. Experienced inspectors appear to be unable to detect defects that ultimately compromise the performance of welds that look acceptable but that were not executed in conformance with an approved WPS.

4. It is difficult to detect defects in bottom flange welds that are ultrasonically examined from the top side of the bottom flange because of the interference of the web.

5. The AISC testing program did not reveal the effect of column stiffeners or lack thereof in these connections. This may be because several parameters were changed at the same time the stiffeners were added and because the column flanges were very stiff and did not require stiffeners based on the requirements of the *UBC*. While some argue that the stiffener limits the desirable deformation of the column, it is likely that column stiffeners would improve the performance of joints constructed using smaller columns.

6. The benefits of special welding techniques such as the use of smaller diameter electrodes and electrodes with impact toughness requirements, and removal of the backup bar in reinforced connections were not demonstrated by the current testing program. The difficulty in drawing conclusions may have occurred because the testing budget prevented changing only one parameter at a time. Nevertheless, it is argued that providing a minimum level of toughness and removal of potential stress risers in the joint will be shown to be beneficial.

INSPECTION OF BUILDINGS

The number of buildings discovered with weld connection failures continues to increase as more building owners and engineers become aware of this problem. Undoubtedly, there are other buildings that have yet to be discovered. The issue of how to inspect a building with potential frame connection failures is difficult is prescribe because in many cases there is little or no nonstructural damage accompanying the weld failure that would suggest significant structural distress. Many of the failures were found through careful inspection of the fire proofing for tell-tale hairline cracks, although there have been a number of instances where failures did not cause the fire proofing to crack. There does not appear to be a pattern to the joint damage that corresponds to demands calculated using available analytical techniques. Therefore, it is difficult to use analysis or engineering intuition to accurately predict where damaged joints will be found. However, buildings that have been shown to exceed the maximum plumbness tolerance permitted by *AISC Code of Standard Practice*, Section 7.1, have been found to contain damaged joints. It should be noted that the absence of out-of-plumbness is *not* necessarily an indication that significant damage to welds and/or parent material has not occurred.

An arbitrary percentage of visual and ultrasonic joint inspections, supplemented by rational engineering judgement, is one approach that has been suggested to identify buildings with damaged joints. The following inspection criteria are based on work by the City of Los Angeles/Structural Engineers Association of Southern California Steel Frame Committee for inspection of low rise and high rise structures.

Recommended Inspection Criteria
The intent of the inspection is to find and identify significant damage to connections due to earthquake. A small number of root defects/cracks are not considered to be significant earthquake damage. The buildings to be investigated shall be classified Low Rise (6 stories or less) and High Rise (7 stories or more).

Low-Rise Buildings
The criteria to be used in determining the number of moment connections to be investigated is as follows:

1. Determine if post-earthquake out-of-plumbness exceeds AISC Frame Tolerances as defined in the AISC Code of Standard Practice, Section 7.11. As of this date, records on buildings that have been found to exceed AISC plumbness tolerances indicate damage to moment connections. Experience to date also has shown that the absence of out-of-plumbness is *not* necessarily an indication that significant damage to welds and/or parent material has not occurred.

2. The larger of a minimum of 15% of all steel frame moment connections in the building or 8 steel frame moment connections, in proportion to the number of connections in each direction, shall be inspected both visually and by ultrasonic testing. Visual inspection shall include all welds (including welds to shear tab, doubler plate and continuity plate) and beam web bolts. A steel frame moment connection is defined as a beam-to-column connection, consisting of top and bottom flange complete joint penetration welds and a shear tab connection, and pertaining to either a major or minor column axis orientation. Only flange welds require ultrasonic testing. This initial sampling shall be specified by the Structural Engineer using *both* of the following bases:

    a. Rational selection, i.e., shown by analysis and/or deemed by engineering judgment to be more heavily stressed during the Northridge earthquake because of location and/or building configuration.
    b. Random selection.: experience to date has shown that discovered damage in many instances appears to be random and does *not* necessarily correlate with moment connections shown by analysis and/or deemed by engineering judgment to be more heavily stressed.

    The inspection sample shall include moment connections on multiple levels of the building.

3. Inspection of the moment connection shall require a visual inspection and ultrasonic testing by a level II ultrasonic technician certified in accordance with AWSD1.1 and SNT-TC-1A. All Level II certifications shall be current within the past 3 years. Certification shall be obtained from a Level III technician with current certification by examination.

    A field inspection report shall be completed for each penetration weld tested, (i.e., there are two penetration welds per steel frame moment connection to be reported on) regardless of whether cracks or defects are detected. This field inspection report form shall, as a minimum, objectively identify for each CJP weld tested the following:

Reinforced Joint Detail for Improved Performance: Until additional research is available, beam/column joints in ductile steel frames should utilize reinforced connections capable of developing 1.5 times the specified strength of the frame girder. Generally speaking, this appears to be consistent with providing plates with approximately 70 to 100 percent of the girder flange area. At this time, reinforcement schemes using either flat coverplates or upstanding ribs, similar to the geometries found in Figures 8 and 9, appear to be effective. No relative benefit was seen in using the tapered coverplates on the top and bottom flanges, as shown in Figure 7.

Rational principles should be used to justify demand on the components in the joint. Should it be concluded that a reinforced joint is needed, it should be detailed to shift the plastic hinge approximately $d/2$ away from the face of the column, where d is the depth of the frame beam. Demand at the face of column should reflect the location of the plastic hinge. It appears that demand on the CJP weld should be reduced to approximately 45 ksi based on the expected maximum moment demand (not specified plastic moment) and the strengthened section. Demand on the column panel zone should consider the location of the plastic hinge and the increased shear associated with the shifted hinge. If sufficient girder strengthening is provided to effectively shift the plastic hinge, it appears reasonable to calculate demand on the panel zone using M_p of the unstrengthened section, transferred to the face of the column, not the M_p of the strengthened section.

Welding Procedure Specification: The engineer of record should require that a WPS be developed for each weld by the Contractor. The WPS should be submitted for review by the engineer accompanied by the welding electrode manufacturer's technical literature for the filler metal proposed for the WPS. It is also essential that the welding inspector verify that all parameters of the WPS are adhered to and will require the inspectors be properly equipped to monitor same.

Projects using welded beam-column joints in ductile steel frames should develop and strictly enforce welding procedures specifications (WPS) for the project welding. The WPS should follow the requirements of AWS D1.1 and specify, at a minimum, the following:

a. Procedure Identification
b. Base Metal Identification
c. Welding Process
d. Type of Welding
e. Position of Welding
f. Filler Metal Specification
g. Filler Metal Classification
h. Number of passes (single or multiple)
i. Welding Current
j. Welding Polarity
k. Pre-Heat and Interpass Temperatures
l. Controlled Cooling Requirements
m. Welding Parameters (electrode diameter, amperage range, voltage range, travel speed range, wire feed speed range, and electrical stickout)

The welding parameters are a function of each electrode. These parameters should be developed by the fabricator in consultation with the electrode manufacturer. It is recommended that the fabricator submit with the WPS a copy of the electrode manufacturer's technical information to confirm the parameters listed in the WPS.

Steel Overstrength: Designers should consider the impact of overstrength in the design of beam to column joints in ductile steel frames.

Welding Inspection: Welds should be ultrasonically inspected from both sides of the bottom flange to insure that the entire body of the weld is examined. Ultrasonic welding inspection should be performed by a certified welding inspector per AWS QC1 with at least a UT Level II certification per the requirements of the recommended practice of ASNT SNT-TC-1A.

Ultrasonic inspections of the CJP welds should be conducted from both the top and bottom sides of the flange, and from the back side of the column flange, as necessary, to obtain as complete an understanding as possible of potential rejectable welding defects.

Removal of Run-off Tabs: Run-off tabs should be formed in accordance with AWS requirements; weld "dams" at the beam flange are not permitted. Run-off tabs should be removed after welding and the flange edge should be ground smooth to remove notches, nicks, gouges, etc.

Unresolved Issues: The following are unresolved issues. Among the many questions that still remain to be answered, the following issues have elicited comments from different parties since the earthquake. The recommendations that follow from these unresolved issues reflect professional judgement:

a. **Welding Electrode Diameter:** The AISC tests were unable to distinguish the effect of welding electrode diameter on joint performance. Welds using electrodes up to 0.120 in. in diameter performed well.

   Recommendation: Limit welding electrode diameter to 7/64 in.

b. **Column Stiffeners:** Although tests on smaller specimens have shown that improved joint ductility can be expected when column stiffeners are used, the AISC tests were unable to distinguish the effect of column stiffeners on full-scale joint performance. The columns tested in the AISC program had very thick flanges and are not representative of all columns.

   Recommendation: Full-height column stiffeners with a thickness of at least $t_f/2$ (of the frame girder) should be used when an analysis indicates that thinner stiffeners or no stiffeners are required.

c. **Weld Electrode Toughness Requirements:** The AISC tests were unable to distinguish the effect of weld impact toughness on joint performance. This may be a result of the slow loading rate used in the test.

Recommendations: Filler metal shall be capable of delivering a minimum of 20 ft-lbs at -20° F as measured by a standard Charpy V-notch test in accordance with the applicable AWS filler metal specification.

d. **Back Up Bar Removal:** The AISC tests were unable to distinguish the effect of back up bar removal on joint performance. Removal of the back up bars, *in the absence of joint reinforcement and proper welding procedures*, has not been shown to be sufficient to prevent premature failure of the joint. The strengthened joints were tested with the top back up bar in place and the bottom back-up bar removed.

Recommendation: At a minimum, remove the bottom flange back up bar and add a reinforcing fillet weld.

It is clear that a *comprehensive* test program of large-member bolted web-welded flange frame joints is required. In reviewing the available test data base, it is apparent that levels of anticipated performance were not achieved in a majority of the pre-Northridge Earthquake test programs, although concerns related to these poor performances were not widely voiced. To the economic detriment of the steel industry, an underfunded, incomplete testing program will not prevent the use of what may well turn out to be unneeded connection enhancements . Further, structural engineers may well express a healthy, understandable level of skepticism of simple fixes unless the promised performance is substantiated with comprehensive testing.

The Committee supports the immediate initiation of a comprehensive testing program so that repair options can be assessed and new building frame designs can be executed with confidence. The testing program should attempt to identify the causes of the joint failures and the relative contributions of the various methods of improving the joint performance. Knowing the causes will allow for meaningful assessments of existing buildings which did not suffer joint failures in the Northridge Earthquake but are vulnerable to other, significant seismic events. In a similar way, knowing the relative contribution of each repair option or performance enhancement would permit engineers, fabricators, and building owners to assess the economic benefit associated with each level of improved performance.

Unlike many concrete and precast buildings which suffered failures leading to collapse, none of the steel buildings in which the joint failures occurred has collapsed. While the lack of catastrophic failures should not be taken as a sign that the performance of the welded frame joint was acceptable, the Northridge Earthquake has given the building industry the opportunity to thoughtfully assess the causes of unacceptable performance and suggest effective solutions to improving joint reliability. It is essential, however, that the current concern of the engineering profession and the steel and welding industries be maintained and translated into effective action until acceptable solutions are developed.

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APPENDIX C

COMMENTARY: ENGINEERING IMPLICATIONS OF THE NORTHRIDGE EARTHQUAKE

ENGINEERING IMPLICATIONS OF THE NORTHRIDGE EARTHQUAKE

1. INTRODUCTION

The magnitude 6.7 M_w Northridge earthquake that occurred on January 17, 1994 provided a significant test for many buildings designed in accordance with these recommendations. Although of relatively short duration, spectral values in broad period ranges at many sites exceeded the design Bluebook design spectrum. The performance of these tested buildings, both good and poor, must be carefully studied to determine engineering implications and to formulate any revisions to these recommendations that may be appropriate.

Overall impressions of damage patterns or the performance of a single building may not be sufficient to justify changes in recommendations that will affect the design of all future buildings. The criteria used to determine the appropriateness of changes to seismic design recommendations should include:

1. The characteristic of the ground motion. The exact motion at any one site is not known unless the site or building is instrumented. The possibility that extraordinary damage was caused by unusual ground motion or motion particularly tuned to a structure must be considered. The effects of duration are poorly quantified, but the acceptability of performance for any ground motion intensity should consider this factor.

2. Couple the ground motion intensity with expected or acceptable performance. Given that damage is expected for moderate and major levels of ground motion (Ref. Bluebook page 1-C), the extent of actual damage should be measured against expectations before considering changes to design recommendations.

3. Damage to older buildings, designed to earlier versions of these recommendations should be carefully screened and considered separately. This performance may include effects of multiple deficiencies and therefore not be applicable as reinforcement for specific changes to current recommendations.

4. Consider how well the current recommendations were implemented and whether improvements can be most efficiently be made by more stringent design recommendations or better implementation of the current ones.

Review and analysis of the results of any damaging earthquake is educational. Damage or damage patterns which, for one reason or the other, do not warrant code changes, should be flagged so designers can consider individually whether change their design practice with respect to structural systems, detailing, or interaction with the construction process. This section has been added to capture concerns which may have not yet resulted in changes in formal design recommendations but that should be considered in designs immediately.

The primary design issues resulting from the Northridge earthquake include: 1) concern regarding whether the damage and resulting economic losses to be expected using current Bluebook performance goals is generally acceptable, 2) the intensity and characteristics of the ground motion particularly comparison with standard design spectra, possible near field effects, and possible effects of vertical accelerations, 2) the performance of wood framed buildings, particularly gyp board and stucco shear walls, and the performance of certain commonly used hardware, 4) the collapse of several concrete parking garages, and the potential lack of deformation compatibility in many other concrete buildings, 5) the failure of wall-diaphragm ties or cross building ties in stiff-walled buildings with flexible diaphragms - most notably tilt-ups, 6) the surprising failure of many beam-to-column joints in steel moment resistant frames, and the brittle fracture of certain steel element in other situations. Each of these areas of concern will be discussed separately.

2. BLUEBOOK PERFORMANCE GOALS

The well known three level performance goals found at the beginning of the Commentary has served SEAOC well for nearly 40 years, particularly for the continuing development of code provisions to prevent collapse and protect life safety:

> Structures designed in conformance with these Recommendations should, in general, be able to:
>
> 1. Resist a minor level of earthquake ground motion without damage;
>
> 2. Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some nonstructural damage;
>
> 3. Resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as nonstructural damage.
>
> It is expected that structural damage, even in a major earthquake, will be limited to a repairable level . . . .

> . . . In order to fulfill the life safety objective of these Recommendations, there are requirements that provide for structural stability in the event of extreme structural deformations . . . . While damage to the primary structural system may be either negligible or significant, repairable or virtually irreparable, it is reasonable to expect that a well-planned and constructed structure will not collapse in a major earthquake.

However, there are growing concerns within SEAOC about whether the current elastic equivalent lateral force seismic code format can be used to reliably predict performance over the full range of ground motions indicated, except in the simplest situations. Further, if performance at several levels of ground motions becomes more an integral part of the code, these goals probably need complete redefinition.

Perhaps more importantly, damage in the Loma Prieta and Northridge earthquakes has caused the public the question whether these traditional code performance goals are acceptable. The public is more aware of the costs of repair of damage and from lost use of buildings and concern may be shifting beyond life safety to economic considerations. From a public policy standpoint, there is also interest in minimizing the demand for emergency shelter after earthquakes by attempting to provide performance which will yield less (or no) inhabitable housing units.

One of the most difficult aspects of the multi-level description is the characterization of ground motion. Was the ground motion level at a given site *minor, moderate, or major*? The ground motions at some sites during the Northridge earthquake had spectral values among the largest ever recorded, but the duration of strong shaking was one third to one half of what is projected in other circumstances in Zone 4. Should damage be expected in accordance with performance level 2. (moderate) or 3. (major)? Prior to the current interest and open discussion about acceptable performance, the Bluebook performance goals have been used as a two-edge sword. On the one hand, any structure that did not collapse was considered to have met the goal (level 3.), regardless of the level of damage or severity of ground motion. On the other hand, patterns of damage were almost always countered with preventive code changes (apparently using level 2.), regardless of the level of damage or severity of ground motion.

Considering damage levels, it is becoming well recognized that, because of expected nonlinear structural behavior, deformations should replace forces as the key parameter. In order to improve damage predictability, target damage limitations must be linked to deformations and actual deformations must be better estimated.

In the future, as our ability improves to both define probable future ground motions and predict damage, more meaningful damage levels must be identified and coupled with more definitive ground motions so that the success or failure of code provisions can be more accurately gauged.

In current designs, if project performance objectives are set more definitively than the Bluebook's, the nonlinear behavior of the structure and the deformation demand of the expected or design ground motion should at least be considered.

3. PERFORMANCE OF WOOD FRAME BUILDINGS

The vast majority of structures subjected to the earthquake were of wood frame construction. With the exception of a few notable cases, the structures generally withstood the earthquake without collapse and for the most part without extensive structural damage. A study performed for the Department of Housing and Urban Development analyzed a random sample of several hundred wood framed structures within a ten mile radius of the epicenter (NAHB, 1994). Two groups of structures were analyzed: single family detached properties (SFD), and single family attached and multifamily low-rise properties (SFA/MFLR). The SFD results indicated remarkably low levels of damage with 90 percent of the structures receiving no foundation damage, 98 percent receiving no wall framing damage, and 99 percent receiving no roof framing damage. The SFA/MFLR data was found to be inconclusive because of sampling problems. However the general trend of low damage occurrence in the data is in general agreement with the Los Angeles Department of Building and Safety's inspection database. It was noted that much of the MFLR damage was due to a soft or weak story.

Multi-story structures with soft or weak first stories (generally resulting from wood framed parking provisions) performed poorly, with several experiencing either a collapse of the first story or a gross residual horizontal deflection which has resulted in a questionable ability to economically repair the building. Hamburger (1994) has estimated that approximately 200 structures either had collapse or gross residual soft story deflection. The collapse of the three-story Northridge Meadows apartment building resulted in the loss of 16 lives. Extensive cosmetic damage occurred throughout the area to wood frame structures in the form of cracking of interior wall finishes, and exterior plaster (stucco) cracking. In many buildings, these finish materials were also being utilized as structural elements for the resistance of lateral forces. The cracking and damage experienced thus has not only resulted in cosmetic distress but has compromised the lateral load-resisting system of the structure. When the stucco wall is erected, it is backed with a wire-mesh/building paper attached to the stud framing with power-driven fasteners. In many cases, the wire mesh was installed flush with the paper, providing little bond with the stucco. Two types of failures are noted by Hamburger (1994): staples used to connect the stucco wall pulled out of the framing, and the stucco itself pulled away from the mesh.

The extensive cracking of the finish materials of residential buildings has resulted in the need for costly repairs to many buildings not considered to be significantly damaged structurally. The extent of "non-structural" damage and associated cost may well be much more extensive than the general public had anticipated.

Narrow plywood walls, primarily in the first story of multi-story buildings, experienced significant lateral displacement and incipient structural failures were observed (Andreason and Rose, 1994). Hamburger (1994) reports on splitting of end studs at holddown bolt locations, in part because of inadequate stud size, and mislocation of the holddown. Many problems with sill bolting were observed due to poor construction practices.. Anchor bolts were not properly located and were bent to fit into holdowns. This led to damage when they straightened in tension. Many bolts were placed too close to the edge of the concrete and failed due to inadequate edge distances. Cripple wall failures were limited, primarily because most homes in the area are built with slab-on-grade construction and thus there is no crawl space.

A design issue which has recently been brought to light is the common assumption for light frame construction that the load carried by the various shear walls is directly proportional to their length. Narrow shear walls, those with height-to-width ratios greater than two, have been found to be much more flexible than assumed even when properly constructed due to the bending or overturning effects. Thus, these walls are not as effective in resisting earthquake loads as previously assumed. The longer walls, where present, will take a much higher portion of the load, and the horizontal distortion will be much larger than anticipated. These larger-than-anticipated deflections or drifts can result in more damage to the non-structural elements in the structure.

Recent testing of narrow shear walls (Allred, 1994) has also indicated some concern with holddown connections at the ends of the walls. Observed failures included splitting of the wood post at the bolt holes, and substantial rotation and elongation of the holddowns. Sliding of the sill plate after the anchor bolts split the wood was also observed. The American Plywood Association is also studying this issue. Their final report on testing was due in 1994 too late for this document. Early results were reported by Commins and Gregg (1994), indicating that the National Design Specification (NDS) bolt spacing values may be inadequate, plywood nailing and holddowns distorted significantly, and code deflections were exceeded when code forces were applied. Unfortunately, while the testing was cyclic, displacements were not taken to the levels that might be experienced during an actual earthquake, but rather to levels near the much lower values that might be used in code design.

As a result of a study of the performance of wood framed buildings by a task force of the City of Los Angeles and the Structural Engineers Association of Southern California, a series of recommendations for adoption by the City as it relates to wood frame buildings has been made. Some of the major items in the report and the Emergency Enforcement Measures that the L. A. City Building Department has adopted are as follows:

1. Reduction of the allowable shear value for stucco and drywall to approximately half of that previously allowed by the code. (Note this is an additional reduction from that in the 1988 Code.) Restrict the use of staples to attaching wire lath in stucco and restrict the height-to-length ratio for walls used for lateral resistance to no more than 1:1. In addition, these walls would be restricted to one-story buildings or the upper stories of multi-level buildings.

2. For plywood shear walls, reduce the code values by approximately 25 percent and restrict the height-to-length ratio to 2:1 in lieu of the previous 3.5:1 ratio. For heavily loaded walls, the width of members at the boundary and plywood joints would have to be at least 3x members.

3. Require that the size of holes for bolts at the hold-down anchors be verified to be no more than 1/16 inch oversize and that the hold-down bolts be tightened properly.

4. Require that column deflection be limited to 0.005 H (where H is the height of the column) and that design of cantilever columns be based on a K = 2.l buckling factor.

5. Rotation for the distribution of lateral forces will not be allowed.

6. The lateral force-resisting systems shall be clearly shown on the plans and the elevations and detail references of all shear walls be clearly shown on the plans.

4. PERFORMANCE OF CONCRETE BUILDINGS

Pre-1973 nonductile reinforced concrete frame buildings did not perform well in the Northridge earthquake, as was only to be expected. The 7-story Barrington building provided one of the worst examples of such nonductile building performance. The Kaiser Permanente office building at Grenada Hills suffered collapse of the second story throughout the length of the building and collapse of the end bays throughout the full height. The joints had completely failed because of grossly inadequate confinement reinforcement. The structural system for the Bullock's department store at the Northridge Fashion Center, which collapsed, consisted of lightweight concrete waffle slabs supported by circular columns with a few poorly constructed discontinuous and ineffective shear walls.

Structures with shearwalls as a primary lateral load resisting system performed well with respect to life safety and prevention of collapse. However, some shearwalls showed sliding along horizontal construction joints, diagonal flexure-shear cracking and significant damage to coupling beams and short piers between openings.

Post-1973 reinforced concrete buildings generally performed well, with the noted exception of some parking structures.

There were many concrete parking structures close to the epicenter and away, both old and new, that came through the earthquake with very little or no damage. As a class, however, parking structures (cast-in-place as well as precast) fared considerably worse than other concrete buildings, even though most were post-1973 structures.

The following observations concerning the seismic design implications of the Northridge earthquake for concrete buildings should be noted.

Vertical acceleration - Immediately following the earthquake, some of the damage was blamed on vertical acceleration which was supposedly exceedingly high at many locations. Recorded peak horizontal accelerations were plotted by Moehle (EERC, 1994) against the peak vertical accelerations recorded at 38 stations. At a few locations vertical accelerations were nearly equal to the horizontal accelerations, and at one station the peak vertical acceleration exceeded the peak horizontal acceleration. However, in general, peak vertical accelerations were about two-thirds the peak horizontal accelerations, as would normally be expected. Priestley (UCSD, 1994) concluded that vertical accelerations played no significant role in the damage sustained by seven highway bridges which failed in the Northridge earthquake. However, as the absolute value of vertical accelerations increase, its potential for significant interaction with both coincident lateral motion and gravity load increases, particularly in stuctures that may have amplifying vertical response.

Bluebook Section 1E10 requires explicit consideration of vertical acceleration in the design of horizontal cantilever components as well as horizontal prestressed components. Whether explicit consideration of vertical acceleration needs to be expanded beyond that remains unclear at this time.

Structural Systems - The Bluebook formally recognize only four different structural systems for reinforced concrete buildings: the moment-resisting frame system, the bearing wall system, the dual (shearwall-frame interactive) system and the building frame system. All other structural systems for reinforced concrete buildings fall under the category of undefined systems. In recent years, there has been a proliferation of reinforced concrete structures, both parking garages and commercial buildings, that combine ductile and non-ductile frames in the same building. Typically, two ductile frames on two parallel faces of a building are relied upon for entire lateral resistance in the direction parallel to those faces. All other frames spanning in the same direction are designed for gravity only. Are these building to be designed as moment-resisting frame systems or as undefined systems? The conservative answer is probably the latter. In view of the popularity of this system, guidance needs to be developed as to its design.

The Bluebook specifically requires that any structural element that is designated to be not part of the lateral load resisting system of a building in a moderate or a high seismic zone be designed to be able to sustain factored gravity loads imposed on it under a lateral displacement equal to ($3R_w/8$) times the lateral displacement of the lateral load resisting system elastically computed under the code-prescribed seismic forces. This deformation compatibility requirement is crucial to the safety of the building frame system where lateral loads are resisted by shearwalls, and frames carry gravity loads only. It is even more crucial to the safety of buildings that combine a small number of ductile lateral load-resisting frames with a significant number of non-ductile frames designed for gravity only. Unfortunately the deformation compatibility check is seldom properly carried out by design engineers, and even more seldom enforced by local jurisdictions.

The U.B.C. further requires that a column that is not part of the lateral force resisting system in Seismic Zone 3 or 4 be detailed like a column that is part of the resisting system in Seismic Zone 2 (Sec. 1921.7.2.2 of U.B.C.-94). In the absence of proper deformation compatibility checks, this required level of detailing is not sufficient, and needs to be made more stringent. Proposals in that regard are under consideration by ACI 318 Subcommittee H (Seismic Detailing) as well as by ICBO for possible inclusion in the 1996 Supplement to the U.B.C.

In the absence of proper deformation compatibility checks, a requirement needs to be considered that would restrict the maximum number of non-ductile elements or subsystems that can be combined with a given number of ductile elements or systems along a particular direction of a building.

Section 1909.3.4.2 of U.B.C.-94 appears to allow the use of overstrength, non-ductile columns even as part of the lateral force resisting system. This section is confusing, may lead to unsafe designs, and has been proposed for deletion by ACI 318 as well as ICBO.

<u>Precast, Prestressed Concrete</u> - It is felt that specific code requirements are needed for precast, prestressed concrete structures in high seismic zones. Such specific provisions will replace the current vague requirement that precast, prestressed concrete structures be equivalent to monolithic concrete structures in terms of strength and toughness. The interpretation, implementation and enforcement of the

current requirement has been nonuniform for obvious reasons. The Building Seismic Safety Council has just approved of a set of specific provisions for precast, prestressed concrete structures in regions of high seismicity, which will appear in the 1994 edition of the NEHRP Recommended Provisions (BSSC, 1994).

Continuous Load Path - Bluebook Section 1H2e contains "Ties and continuity" requirements. Section 1H2f contains "Collector elements" requirements which read: "Collector elements shall be provided which are capable of transferring the seismic forces originating in other portions of the building to the element providing the resistance to those forces. Section 1H2h contains requirements for "Anchorage of concrete or masonry walls." In spite of all of this, shearwalls in more than one precast parking structure that suffered partial collapse were observed to be relatively intact. The most informed speculation has been that the collector elements contained insufficient reinforcement which yielded early on, rendering the elements unable to transmit the lateral forces to the shearwalls. There may have been other modes of diaphragm failure. This is a breakdown of the continuous load path and needs to be investigated.

Cast-in-place Topping Acting as Diaphragm - An area of weakness in modern precast parking structures is the flexibility of the thin cast-in-place topping slab that forms the horizontal floor and roof diaphragms. The U.B.C. currently requires these topping slabs to be designed to act as diaphragms independently of the precast elements. Composite design of the topping plus the precast elements to act as diaphragms may very well be preferable.

Damage Control - The Northridge earthquake has once again pointed out the desirability of including meaningful damage control criteria in seismic provisions. Whether the current limitation on interstory drift computed elastically under code-prescribed seismic forces provides such damage control is highly questionable. This is particularly so because the stiffness assumptions under which the drift computations are to be made are not specified. The code-prescribed design seismic forces are dependent on the structural period which in turn also depends on stiffness assumptions. However, these stiffness assumptions may not be explicit if the approximate formulas of the code are used for period computations or if the code limitation is invoked, which restricts the design force level based on rationally computed period to be no less than 80 percent of the design force level based on the approximate period of the code.

Proper consideration of the above will hopefully facilitate the successful design of reinforced concrete buildings in regions of high seismicity.

5. PERFORMANCE OF TILT-UPS

The Northridge event was the first since the 1971 San Fernando Earthquake to subject a large number of tiltup and masonry buildings to ground motions as strong as those projected for the most severe events. Within the San Fernando Valley, which encompasses the most strongly shaken areas of this earthquake there are an estimated 1,200 (Brooks, 1994) such buildings, most with panelized plywood roofs. An estimated 400 (Deppe, 1994) of these buildings within the administrative limits of the City of Los Angeles experienced severe damage including partial roof collapses and in some cases collapse of perimeter walls. Additional damage, not included within the above statistics occurred outside the Los Angeles City. Many of the damaged buildings were designed to recent editions of the *Uniform Building Code*. While no lives were lost as a result of this performance, the damage sustained by many buildings was clearly life threatening.

The primary structural failure occurred at the out-of-plane connections between the perimeter masonry or concrete walls and the panelized plywood roofs. Many failures occurred in buildings-provided with specially detailed wall ties, mandated by the *Uniform Building Code* (U.B.C.) in every edition since 1973. Nearly every conceivable type of wall tie connection experienced failure including catalog hardware supplied by timber connector manufacturers as well as custom designed and fabricated details. Wall tie failures included: shear cone pull-out of anchors within the masonry or concrete walls; splitting of timber members at bolted connections; fracture of light gauge metal hardware, particularly at locations of bends, swivel joints and bolt holes.

These buildings are classical "box" type structures with flexible diaphragms. Research on instrumented buildings following the 1984 Morgan Hill (Celebi, 1989) and 1989 Loma Prieta Earthquakes (Bouwkamp, 1991) indicates that the dynamic response of structures with predominantly solid walls is dominated by the diaphragms, with initial periods in the range of 1 to 3 seconds. Walls aligned parallel to the direction of ground motion behave in a relatively rigid manner and do not significantly amplify ground motion (recordings indicate less than 40 percent amplification of peak ground acceleration). However, the diaphragms do significantly amplify the motion transmitted by the walls. For low levels of ground motion, peak accelerations recorded at diaphragm centers are often three times those recorded at the ground. For strong ground motion, the diaphragms respond with less amplification. Diaphragm shear strength appears to be the limiting factor. Once a diaphragm yields in shear, it can not deliver any more force to the mass supported by the roof, thereby limiting the amount of acceleration which can be induced

Typical diaphragms in U.B.C. Zone 4 are designed with code level shear strengths equal to approximately 20 percent of the supported weight. Published test data on plywood diaphragms (Elliot, 1980) indicates that code level design shears have a factor of safety under monotonic loading conditions ranging from 3 to 4. Therefore, roof diaphragms without significant overstrength can deliver an effective shear equal to approximately 80 percent of the diaphragm's supported weight. Assuming diaphragm accelerations vary from approximately Z at the edge to $3Z$ at the center, the average roof acceleration is approximately $2Z$. Equating the ultimate diaphragm shear ($0.8W$) with the average roof acceleration times the effective mass ($2ZW$), it can be seen that such a roof would reach its ultimate strength when subjected to a peak ground acceleration of $0.4g$. The maximum acceleration which would ever be expected at the center of the such roof diaphragms would be three times this amount, or a maximum of $1.2g$, whether or not the actual ground motion exceeds a peak acceleration of 0.4g. Stronger diaphragms, however, could produce larger accelerations.

Given this data, ultimate demands on out-of-plane roof to wall connections can be estimated. For a one story unit strip of wall, with height h and weight w lbs./ft., subjected to acceleration "Z" at its base and "$3Z$" at its roof support point, simple calculations indicate a maximum wall anchorage, force, at the roof level of $1.167\ Z\ w\ h$. This force is directly proportional to the strength of the ground motion, represented by the coefficient Z. but is also limited, as previously described by the shear strength of the diaphragm. For roofs having a shear strength conforming to but not significantly exceeding code requirements, the maximum force which can be delivered at the top of the wall can be calculated by substituting the code value for Z in the above expression. For Zone 4, this yields a maximum wall anchorage force of $0.467\ w\ h$ pounds/foot of wall. Stronger diaphragms would be able to produce larger forces.

Based on failures in the 1971 San Fernando Earthquake, codes since the 1973 U.B.C. have required positive direction connections of walls to diaphragms, and continuous diaphragm ties to distribute anchorage loads into the diaphragms. Anchorage forces are governed by the equation:

$$F_p = Z\ I\ C_p\ W_p$$

To account for diaphragm amplification effects, anchorage forces within the middle 1/2 of the diaphragm are increased an additional 50 percent. For Zone 4, this results in anchorage design forces of $0.225\ w\ h$ in the middle half of the diaphragm and 0.15 w h elsewhere. In general, hardware used for wall anchorage incorporates a minimum factor of safety of 1.7 against failure. This results in ultimate connection strengths of $0.38\ w\ h$ and $0.26\ w\ h$, respectively in the two diaphragm regions.

It was previously shown that actual peak demands on wall anchors at the centers of roof diaphragms which meet but do not exceed code requirements is $0.467\ w\ h$. Demands on wall anchors located outside the central region can be approximated as 2/3 that at the center, or approximately $0.31\ w\ h$. This indicates that actual demands on wall anchors can exceed strengths provided under the 1994 U.B.C., by about 20 percent. For diaphragms with substantial overstrength, the excessive demands on wall anchors can be much larger. Long rectangular diaphragms commonly have substantial overstrength aligned with the long diaphragm dimension in such cases, current design practice may result in anchors which are under-designed by as much as 100 percent. Inadequate wall anchor strength would not be a significant concern if anchors exhibited ductile behavior. Unfortunately, observation of the performance of common anchorage systems in the Northridge Earthquake indicates that they do not do this.

As much as 40 percent of the total number of tiltup and masonry buildings located in that portion of the City of Los Angeles within the entire San Fernando Valley experienced severe, life threatening damage. The percentage of buildings experiencing such damage within the areas of strongest ground motion, was much higher. Analytical evaluations indicate that the 1994 U.B.C. requirements may be inadequate to protect buildings when actual ground motions approach or exceed the nominal peak accelerations represented by the zone Coefficient Z. The problem appears to be most severe for buildings located on soft soil (S3) sites or within the near-field zone of the causative fault, due to the long period nature of motion experienced on such sites.

Changes to the U.B.C. are being developed but were not enacted in the 1994 edition. Since current C_p values appear adequate for design of the wall panels themselves, if not their connections, consideration is being given to retaining current design force levels but requiring a larger factor of safety in wall anchorage design. In the interim period, it appears that the following may be appropriate for design purposes:

1. For diaphragms with actual shear strengths matching the code requirement, the C_p coefficient used or design of wall anchorage should be increased to a value of 1.5 for wall anchorage in the central regions of diaphragms and 1.0 elsewhere. Coefficients used for design of wall panels themselves need not be increased.

2. For diaphragms with actual shear strengths that substantially exceed the code C_p requirement, the C_p coefficients recommended above should be increased by the ratio of actual shear strength to code required shear strength. This increase probably need not exceed a factor of 1.5, based on the expected strength of ground motion.

3. The use of anchor devices subject to brittle fracture across weak net sections of limited length, such as bolt and rivet holes should be avoided, as should hardware limited by the capacity of bolted connections to wood framing. Hardware that permits controlled yielding rather than brittle fracture should be used when possible.

4. The use of anchorage devices that are limited in strength by the pull-out capacity of portions of the anchor embedded in concrete or masonry should be avoided.

6. PERFORMANCE OF STEEL MOMENT FRAME BUILDINGS

To date (end of 1994), damage has been found in over 100 steel moment resisting frame buildings caused by the Northridge Earthquake. The majority of this damage has consisted of fractures of the bottom flange weld between the column and girder flanges, although there have also been a large number of instances where top flange fractures occurred. In some cases, damage at these connections included tearing and cracking of the column flanges and/or web. Tearing of the girder shear tab connection also occurred in a number of instances. Buildings of one to twenty-seven stories have been affected, with the majority in the range below six stories. Most of the damage occurred to structures of recent construction, although buildings up to twenty years old have also experienced weld fractures. The majority of the damaged steel frame buildings are located in the recently-developed San Fernando Valley. Similar, but less extensive, damage has been found in buildings in Santa Monica and West Los Angeles.

Since the data about the damage suffered by these buildings is still being collected, it is difficult to accurately determine the relationships between steel moment frame buildings affected by the Northridge Earthquake and those in other areas. It is likely though, that very similar, if not identical, conditions are common in design and construction practices in other parts of the state and country.

Almost without exception, the connections which failed were of the type prescribed by paragraph 4.F.1.b. of these Recommended Requirements. This type of connection was first specifically prescribed in the 1988 recommendations and in the 1988 U.B.C., but similar connections have been in common use for about 20 years. The history of this type of connection (Welded flanges/bolted web plate) dates originally to the work of Popov and Stephen in the early 1970s (Ref. 4-1). This investigation, using W18x50 and W24x76 beams, compared the performance of all welded connections to that of welded flange/ bolted web connections. Although the investigation showed that the welded flange/bolted web connections did not perform as well as the all welded, and that their performance was somewhat erratic, nonetheless these connections did develop substantial inelastic beam rotations, and were accepted by structural engineers as being adequate for seismic applications. The conclusions drawn from this investigation, coupled with the basic economy of the connection and an inherent developing trust in welding, led to such widespread acceptance by structural engineers and fabricators that the welded flange/bolted web connection became fully accepted and was written into the code. Subsequent tests by Popov with others, and by Englehardt as recently as June of 1992, further demonstrated the variability of performance of these connections. (References to be added.)

Despite the lack of complete reliability of the connection as demonstrated in the tests, the number of failures at Northridge would have to be considered as very unexpected. A number of factors are felt to have contributed to the unexpectedly poor performance:

1. Unreliability of material properties, particularly excessive strength of A36 beams and girders.
2. A basic joint configuration not conducive to ductile behavior.
3. Web connections with bending strength less than that of the web itself.
4. Poor execution and quality control of welding.
5. Unreliable through-thickness strength of column flange material.
6. Large member sizes and large welds.
7. High residual stresses caused by fit-up sequence of highly restrained joints.
8. Possible effects of unusual earthquake characteristics such as large vertical accelerations.

A number of groups concerned with the design and construction of steel moment resisting frame buildings have begun work on determining the causes of the damage. The American Institute of Steel Construction (AISC) convened a task force on March 14, 15, 1994 to investigate the effects of the earthquake. This task force published a preliminary report in May, which included initial recommendations on possible repair and improved design procedures for these structures. Because of the preliminary nature of the available information, *no* official recommendations have been published to date by any committee of SEAOC.

The only post-Northridge testing completed to date is an emergency testing program was initiated by AISC and one building owner to attempt to arrive at detailing and fabrication methods for a particular set of beam and column sizes common to the owner's project which would provide for satisfactory performance. The stated performance goals for the tests were:

A. Ability to achieve plastic rotations of .02 to .03 radian without significant deterioration in strength.

B. No brittle failures.

The tests to date have consisted of quasi-static cyclic loading of eight pairs of full scale beam/column assemblies using heavy columns (W14x426 & W14x455) and large beams (W36x150). One specimen in each pair was made by Fabricator A and the other by Fabricator B. The results can be summarized briefly as follows:

1. Designs with the basic configurations of current designs, using Section 4F.1.b of these recommendations, failed in a brittle manner at rotations of only about a quarter of the minimum acceptable and with very few cycles. This was true even when better than normal welding techniques, removal of backup bars, added fillet welds, full web welding, etc., were included.

2. With a few exceptions, specimens with the beam/column connection area reinforced with additional plates to greatly reduce weld stresses and force the inelastic rotations away from the connection, were successful in meeting the performance goals. The most common and effective contributor to rotation in these tests was flange buckling at the end of the reinforced section. A preliminary recommendation is to utilize reinforcing plates approximately 3/4 of the thickness of the beam flange for connections where both the flange and the reinforcing plate are welded to the column flange. It should be noted when making calculations involving M_p of beams and girders that recent studies have shown that the yield strength of A36 steel is likely to be in the range of 50 ksi as discussed in the Commentary.

3. In several specimens where written welding procedure specifications (*WPS*) were not followed by one fabricator's erection welder, the matching specimen produced by the other fabricator's welder following *WPS* performed significantly better. In practice the *WPS* for prequalified or specially qualified welds are often not available or followed in the field. Many welders operate on their "experience" of what is required to pass ultrasonic tests. This experience method by a well qualified welder is what was used in lieu of WPS in the first set of specimens.

4. Continuity plates, at least half as thick as the beam flanges, are required for most column sizes and continuity plates at least as thick as the beam flanges are considered to be good practice by many structural engineers. The columns in these tests had very thick flanges (3

inches+) which by Section 4.F.4 of these recommendations would not require continuity plates. Where continuity plates were added in the tests, there was no clear indication of improved performance, however, it can be expected that for columns with thinner flanges the effect of added continuity palates would be more pronounced, and they are recommended for most cases even when not required.

5. A preliminary recommendation of AISC was to fully weld webs or web shear tabs to help relieve flange weld stresses. For the reinforced flange connections, the need for, or efficacy of, such welding has not been proven in the specimens tested. Nonetheless, it may be considered as a logical way to further improve the overall strength of the connections.

The above are broad generalizations based on a number of samples too small to be statistically reliable and based on large beam and column sizes not necessarily typical of common practice. Nonetheless, they are meaningful in providing a general direction or anticipation of future testing results. The following observations can be made.

1. The current design practice as described in 4.F.1.b of these Recommendations is unreliable. Apparently, the basic assumption that sufficient yielding can occur in the welds and the immediately adjacent beam flange to provide the needed inelastic rotation is incorrect for many conditions encountered in practice.

2. Reinforcement of beam column joints to significantly reduce stresses in welds and force the inelastic rotations to occur away from the connection is effective in improving performance of the detail and, in most cases, will be effective in achieving the desired performance.

3. Enforcement of appropriate and approved written welding procedure specifications (*WPS*) and improved quality control are also important to achieve the desired performance.

4. Special techniques and materials of welding such as use of smaller wire, use of notch tough wire, removal of backup bars, etc., while probably beneficial, have not been proven to be of sufficient benefit to enable the required joint rotation in unreinforced connections, and may not even be necessary in reinforced joints where weld stresses are reduced. However, removal of backup bars and runoff tabs and grinding smooth at bottom flange welds is recommended at minimum.

The above observations indicate some directions to solving the problem, but address only the joint and connection and may not represent the most effective solution in all cases. Further studies are needed to address the effects of the following:

1. Overall frame action and performance;
2. System redundancy and relative strength;
3. Composite action with concrete slab;
4. Stiff vs. yielding panel zones;
5. Continuity plates;
6. Web or web tab welding;
7. Member sizes;
8. Rate of loading;
9. Axial load on columns as it affects the through-thickness failures of flanges;
10. Special steels for columns with improved through-thickness properties;
11. Special testing to detect flaws in columns, as well as many others.

Another issue for structural engineers to consider is the shear connections of nonmoment frame girders to columns. Considering that many web shear tabs cracked under the rotations imposed on them after the flange welds failed, and recognizing that the nonframe connections must undergo similar rotations, the advisability of beams or girders being rigidly connected to shear tabs by slip-critical bolts or by welds must be evaluated.

What is needed to establish complete criteria for reliable moment frame performance, consistent with performance expectations over the wide range of actual conditions, is a comprehensive program, which will require significant funding and may take years to complete. Meanwhile, any recommendations for moment frame design and detailing must be considered as interim recommendations and the design of moment frame structures should be approached with caution and with recognition that greater than normal design effort and diligence will be required to achieve designs which can be expected to provide the performance needed.

7. STRONG GROUND SHAKING RECORDED DURING THE 1994 NORTHRIDGE EARTHQUAKE

Strong ground shaking was recorded during the 1994 (magnitude M_w = 6.7) Northridge earthquake, particularly at sites located near fault rupture. These near-sources tended to have spectra significantly larger than the design spectrum of the SEAOC Bluebook (Figure 1B, SEAOC 1994). While near-source ground shaking was somewhat stronger than that expected for an M_w = 6.7 earthquake, recorded ground shaking is consistent with both theoretical and empirical-based predictions of strong ground motion expected for large magnitude earthquakes.

The SEAOC spectrum does not envelop ground shaking expected for sites located within 10 - 15 km of fault rupture. The ad hoc Ground Motion Subcommittee of the Seismology Committee is developing recommendations for possible incorporation of a "near-field" factor in the 1997 SEAOC Bluebook (and 1997 *Uniform Building Code*). This near-field factor would be similar to that now required for design of base-isolated buildings.

It is the objective of the Ground Motion Subcommittee to refine current seismic criteria to better reflect the level of earthquake ground shaking expected at a building site. Modification of seismic criteria for building design (e.g., reduction by *R* factors) is the responsibility of the Seismology Committee. If a near-field factor is adopted by the Seismology Committee, *R* factors will likely be adjusted based on expected and observed building performance. Design forces would likely increase to some degree for most buildings located very near faults. Increases in design force would likely be substantial for buildings with lateral force resisting systems that are considered by the Seismology Committee to be potentially poor performers for beyond-Code levels of ground shaking.

Background

The Northridge earthquake of January 17, 1994 in the San Fernando Valley. The epicenter (point on surface above the earthquake's focus) was about 1 mile south-southwest of Northridge or about 20 miles west-northwest of Los Angeles. The moment magnitude of the earthquake is M_w = 6.7.

Analysis of aftershock data indicates that the mainshock fault rupture plane covers an approximately rectangular zone extending about 15 km west-northwest from the epicenter and about 15 km to the northwest-northeast. The fault rupture plane slopes upward at about 45° from a depth of about 18 at the main shock's focus to within about 5 km of the surface. The fault rupture plane slopes upward to the northeast so that areas closest to fault rupture lie roughly along Interstate 5 between Sylmar and Newhall.

Strong ground motion was recorded at a number of stations located within 10 - 15 km of fault rupture. These recording stations include:

California Strong Motion Instrumentation Program (CSMIP),

(1) Arleta (Nordhoff Avenue Fire Station)
(2) Sylmar (Los Angeles County Hospital)
(3) Newhall (Los Angeles County Fire Station)

National Strong-Motion Program (NSMP)

(1) Sepulveda V.A. Hospital (United States Department of Veterans Affairs)
(2) Jensen Filter Plant (Metropolitan Water District of Southern California)

LADWP Power System Strong-Motion Network

(1) Rinaldi Receiving Station
(2) Sylmar Converter Station (SCS)
(3) Sylmar Converter Station East (SCSE)

The CSMIP recording stations are operated by the California Department of Conservation Division of Mines and Geology (CDMG). The NSMP is administrated by the United States Geological Survey (USGS), Branch of Earthquake and Geomagnetic Information, in cooperation with other Federal, State and local agencies. The LADWP Power System Strong-Motion Network is privately owned and operated by the Los Angeles Department of Water and Power.

A summary of ground motion recorded at stations listed above is provided in Table 1. Data presented in this table are taken from CSMIP [CDMG 1994], NSMP [USGS 1994] and LADWP [LRB 1994] reports. Estimates of the closest distance to fault rupture are based on the reported location of the instrument and the approximate location of the mainshock fault plane. Estimates have been rounded to the nearest 5 km reflecting imprecise knowledge of these distances. The data include only records from "free-field" stations (or instruments located at the ground floor of buildings when free-field records are not available). Records from the upper-floors of buildings or from dams are not included.

The records summarized in Table 1 are a fairly complete and representative sample of the strongest shaking recorded during the 1994 Northridge earthquake. One notable omission is the exceptional strong shaking recorded at Tarzana - Cedar Hills Nursery [CDMG 1994]. This station recorded ground motion with peak ground accelerations greater than 1g in each of the three orthogonal directions. Very large accelerations were recorded for at least 10 seconds. Aftershock evaluation verified instrument functionality; however, comparisons with other nearby instruments revealed highly localized amplification at this site. The Tarzana record should not be ignored, but the record should not be considered representative of ground motion recorded during the 1994 Northridge earthquake.

Near-Source Ground Motion - Horizontal Direction

Since 1992, the ad hoc Ground Motion Subcommittee of the SEAOC Seismology Committee has been considering refinements to ground motion criteria. Scope and status of Subcommittee activities have been reported annually at the SEAOC convention [Kircher et al., 1993]. The primary objective of the Subcommittee is to develop seismic criteria that better reflect actual ground shaking expected at building sites. One of the primary considerations of the Subcommittee has been development of a "near-field" factor for sites near earthquake sources. This factor would be incorporated into design values required for equivalent lateral force (static) analysis design and the seismic criteria required for dynamic analysis. At present, a near-field term is required only for design of isolated buildings.

Ground motion recorded during the 1994 Northridge earthquake at sites near fault rupture provide additional and compelling evidence for design consideration of ground motion beyond the level inherent to current SEAOC seismic criteria. As summarized in Table 1, horizontal components of ground acceleration recorded at approximately 5 km from fault rupture have instrumental peak ground acceleration (PGA) values of about 0.7g, on the average, in contrast to the 0.4g "effective" PGA of the SEAOC spectrum.

A more meaningful evaluation of near-source ground motion may be made by directly comparing response spectra. Such comparisons avoid issues related to inherent differences in instrumental and effective values of PGA, and provide better information on ground shaking at periods that most effect building response. Figure 1 compares spectra recorded at three CDMG stations: Arleta, Sylmar and Newhall. For each station, the spectra of the two horizontal components have been averaged together. This averaging tends to smooth the peaks and valleys of individual components (permitting easier comparison with the SEAOC spectrum), but eliminates biases between components due to directivity, and other source-site effects. In most near-field records, one horizontal component of ground shaking tends to be significantly stronger than the other.

As shown in Table 1, the Arleta station was closest of the three stations to the epicenter (10 km), but the farthest from fault rupture (15 km). Both Sylmar and Newhall stations were farther from the epicenter, but significantly closer to fault rupture (5 km). The comparison of spectra of the Arleta, Sylmar and Newhall spectra reflects these differences in distance to fault rupture. Arleta is about one-half to three-quarters of the SEAOC spectrum at periods of interest, while the Sylmar and Newhall spectra are both about 1.5 to 2 times the SEAOC spectrum. Figure 2 compares composite Sylmar/Newhall spectra with the SEAOC spectrum. Both the average and the envelope of the four spectra of individual horizontal components are compared. The average spectrum indicates that ground shaking at Sylmar and Newhall near-field sites typically exceeded the SEAOC spectrum by a factor of about 1.5 to 2 at virtually all periods of interest to building design (i.e. all periods between about 0.5 seconds and 3.0 seconds). The envelope spectrum suggests that motion in the strongest direction of shaking recorded at the critical station exceeded the SEAOC spectrum by about a factor of 2 to 2.5, at all periods of interest to building design.

Building proximity to fault rupture is an important parameter, since ground motion attenuates with distance. Figure 3 illustrates ground motion attenuation from a fault in terms of 1-second response. The trend in ground motion attenuation is similar for other periods. In this figure, empirical-based predictions of attenuation are based on the work of Boore et al. (1993), although the work of other researchers could have been used. Soil type is assumed to be Boore et al. soil type B/C, corresponding to medium soil site conditions (i.e., SEAOC Soil Type S2). Attenuation values are provided for two earthquake magnitudes, M_w = 6.7 and M_w = 7.5. In both cases, attenuation is shown by values that range from the "median" to "+ 1 ls" levels of predicted response. The range of predicted values is large, reflecting the large scatter in measured ground motion.

As shown in Figure 3, the empirical-based predictions of the shaking level drop off significantly with distance. Superimposed on the empirical-based predictions shown in the figure are response values obtained from 1994 Northridge earthquake spectra of six of the near-field records listed in Table 1. Five of the records (Rinaldi, Newhall, Sylmar, SCS and SCSE) are at about 5 km from fault rupture and the sixth record (Arleta) is at about 15 km. Also shown for reference is the value of the SEAOC spectrum for Soil Type S2 at a period of 1 second. This value of the SEAOC spectrum is 0.6 g for Zone 4 and 0.45 g for Zone 3. When multiplied by the building's weight, these accelerations are the same as the design values required for equivalent lateral force design of a 1-second building, before reduction by the Rw factor. Within a given seismic zone, the SEAOC spectrum (and corresponding design values) do not vary with distance from earthquake source(s). As illustrated in the figure, the SEAOC spectrum would be expected to underpredict ground shaking for sites located within about 10 - 15 km of fault ruptures. The near-field records of the Northridge earthquake confirm this expectation.

<u>Near-Source Ground Motion - Vertical Direction</u>

Typically, vertical ground motion is assumed to be about two-thirds of horizontal ground motion. In this sense, the vertical motions recorded during the Northridge earthquake are not exceptional. For free-field stations listed in Table 1, the average vertical PGA is about 0.6 g, in contrast to an average horizontal PGA of about 0.7 g. The important aspect of these values is that both are large, the instrumental vertical PGA is over twice that corresponding to two-thirds of 0.4 g, the effective horizontal PGA of the SEAOC spectrum.

Large vertical PGA's, in themselves, would not necessarily be significant. Typically, the strongest portion of vertical motions tend to shake the site before the strongest portion of horizontal motions can excite the building. Close to fault rupture, however, the strong vertical and horizontal motions can act concurrently on buildings. Near-field records from the 1994 Northridge show significant correlation of vertical and horizontal ground shaking.

The most important aspect of evaluating the effects of vertical earthquake ground shaking on buildings is the determination of the vulnerability of building elements to such motions. Certain buildings have elements, such as the long, vertically-flexible spans of a parking garage, that would be expected to be dynamically excited by vertical earthquake shaking. Vertical (Up component) spectra from a number of near-field locations show peak response in the range of about l to 2 g's. These high response accelerations were found at periods from 0.1 to 0.5 seconds, the range of periods typical of vertical modes of vibration. This level of vertical earthquake loads when combined with building weight would generate short duration loads on elements as much as the supported weight acting upward or as much as 3 times the supported weight acting downward. Buildings that have significant design margin on vertical (beyond the weight of the building) can sustain high, but short-duration, vertical earthquake accelerations without failure. Elements of buildings located near fault rupture that do not have such design margin would be susceptible to vertical-earthquake induced failures.

<u>Duration</u>

The duration of strong ground shaking of the 1994 Northridge earthquake was relatively short, consistent with that expected for a magnitude M_w = 6.7 event. Teleseismic and strong motion data indicate two main subevents occurred, separated in time by about 2 - 3 seconds (with the possibility of a smaller event 4 -5 seconds later). These data suggest the duration of fault rupture was about 6 - 8 seconds, consistent with the 8- to 10-second duration of very strong shaking recorded at near-source sites. With respect to the strongest shaking recorded at sites located at about 5 km from fault rupture, the time histories typically include only one or two cycles of strong velocity pulses that most effect the long-period portion of response spectra.

The relatively short duration strong ground shaking likely reduced the effects of the very high demands placed on buildings located near fault rupture. Buildings were required to sustain only a few cycles of extreme response and properly-designed lateral force resisting systems of most building types generally performed well. Caution must be exercised in extrapolating these favorable observations to earthquakes of larger magnitudes, since the duration of strong shaking would be expected to be much longer.

Table 1: 1994 Northridge Earthquake Ground Motion Recorded Near Fault Rupture [LRB 1994, CDMG 1994, USGS 199]

Recording Station			Site Geology	Distance (km)		Maximum Acceleration	
Name	Owner (Instr. No.)	Location		Epi-center	Fault Rupture	Comp	PGA (g)
Sepulveda VA Hospital	VA (NSMP)	Ground Level		7	10	360°	0.94
						Up	0.48
						270°	0.74
Rinaldi Receiving Station	LADWP (5968)	Free Field	Alluvium	10	5	318°	0.48
						Up	0.85
						228°	0.84
Arleta Nordhoff Ave. Fire Station	CDMG (CSMIP 24087)	Ground Level	Alluvium	10	15	90°	0.35
						Up	0.59
						360°	0.29
Jensen Filter Plant - Generator Building	MWD (NSMP)	Ground Level		12	5	022°	0.56
						Up	0.52
						292°	0.98
Sylmar Converter Station (SCS)	LADWP (306-3)	Free Field	Alluvium	12	5	052°	0.61
						Up	0.64
						142°	0.90
Sylmar Converter Station East (SCSE)	LADWP (6273)	Free Field	Rock	13	5	015°	0.83
						Up	0.38
						285°	0.49
Sylmar (LA County Hospital)	CDMG (CSMIP 24514)	Free Field	Alluvium	16	5	360°	0.91
						Up	0.60
						90°	0.61
Newhall (LA County Fire Station)	CDMG (CSMIP 24279)	Ground Level	Alluvium	20	5	90°	0.63
						Up	0.62
						360°	0.61

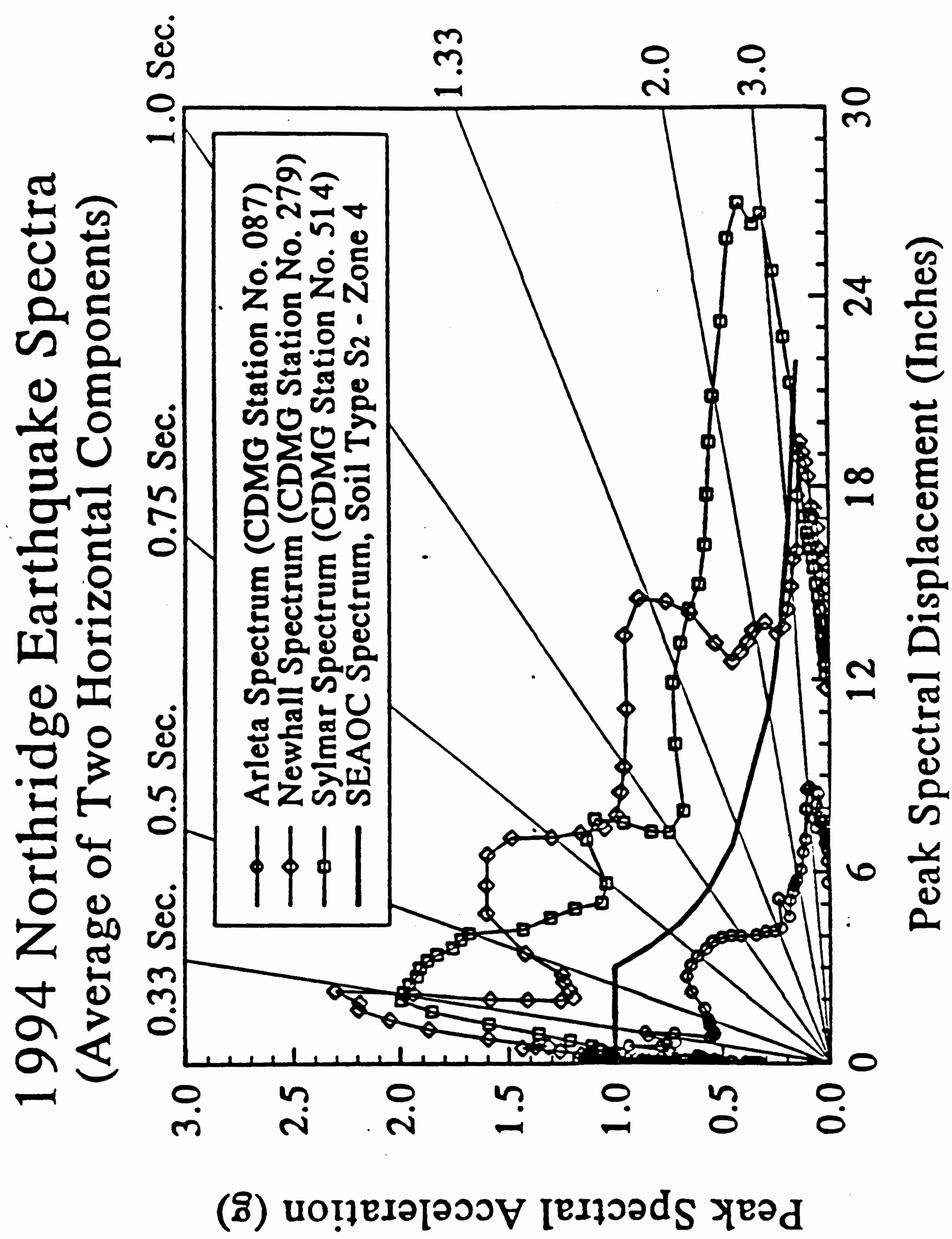

Figure 1: Comparison of Selected Spectra from the Northridge Earthquake

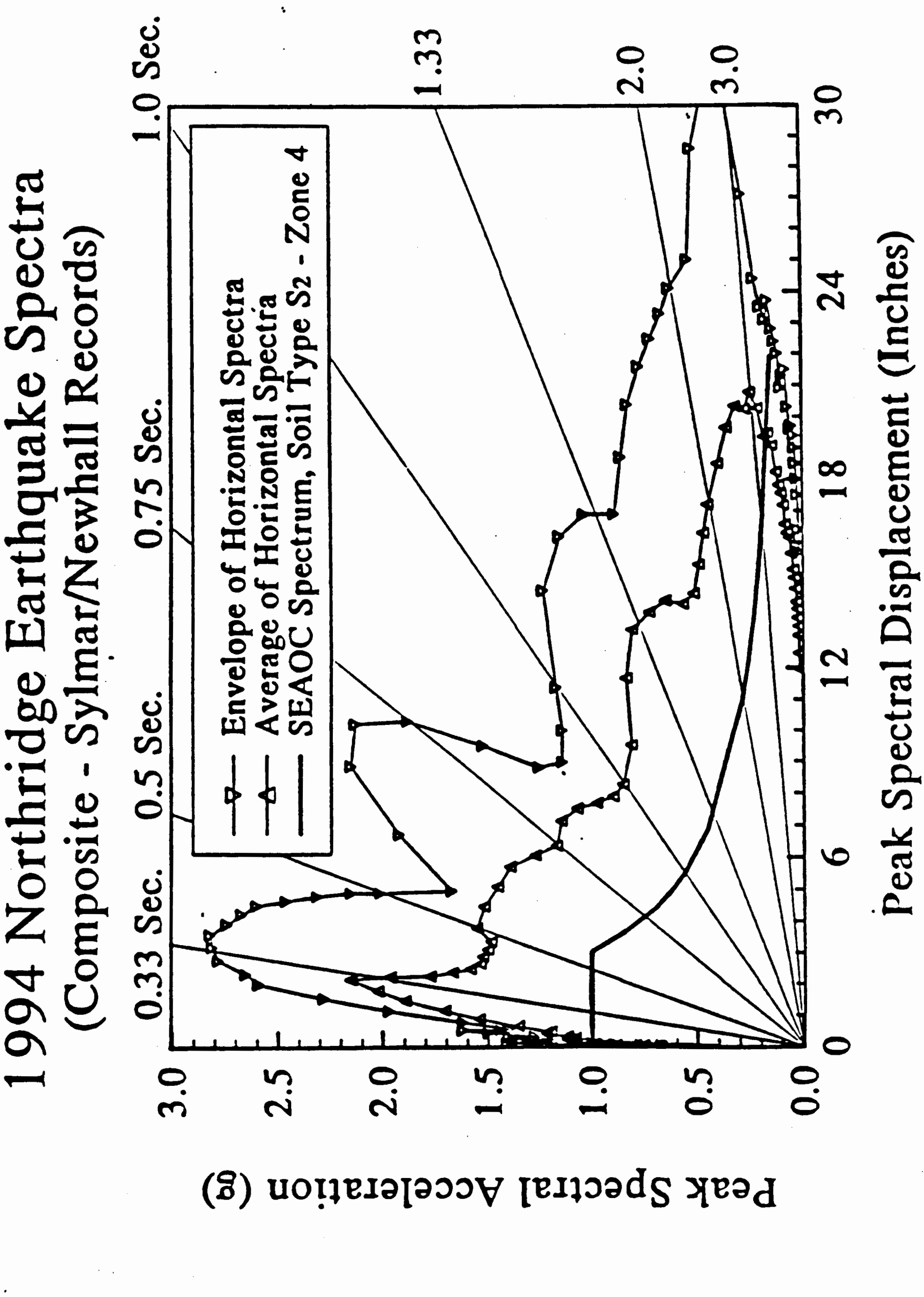

Figure 2: Composite Spectra from the Northridge Earthquake

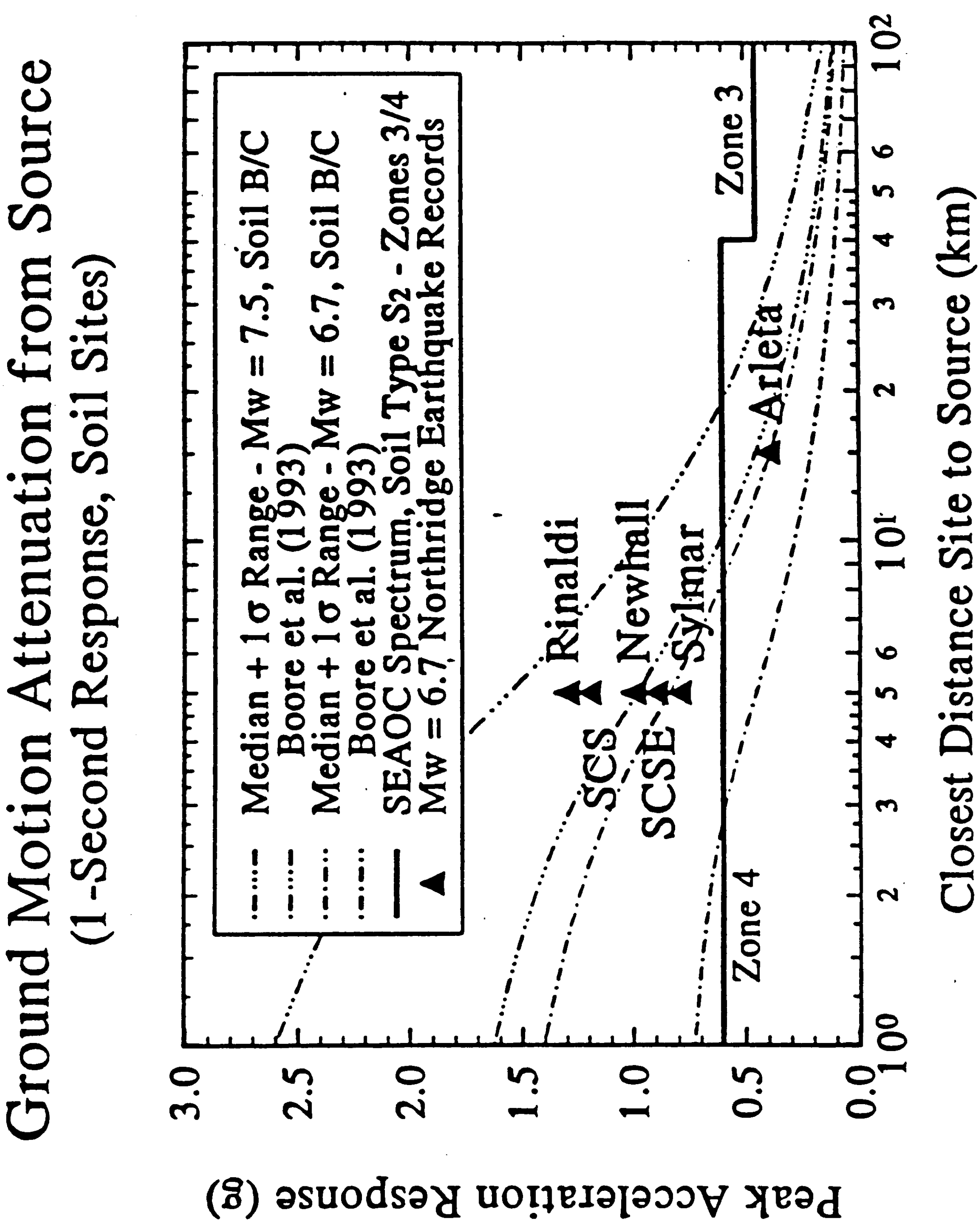

Figure 3: Ground Motion Attenuation from the Northride Earthquake

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APPENDIX D

CODE CHANGES FOR THE 1996 UBC SUPPLEMENT

CODE CHANGE SUBMITTAL

International Conference of Building Officials
5360 South Workman Mill Road
Whittier, CA 90601

July 13, 1994

I. PROPONENT: SEAOC Seismology Committee ____________________
Robert Chittenden, Chair
1050 Fulton Ave., Suite 150
Sacramento, CA 95825
[916) 427-3647

SUBMITTED ON BEHALF OF: STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA

II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 101.2 Purpose, page 1-1, revise as follows:

> **Sec. 101.2 Purpose.** The purpose of this code is to provide minimum standards to safeguard life or limb, health, property, and public welfare by regulating and controlling the design, construction, quality of materials, use and occupancy, location and maintenance of all buildings and structures within this jurisdiction and certain equipment specifically regulated herein. <u>The purpose of the earthquake provisions herein is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function.</u>

Reason:

Section 102 states that "the purpose of this code is to provide minimum standards **to safeguard life or limb, health, property and public welfare** by regulating ...".

On the other hand, the Commentary to the SEAOC "Recommended Lateral Force Requirements", 1988 Ed., upon which the UBC seismic provisions are based, states that the "primary function of these recommendations is to provide minimum standards for use in building design regulation to maintain public safety in extreme earthquakes likely to occur at the building's site. The SEAOC recommendations primarily are intended to safeguard against major failures and loss of life, not to limit damage, maintain function, or provide for easy repair.". Nor do they adequately address containment of toxic/hazardous materials.

Since the seismic provisions do not directly safeguard property [damage], health [containment of toxic/hazardous materials], or public welfare [continued function of essential facilities, disruption of economy, large dollar losses, etc.], the lead-in paragraph should specifically state that the intent of the seismic provisions is different from the intent of the UBC per Section 102.

Future editions of the SEAOC "Recommended Lateral Force Requirements" will specifically recognize the fact that provisions above and beyond the current code provisions are necessary to maintain function and/or minimize property damage. [REF: SEAOC conference proceedings, 1991]

ECONOMIC IMPACT:
Enforcement cost: None

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Sacramento, CA 95825
[916) 427-3647

SUBMITTED ON BEHALF OF: STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA

II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 1631.2.4, Deformation Compatibility, page 2-24, revise as follows:

1631.2.4: Deformation Compatibility. All framing elements not required by design to be part of the lateral force resisting system shall be investigated and shown to be adequate for vertical load-carrying capacity when displaced $3(R_w/8)$ times the displacements resulting from the required lateral forces, <u>computed using the value of *T* as determined by method A of 1628.2.2</u>. <u>When computing $3(R_w/8)$ deflections, the lateral stiffness shall be based on the following:</u>

1. <u>Structural elements only shall be used in the analysis. Stiffness contributions of non-structural elements shall be ignored in the analysis.</u>

2. <u>Section properties of reinforced concrete and masonry elements shall be based on the shear and flexural stiffness at first yield of reinforcing steel. Gross section properties shall not be used.</u>

3. <u>The effects of diaphragm deformations shall be included in the analysis.</u>

4. P- effects on such elements shall be accounted for.

1631.2.4.1: Member Assessment: For designs using working stress methods, member ~~this capacity may be determined using an allowable stress increase of 1.7. The rigidity of adjoining rigid and exterior elements shall be considered as follows: For all designs, the following is required:~~

1. Forces induced in the member by the displacement shall be computed taking into account the stiffening effect of adjoining rigid structural and non-structural elements.

2. Structural or non-structural elements that reduce the effective clear height of the column shall be accounted for in computing the induced forces.Forces induced in concrete or masonry columns shall be computed assuming fixity at top and bottom of the element, regardless of actual fixity or connection.

   Exception: Cantilever elements.

3. ~~**1631.2.4.1: Adjoining Rigid Elements.** Moment-resistant frames may be enclosed by or adjoined by more rigid elements which would tend to prevent the frame from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load-resisting ability of the frame.~~

Remainder of section unchanged

REASON:

A general revision of this section is needed to emphasize the need to properly design elements not part of the lateral system for forces induced by estimated non-linear displacements. Concrete columns in particular are vulnerable to the shears and moments induced by interstory drift in all types of lateral systems. The proposed changes include:

a. The lateral force used to compute the displacements is proposed to be based on the "Method A" period formulas, recognizing that periods based on cracked sections will result in lower lateral forces, hence lower displacements. Because the $3(R_w/8)$ multiplier is in some cases unconservative, conservatism with regard to force level is warranted.

b. Assumptions to be used in computing the stiffness of the structure are proposed. The designer can not use non-structural elements to artificially reduce the displacements. For concrete and masonry materials, cracked section properties are to be used. Because cracked section properties are a function of member curvature, the stiffness at first yield is proposed (a point of defined curvature.) Finally, diaphragm deformations may contribute to deflections of some structural and non-structural elements; this should be reflected in the analysis.

c. In assessing individual members, additional controlling assumptions are specified. Rigid elements adjoining structural elements that produce higher internal forces in the structural element must be accounted for in analysis. For example, concrete parapets/curbs are frequently built monolithically with concrete columns. This has the effect of producing shear forces many times those computing ignoring the adjoining elements. This "short column effect" can in some cases result in a loss of vertical load-carrying capacity of primary columns. Similar examples occur in parking garages where ramps are cast monolithically with columns.

d. Furthermore, it has been noted in observed earthquake damage, that severe column damage in short stiff structures is possible. Static force and response spectrum analysis methods can hide this behavior by ignoring possible dynamic effects on the columns, including short duration story drift reversals. By forcing the assumption of fixity at both ends of concrete/masonry columns, a measure of conservatism is added without causing a severe economic penalty. This provision will also render the practice of detailing artificial hinges into these elements impractical.

The effects of deformation *incompatibility* were widely observed after the 1994 Northridge earthquake. Failures of concrete columns caused by incompatibility with the actual drifts that occurred resulted in the collapse or near collapse of many structures.

ECONOMIC IMPACT:
Enforcement cost: Additional plan review time for enforcement of existing deformation compatibility requirements. The changes herein do not affect enforcement costs.

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Robert Chittenden, Chair
1050 Fulton Ave., Suite 150
Sacramento, CA 95825
[916) 427-3647

SUBMITTED ON BEHALF OF: STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA

II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Table 16 - N Structural Systems, page 2-37, delete as follows:

~~[7] Prohibited in Seismic Zones 2, 3 and 4.~~

Also, renumber footnote 8 to 7 thus [7~~8~~]

Reason:

Editorial. Footnote 7 is repeated in Footnote 8.

ECONOMIC IMPACT:
Enforcement cost: None

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1050 Fulton Ave., Suite 150
Sacramento, CA 95825
[916) 427-3647

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II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Table 16-O, HORIZONTAL FORCE FACTOR C_p, Footnote 3, page 2-38, revise as follows:

> [3] Where flexible diaphragms, as defined in Section 1628.6, provide lateral support for walls and partitions, the value of Cp for anchorage shall be increased ~~50 100 percent for the center one-half of the diaphragm span.~~

REASON:

Recorded accelerations on roofs of single story warehouse buildings have exceeded code recognized values. (Celebi et al, "Seismic Response of a Large Span Roof Diaphragm", EERI Spectra, May 1989 & Bouwkamp et al, "Degradation of Plywood Diaphragms under Multiple Earthquake Loading", SMIP90 Proceedings, May 1991). It has been observed that accelerations at the center of roof diaphragm spans can reach 3 times the peak ground acceleration (PGA), while accelerations at the ends of these spans varies from 1.1 to 1.4 times the PGA. This means that

accelerations in excess of the 0.3g value assumed in the UBC have occurred.
Concrete and/or masonry walls are typically anchored to the diaphragm using metal strap ties of various types embedded in the concrete and attached to wood roof framing using nails or bolts. These devices and the wood fasteners are designed using a factor of safety ranging from 1.5 to 1.7 on the allowable design value. This means that wall anchorage strengths of about 1.7*0.3g=.51g do not compare well with recorded accelerations of 0.8 to 1.2g's. Many examples of failed code compliant wall anchors were noted in the 1994 Northridge earthquake.

In the 1994 UBC, Cp for these elements was increased 50% in the center half of the diaphragm span to 1.125, yielding a design force of 0.45Wp in seismic zone 4 (Ip=1.0). This proposal increases Cp to 1.5 in the center half span, yielding a design force of 0.6Wp in seismic zone 4. Using the industry standard factor of safety of 1.7, this results in a strength of 1.0*Wp for the anchorage.

ECONOMIC IMPACT:
Enforcement cost: None

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Sacramento, CA 95825
[916) 427-3647

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II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 1701.5, Types of Work, Item 16, page 2-45, add new item as follows:

16. **Wood *structural panel* shear walls and diaphragms** . *After installation and before covering,* all connections in ~~structures~~ ~~*wood structural apnel shera walls ans diaphragms*~~ ~~in Seismic Zones 3 and 4, including~~ ~~*fasteners*~~ ~~nailing, tiedowns, framing clips, bolts and straps.~~

    Exception: *Fasteners* ~~Nailing inspection may be ommitted whear shear vlues do not exceed two thirds of the values given in Tables 23-I-J-1, 23-I-J-2, 23-I-K-1 AND 23-I-K-2.~~

REASON:

Post-earthquake inspections of wood frame structures have revealed structural failures of shear walls and diaphragms because of missing or poorly installed connection devices or nailing. The additional requirement for special inspection for these critical items should insure that the hardware and nailing shown on the approved design drawings gets incorporated into the work.

ECONOMIC IMPACT:
Enforcement cost: Minimal

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1050 Fulton Ave., Suite 150
Sacramento, CA 95825
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II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 1701.5 Types of Work, page 2-43, revise Item 3. as follows:

3. **Special moment-resisting concrete frame.** ~~As required by Section 1921.9 of this code. For moment frames resisting design seismic load in structures within Seismic Zones 3 and 4, *the special* a specially qualified inspector *shall* who will provide reports to the person responsible for the structural design *and* shall provide continuous inspection of the placement of the reinforcement and concrete and shall submit a certificate indicating compliance with the plans and specifications.~~

Section 1921.9 Inspection, page 2-247, delete section as follows:

~~**1921.9 Inspection.** For moment frames resisting design seismic load in structures within Seismic Zones 3 and 4, a specially qualified inspector who will provide reports to the person responsible for the structural design shall provide continuous inspection of the placement of the reinforcement and concrete and shall submit a certificate indicating compliance with the plans and specifications.~~

Reason:

Editorial change. It is preferred to collect all of the inspection requirements in Chapter 17.

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Sacramento, CA 95825
[916) 427-3647

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II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 1703 - NONDESTRUCTIVE TESTING, page 2-46, revise as follows:.

Section 1703 - NONDESTRUCTIVE TESTING

In Seismic Zones 3 and 4, welded *fully restrained* connections between the primary members of <u>ordinary moment frames and</u> special moment-resisting frames shall be tested by nondestructive methods for.........

Reason:

The January 1994 Northridge earthquake demonstrated that some problems existed with complete penetration groove welds in beam to column connections of all types of steel moment frames, both ordinary and special. Fractured connections have been discovered at the complete penetration groove weld of the bottom flange to column connections [and occasionally the top flange to column connection]. These proposed additional testing requirements are necessary to supplement existing welding inspection and testing requirements in order to minimize similar behavior in future earthquake events.

ECONOMIC IMPACT:
Enforcement cost: minimal.

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1050 Fulton Ave., Suite 150
Sacramento, CA 95825
[916) 427-3647

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II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

1909.3 Design Strength, pages 2-160 and 2-161, revise as follows:

1909.3.4 *In Seismic Zones Nos. 3 and 4,* strength reduction factors shall be as given above except for the following:

1909.3.4.1 *The shear strength reduction factor shall be 0.6 for the design of walls, topping slabs used as diaphragms over precast concrete members and structural framing members, with the exception of joints, if their nominal shear strength is less than the shear corresponding to development of their nominal flexural strength.* <u>*The nominal flexural strength shall be determined corresponding to the most critical factored axial loads and including earthquake effects.*</u> *The shear strength reduction factor for joints shall be 0.85.*

Section 1909.3.4.2, delete as follows:

~~1909.3.4.2 The strength reduction factor for axial compression and flexure shall be 0.5 for all frame members with factored axial compressive forces exceeding ($A_g f'_c/10$) if the transverse reinforcement does not conform to Section 1921.4.4.~~

Reason:

The sentence inserted in Section 1909.3.4.1 exists in both the '89 ACI Code and the '90 Blue book. The omission from the UBC was in error. The missing sentence needs to be in the UBC as it points the designer to the conditions necessary to determine strength.

The reason for deletion of Section 1909.3.4.2 is that this section allows an inadequate trade-off between strength and ductility in concrete frames. Specifically columns in lateral moment resisting frames that do not conform to the transverse reinforcement requirements of 1921.4.4 shall be designed for a strength reduction factor of 0.5. The seismic performance enhancement provided by the transverse reinforcement is much larger than the enhancement from the moderately reduced strength factor. Additionally, columns without the transverse reinforcement of Section 1921.4.4 performed poorly in the Northridge Earthquake, particularly those carrying large axial loads.

ECONOMIC IMPACT:
Enforcement cost: Additional plan review time for enforcement of existing deformation compatibility requirements. The changes herein do not affect enforcement costs.

CODE CHANGE SUBMITTAL

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5360 South Workman Mill Road
Whittier, CA 90601

July 13, 1994

I. PROPONENT: SEAOC Seismology Committee
Robert Chittenden, Chair
1050 Fulton Ave., Suite 150
Sacramento, CA 95825
[916) 427-3647

SUBMITTED ON BEHALF OF: STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA

II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 1914.8. Alternate Design Slender Walls, page 2-212, add a new sub-section as follows:

> <u>Sec. 1914.8.2.5 All diaphragm-to-wall panel connections shall be attached to, or hooked around, reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.</u>

Reason:

A prevalent type of failure in Simi Valley, as a result of the Northridge earthquake, to post-1973 tilt-up concrete structures, was the pull out of "PA" type anchors which spalled the fascias. A simple revision may prevent this type of failure by using existing code language [Section 1631.2.4.2.6]. This language is currently only applicable to non-bearing, non-shear wall panels; thus tilt-ups have been inadvertently left out. This change ensures that wall anchors are positively anchored in the concrete wall.

ECONOMIC IMPACT:
Enforcement cost: None

CODE CHANGE SUBMITTAL

International Conference of Building Officials
5360 South Workman Mill Road
Whittier, CA 90601

July 13, 1994
Revised Sept 6, 1994

I. PROPONENT: SEAOC Seismology Committee
Robert Chittenden, Chair
1050 Fulton Ave., Suite 150
Sacramento, CA 95825
[916) 427-3647

SUBMITTED ON BEHALF OF: STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA

II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 1921.4.5, Shear strength requirements, page 2-239, add a new sub-section as follows:

1921.4.5.3 For circular columns, the area used to compute V_c shall be 0.8 A_g. The shear strength V_s provided by the circular transverse reinforcing shall be computed by

$$V_s = \frac{A_b f_{yh} D'}{2s}$$

where A_b is the area of the hoop or spiral bar of yield strength f_{yh} with pitch s and *hoop* diameter D'.

REASON:

The code does not address the application of Equation 11-17 to compute shear reinforcment for circular members. The contribution of spiral or hoop reinforcement to shear resistance is not as effective as straight bars typically used in rectangular sections. The maximum bar force exposed by a potential joint crack is not parallel to the applied shear force except at the column centerline. The proposed equation is taken from Priestly, et al. REF: *Seismic Shear Strength of Circular Reinforced Concrete Columns*, by Ang Beng Ghee, M.J.N. Priestly, and T. Pauley, ACI Structural Journal, Jan-Feb 1989.

ECONOMIC IMPACT:
Enforcement cost: Additional plan review time for enforcement of existing deformation compatibility requirements. The changes herein do not affect enforcement costs.

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II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 1921.6.11 Minimum thickness of diaphragms., page 2-245, revise as follows:

> **1921.6.11 Minimum thickness of diaphragms.** Diaphragms used to resist prescribed lateral forces shall not be less than 2 inches (51 mm) thick. The thickness of t~~Topping slabs placed over precast floor and roof elements shall not be the greater of less than 2-1/2 inches (64 mm) 3 inches (76 mm) thick or 6 d_b, where d_b is the nominal diameter of the largest mild steel reinforcing bar laid in the slab~~.

Reason:

Thin topping slabs over precast concrete members performed poorly during the January 1994 Northridge Earthquake. The slabs had excessive deformations because of low stiffness. In addition, there was inadeqaute cover to develop or splice mild steel reinforcing, which is typically laid in these slabs. The minimum cover required to develop the strength of mild steel reinforcing is 2.5 d_b.

ECONOMIC IMPACT:
Enforcement cost: None.

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[916) 427-3647

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II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 2108.2.4 Wall design for out-of-plane loads., page 2-337 and 2-338, add a new sub-section as follows:

> **Sec. 2108.2.4.7 Diaphragm-to-wall connections.** All diaphragm-to-wall panel connections shall be attached to, or hooked around, reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

Reason:

A prevalent type of failure in Simi Valley, as a result of the Northridge earthquake, for post-1973 concrete block structures, was the pull out of "PA" type anchors which spalled the fascias. A simple revision may prevent this type of failure by using existing code language [Section 1631.2.4.2.6]. This language is currently only applicable to non-bearing, non-shear wall panels; thus concrete block walls have been inadvertently left out. This change ensures that wall anchors are positively anchored in the concrete block wall.

ECONOMIC IMPACT:
Enforcement cost: None

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[916) 427-3647

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II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 1628.7.2 Seismic Zones 3 and 4., Item 2., page 2-19, revise as follows:

2. Such columns shall be capable of carrying the above described axial forces without exceeding the axial load strength of the column. For designs using working stress methods this capacity may be determined using an allowable stress increase of 1.7. <u>The additional one third allowable stress increase per Section 1603.5 is not applicable.</u>

Section 2211.4.2 Member Strength, page 2-359, last paragraph - add the following:

2. **Member Strength.** Where this section requires the strength of the member to be developed, the following /shall be used:

	STRENGTH
Flexure	$M_s = Z F_y$
Shear	$V_s = 0.55 F_y d t$
.	.
.	.
.	.
Connectors	

Full-penetration welds	F_yA
Partial penetration welds	1.7<u>xF_s</u> ~~allowable~~
Bolts and fillet welds	1.7<u>xF_s</u> ~~allowable~~

<u>where F_s is the allowable stress value defined in the applicable chapter of Section 2251. The allowable stress values specified in Section 2251 shall not be increased by the one third allowable stress increase per Section 1603.5.</u>

Members need not be compact unless otherwise required by this section. **note; ses 1628.7.2 pg 2-109**

REASON:

Make clear that the allowable stress increase per Section 1603.5 is <u>not</u> to be used with the member strength values. Some users seem to be confused by tis section, particularly the values for welds which refer to "allowable".

ECONOMIC IMPACT:

Enforcement cost: None.

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Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 2211.7.1.1 Required Strength, page 2-360, revise as follows:

The girder-to-column connection shall be adequate to develop the lesser of the following:

1. <u>1.75 times</u> ~~Tthe strength of the girder in flexure calculated at the location of the plastic hinge.~~

2. The moment corresponding to development of the panel zone shear strength as determined by Formula (11-1) <u>but not less than 1.5 times the strength of the girder in flexure</u>.

Reason:

Performance of SMRF joints in the 1994 Northridge Earthquake and subsequent tests at the University of Texas indicate that the current joint design requirements and construction methods are inadequate to develop the required post-elastic deformations because of premature failure of the girder flange to column flange welds. Requiring that the connection develop a minimum of 1.75 times the girder flexural strength will provide a mechanism to shift the plastic hinge away from the weld at the face of the column flange. Overstrength of the steel and strain hardening exacerbate the problem. Mean yield strengths for A-36 [F_y = 36 ksi] steel are approximately 49 ksi. Strain hardening contributes approximately 25% more to the steel tensile strength. The product of mean overstrength and strain hardening is approximately 1.75.

Section 2211.7.1.2 Connection Strength, page 2-361,delete and renumber the remaining sections.

Section 2211.7.1.3 Alternate Connection Strength, page 2-361, renumber, retitle, and revise section as follows:

> **2211.7.1.~~3~~ 2 ~~Alternate Connection Strength.~~** ~~Connection configurations utilizing welds or high-strength bolts not conforming with Section 2211.7.1.2 may be used if they are shown by test or calculation to meet the criteria in Section 2211.7.1.1. Where conformance is shown by tests, the referenced tests shall be substantially similar, to the satisfaction of the building official, to the connection detail and include cyclic test results. Where conformance is shown by calculation, 125 percent of the strengths of the connecting elements may be used 1.5 times the strength required by Section 2211.7.1.1. shall be provided.~~

Reason:

Examination of damaged beam/column connections following the Northridge Earthquake and recent tests of welded flange/bolted web connections suggest that the currently accepted beam/column joint is unable to reliably develop adequate levels of post-elastic deformation. A number of suggested alternatives have been proposed and tested to shift the location of the plastic hinge away from the complete penetration groove welds at the column face and into the span where large post elastic strains can be developed. The proposed change requires demonstration of the adequacy of the proposed connection by test or calculation. When compliance is by calculation, additional capacity is required by requiring 1.5 times the strength specified in 2211.7.1.1. The current requirement of 125% of the connecting element strengths should be discontinued so that there is incentive to use testing to establish the adequacy of the connection.

Add a new section as follows:

2211.7.1.4 Welding Requirements. When welding is used to develop the required strength of the girder-to-column connection, the welding shall conform with the following:

1. The welding requirements in AWS D1.1 as modified in Division VIII or Division IX.

2. The special inspection requirements in Section 1701.

3. A welding procedure specification, with the information required by AWS D1.1, acceptable to the engineer of record and furnished to the building official. The welding procedure specification shall be used inn providing the required special inspection.

4. The filler metal used shall have a notch toughness not less than of 20 ft-lbs at 70 degrees F as measured by a standard Charpy V-notch test, ASTM E-23, in accordance with the applicable filler metal specification referenced in AWS D1.1.

Reason:

In addition to revising the joint design to develop a reliable plastic hinge, testing following the Northridge Earthquake has demonstrated the necessity of applying an approved welding procedure specification that meets the strict requirements of AWS D1.1.

2211.7.9 Changes in beam flange area., page 2-263, revise as follows:

Abrupt changes in beam flange area are not permitted within the possible plastic hinge regions of special moment resisting frames except as required by Section 2211.7.1.1.

Reason:

This removes potential conflicts between the proposed changes in 2211.7.1.1 with this section, which does not currently permit changes in beam flange area in regions subject to plastic hinging.

ECONOMIC IMPACT:
Enforcement cost: Initially there will be an increassed cost for testing while a national testing program to requalify joint designs is completed. If this national testing program, which has a high priority, is completed prior to adoption of this change by local jurisdictions, then no impact.

CODE CHANGE SUBMITTAL

International Conference of Building Officials
5360 South Workman Mill Road
Whittier, CA 90601

July 13, 1994
Revised Sept 6, 1994

I. PROPONENT: SEAOC Seismology Committee
Robert Chittenden, Chair
1050 Fulton Ave., Suite 150
Sacramento, CA 95825
[916) 427-3647

SUBMITTED ON BEHALF OF: STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA

II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 2211.8.2.5, Compression elements in braces, page 2-364, revise as follows:

2211.8.2.5: **Compression elements in braces:** The width-thickness ratio of stiffened and unstiffened compressions elements used in braces shall be as shown in Division IX., Table B5.1, for compact sections.

The width-thickness ratio of angle sections shall be limited to $52/f_y$ (For **SI:** $0.31E/f_y$). Circular sections shall have outside diameter-wall thickness ratio not exceeding $1{,}300/F_y$ (For **SI:** $7.63\ E/f_y$, rectangular tubes shall have outside width-thickness ratio not exceeding $110/F_y$ (For **SI:** $0.65E/F_y$).

Exception: Compression elements stiffened to resist local buckling

REASON:

The limit on width-thickness ratio for tube members was limited to 110/Fy for special concentric braced frames, adopted in the 1994 UBC. Fracture of tube braces in ordinary concentrically braced frames was observed in the 1994 Northridge earthquake, believed to be a result of thin walled tubes. The change limits width-thickness ratios to a consistent value for all tube steel bracing members. The proposed language is identical to section 2211.9.2.4.

~~**TABLE 16 N, STRUCTURAL SYSTEMS**, page 2-37, add footnote 4 to items 2.4.a, 4.3.a, and 4.3.b as follows:~~

2. Building Frame System	4. Ordinary braced frames ~~a. Steel[4]~~	8	160
3. Moment-resisting frame system	...		
4. Dual Systems	3. Ordinary braced frames ~~a. Steel with steel SMRF[4]~~ ~~b. Steel with steel OMRF[4]~~	~~10~~ ~~6~~	~~N.L.~~ ~~160~~
5. Undefined systems	...		

~~REASON:~~

~~Footnote 4 currently reads "Prohibited in Seismic Zones 3 and 4". This proposed code change prohibits the use of ordinary concentrically braced frames in Seismic Zones 3 and 4.~~

~~All concentrically braced frame systems reach a limiting load whereby the steel bracing buckles. In the ordinary concentrically braced frame system, cyclic buckling and elongation of the brace may lead to fracture of the brace material thereby locally eliminating the primary lateral load resisting system. In the special concentrically braced frame system, the detailed requirements of Section 2211.9 are intended to prevent local fracture of the brace following cyclic buckling behavior. Braces in a special concentrically braced frame system are~~

~~proportioned to buckle in a controlled manner and thus retain their load carrying capacity upon further load cycles. The prohibition of ordinary concentrically braced frames is intended to provide further assurance of life safety protection for concentrically braced frame systems.~~

ECONOMIC IMPACT: The special concentrically braced frame system does not inflict any economic penalty on the cost of the structural system over that of the ordinary braced frame.

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II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 1628.7.2 Seismic Zones 3 and 4., Item 2., page 2-19, revise as follows:

- 2. Such columns shall be capable of carrying the above-described axial forces without exceeding the axial load strength of the column. For designs using working stress methods this capacity may be determined using an allowable stress increase of 1.7. The additional one third allowable stress increase per Section 1603.5 is not applicable.

Section 2211.4.2. Member Strength., page 2-359, first paragraph, revise as follows

2211.4.2 Member Strength. Where this section requires the strength of the member to be developed, the following shall be used:

	STRENGTH
Moment	$M_s = Z F_y$
Shear	$V_s = 0.55 F_y d t$
.	.
.	.
.	.
Connectors	
Full-penetration welds	$F_y A$
Partial penetration welds	1.7 x F_s ~~allowable~~

Bolts and fillet welds 1.7 x F_s ~~allowable~~

where F_s is the allowable stress value defined in the applicable chapter of Section 2251. The allowable stress values specified in Section 2251 shall not be increased by the one third allowable stress increase per Section 1603.5.

Members need not be compact unless otherwise required by this section.

REASON:

Make clear that the allowable values are the values per Section 2251 and that the allowable stress increase from Section 1603.5 is not to be used with the member strength values. Some users seem to be confused by this section, particularly the values for welds and bolts which refer to "allowable".

ECONOMIC IMPACT:

Enforcement cost: None

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II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Section 2311.3 Nails and Spikes, page 2-820, revise as follows:

Section 2311.3.1 Safe Lateral Strength. A common wire nail ... set forth in Table 23-I-G.

A wire nail driven parallel to the grain of the wood ... to the grain. Toenails shall ...perpendicular to grain. <u>For structures in Seismic Zones 3 and 4, toe nails shall not be used to transfer lateral forces in excess of 150 punds per foot from diaphragms to shear walls, drag struts [collectors], or other *seismic shear transfer* elements, nor shear walls to other elements.</u>

Reason:

The typical shear transfer mechanism from diaphragms using toenails is per the following sketch 1a. As the blocking shrinks, the toenails often split the block or provide a weakened plane for splitting. Even if predrilled, the toenails cannot begin to sustain the load value when cycled in shear at high loads. **[Note: cyclic testing has not been performed on toe nail joints transferring shear]**

A much better shear transfer mechanism is the use of shear transfer blocking or metal angles such as a Simpson A34, as shown in Sketch 1b and 1c. The exeption cvalue of 150 pounds/foot allows for conventional residential construction to continue current practices.

ECONOMIC IMPACT:
Enforcement cost: None.

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II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Table 23-I-I, page 2-897, revise as follows:

TABLE 23-I-I -- MAXIMUM DIAPHRAGM DIMENSION RATIOS

Material	HORIZONTAL DIAPHRAGMS	VERTICAL DIAPHRAGMS
	Maximum Span-Width Ratios	Maximum Height-Width Ratios
1. Diagonal sheathing, conventional	3:1	~~2:1~~ 1:1[1]
2. Diagonal sheathing, special	4:1	~~3 1/2:1~~ 2:1[2]
3. Wood structural panels and particleboard, nailed all edges	4:1	~~3 1/2:1~~ 2:1[2]
4. Wood structural panels and particleboard, blocking omitted at intermediate joints.	4:1	[3]

[1] In Seismic Zones 0, 1, and 2, the maximum ratio may be 2:1.
[2] In Seismic Zones 0, 1, and 2, the maximum ratio may be 3 1/2:1.
[3] Not permitted.

Reason:

Problems occurred with plywood, plaster (stucco), and gypsum board (drywall) shear walls in the January 1994 Northridge Earthquake. Specifically, these walls performed poorly in numerous multi-story apartments and condominiums. Part of this poor performance was caused excessive deformations of narrow shear walls with high height/width [aspect] ratio. The allowable values in the code are extrapolated from test data on walls with aspect ratios of 1:1 and 2:1.

Recent testing and UCR on plywood walls with aspect ratios of 3 1/2:1 show deformations far in excess of those anticipated by the previously extrapolated test data. Recent testing by Simpson Strong-Tie at UCSD indicate extremely high story drifts are associated with the tall, narrow shear panels. For example, at code allowable loads, the actual drift in a 27.5 inch by 8 foot high wall exceeds two inches at the top. This indicates that such hiugh aspect ratios are inconsistent with code allowable values for shear walls in all zones and under any type of lateral force.

Also, these high aspect ratio walls have not been rated to a consistent standard that includes a cyclic loading test protocol that closely approximates dynamic seismic demands. As a consequence, shear wall deformations not been properly evaluated and factored into design methods for these high aspect shear walls.

The proposed revision is an interim revision until testing and loading protocols and acceptance criteria are completed and shear walls evaluated according to theses protocols and criteria. These proposed revisions are intended to minimize similar behavior in future earthquake events in the interim.

Reference: "Cyclic Performance of Tall-Narrow Shear Wall Assemblies", Commins, Alfred, D. and Gregg, Robert, C., 1994

ECONOMIC IMPACT:

Enforcement cost: None.

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II. SUBMITTAL

Suggested revision to the **1994 UNIFORM BUILDING CODE**

Table 25-I, page 1-354, revise as follows:

Table 25-I-- ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR VERTICAL DIAPHRAGMS OF LATH AND PLASTER OR GYPSUM BOARD FRAME WALL ASSEMBLIES[1]

TYPE OF MATERIAL	THICKNESS OF MATERIAL	WALL CONSTRUCTION	NAIL SPACING[2] MAXIMUM (In Inches)	SHEAR VALUE	MINIMUM NAIL SIZE[3,4]
1. Expanded metal; or woven wire lath and portland cement plaster (6)	7/8"	Unblocked	6	~~100~~ 90	No. 11 gauge, 1-1/2" long, 7/16" head, furred a minimum of 1/4" ~~No. 16 gauge staple, 7/8" legs~~
2. Gypsum lath	3/8" Lath and 1/2" Plaster	Unblocked	5	~~100~~ 90	No. 13 gauge, 1-1/8" long, 19/64" head, plasterboard blued nail
~~3. Gypsum sheathing board~~	~~1/2" x 2' x 8'~~	~~Unblocked~~	~~4~~	~~75~~	~~No. 11 gauge, 1-3/4" long, 7/16" head, diamond point, galvanized~~
	~~1/2" x 4'~~	~~Blocked~~	~~4~~	~~175~~	
	~~1/2" x 4'~~	~~Unblocked~~	~~7~~	~~100~~	
~~4. Gypsum wallboard or veneer base~~	~~1/2"~~	~~Unblocked~~	~~7~~	~~100~~	~~5d cooler or wallboard~~
			~~4~~	~~125~~	
		~~Unblocked~~	~~7~~	~~125~~	
			~~4~~	~~150~~	
	~~5/8"~~	~~Unblocked~~	~~7~~	~~115~~	~~6d cooler or wallboard~~
			~~4~~	~~145~~	
		~~Blocked~~	~~7~~	~~145~~	
			~~4~~	~~175~~	
		~~Blocked Two ply~~	~~Base Ply 9 Face Ply 7~~	~~250~~	~~Base ply 6d cooler or wallboard Face Ply 8d cooler or wallboard~~
3. Gypsum sheathing board, wallboard, or veneer base[6]	1/2" or 5/8"	Unblocked or Blocked[5]	7	30	5d or 6d cooler or wallboard

See next page for footnotes

1 These vertical diaphragms shall not be used to resist loads imposed by masonry or concrete construction. See Section 2514.2. Values are for short-term loading

due to wind. Values must be reduced 25 percent for normal loading. ~~The values shown in Items 2 ... Seismic Zones 3 and 4.~~

[2] Applies to nailing at all studs, top and bottom plate, and blocking.

[3] Alternate nails may be used if their dimensions are not less than the specified dimensions.

[4] For properties of cooler or wallboard nails, see Section 2340.1.2.

[5] The maximum wall height to width ratio shall be 1:1.

[6] May not be used to carry lateral [shear] loads at the ground floor of multi-level buildings.

Reason:

Problems occurred with plywood, plaster (stucco), and gypsum board (drywall) shear walls in the January 1994 Northridge Earthquake. In particular, plaster and gypsum board walls performed poorly in numerous multi-story apartments and condominiums. Part of this poor performance was caused excessive deformations of narrow shear walls with high height/width [aspect] ratio. The allowable values in the code are extrapolated from test data on walls with aspect ratios of 1:1. Part of the problem is because the plaster and gypsum board assemblies are only tested with monotonic **[not cyclic]** loads. Additional problems on the plaster walls were caused by construction practices that did not result in walls that matched the prototype walls - e.g., self-furring lath that didn' furr, staples, lack of fastening on top and bottom sill, etc.

Recent testing and UCR and UCSD on plywood walls with aspect ratios of 3 1/2:1 show deformations far in excess of those anticipated by the previously extrapolated test data.
Also, these walls have not been rated to a consistent standard that includes a cyclic loading test protocol that closely approximates dynamic seismic demands. As a consequence, shear wall deformations not been properly evaluated and factored into design methods for these high aspect shear walls.

The proposed revision is an interim revision until testing and loading protocols and acceptance criteria are completed and shear walls evaluated according to theses protocols and criteria. These proposed revisions are intended to minimize similar behavior in future earthquake events in the interim.

ECONOMIC IMPACT: Enforcement cost: None.