ICSSC RP 4

NISTIR 5382

Standards of Seismic Safety for Existing Federally Owned or Leased Buildings

and

Commentary

Diana Todd, editor

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Gaithersburg, Maryland 20899

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United States Department of Commerce
[...]nology Administration
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Interagency Committee on Seismic Safety in Construction Recommended Practice 4
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February 1994
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Gaithersburg, MD 20899

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PREFACE

These *Standards of Seismic Safety for Existing Federally Owned or Leased Buildings and Commentary* have been developed by the Interagency Committee on Seismic Safety in Construction (ICSSC) in response to a mandate from Congress in Public Law 101-614. ICSSC Subcommittee 1, Standards for New and Existing Buildings, developed draft standards for evaluating and rehabilitating existing Federal buildings working with consultants from H.J. Degenkolb Associates and Rutherford & Chekene. The draft documents were balloted by the full ICSSC in the fall of 1993, resulting in this final version.

These standards are published for use in conjunction with a proposed Executive Order on seismic safety of existing Federally owned or leased buildings. The proposed order is expected to require that Federal Agencies:

1. within four years prepare cost estimates for mitigating seismic hazards in all their seismically deficient Federal buildings, and

2. implement a limited program of mitigation of seismic hazards in deficient buildings identified in accordance with the requirements of Sections 2.1 of these Standards.

The intent of these Standards is to identify common minimum evaluation and mitigation measures for all Federal departments and agencies, and to ensure that all federal entities have a balanced, agency-conceived and controlled seismic safety program for their existing owned or leased buildings. The proposed Executive Order directs FEMA to propose an economically reasonable program for further mitigation of seismic hazards in Federal buildings. The cost data to be collected under the requirements of the proposed Executive Order will be used in developing the proposed program.

ABSTRACT

These seismic evaluation and mitigation standards, *Standards of Seismic Safety for Existing Federally Owned Or Leased Buildings and Commentary*, were developed for use by the Federal government by the Interagency Committee on Seismic Safety in Construction (ICSSC) in conjunction with the National Institute of Standards and Technology (NIST). The project was funded by the Federal Emergency Management Agency (FEMA). The intent of this document is to provide Federal agencies with minimum standards for the evaluation and mitigation of seismic hazards in their building inventories.

Substantial Life-Safety is defined as the minimum acceptable performance objective for Federal buildings. FEMA 178, the *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, is taken to be the primary basis for defining this life-safety goal. Four compliance categories are established: structural, nonstructural, geologic/site, and adjacency. Situations which require that an evaluation and if necessary, mitigation, be performed are identified.

These Standards and Commentary include: an identification of situtations which trigger application of the Standards, preliminary and detailed evaluation standards, mitigation standards, and advisory standards for achieving performance objectives beyond Substantial Life-Safety.

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COMMENTARY

STANDARDS

1.0 INTRODUCTION

The intent of these Standards is to provide Federal agencies with minimum standards for the evaluation and mitigation of seismic hazards in their owned or leased buildings. These Standards build upon the work of previous efforts by the Interagency Committee on Seismic Safety in Construction (ICSSC) in support of the National Earthquake Hazards Reduction Program (NEHRP).

1.1 Objectives

The primary objective of these Standards is to reduce the life-safety risk to occupants of Federal buildings and to the general public. The minimum performance objective deemed appropriate for Federal buildings in the long term is Substantial Life-Safety, as defined in Sec. 1.1.1.

1.1.1 Seismic Performance Objectives

The term Substantial Life-Safety, as it is to be used in this document, is defined below. Four additional seismic performance objectives are defined to illustrate the complete range of possible objectives.

> **Fully Functional:** Performance objective where the earthquake causes no damage to facilities and has no effect on building function. Achievement of such performance is beyond the scope and intent of these Standards.
>
> **Immediate Occupancy:** Performance objective where the earthquake causes minor damage, facility disruption is minimal, and only some nonstructural repairs and cleanup will be required. The facility is expected to remain occupied and be functional immediately after the earthquake event.
>
> **Damage Control:** Performance objective where the earthquake damage is controlled in order to protect some other feature of the building or its function beyond life-safety, for example, to control economic loss to the building itself, to prevent the release of toxic materials, or to protect building contents. The term "damage control" covers a range of performance objectives, from protection somewhat greater than that required for Substantial Life-Safety to somewhat less than needed for immediate occupancy.
>
> **Substantial Life-Safety:** Performance objective where the 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. This is the minimum performance objective of these Standards. Compliance with FEMA 178 is assumed to achieve this level of performance.

Risk Reduction: Performance objective where the earthquake damage state is greater than acceptable for life-safety but less than would have occurred in the building if no rehabilitative action had been taken. The extent of damage depends on the extent of the improvements made. As used in these Standards, "risk-reduction" includes incremental strengthening as an interim measure in a total process aimed at achieving Substantial Life-Safety.

1.1.2 Additional Objectives

Federal agencies are encouraged to consider more stringent standards for those buildings where a higher performance objective is necessary to control damage or maintain post-earthquake operation for mission readiness.

It is not the intent of these Standards that existing buildings evaluated or rehabilitated according to the life-safety requirements contained herein be held to more stringent requirements than each agency applies to its new buildings.

1.2 Scope - Hazards

These Standards address potential risk to Federal buildings due to all significant seismic hazards which are defined in terms of four compliance categories (see Glossary for definitions):

- Structural,
- Nonstructural,
- Geologic/site, and
- Adjacency.

The level of seismic hazard within the United States shall be the governing acceleration coefficient as represented on Maps 3 and 4 in the 1991 *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*. For sites outside of the map areas, similarly derived governing acceleration coefficients shall be used. The seismic hazard also may be defined by a site-specific study incorporating more detailed information about a particular site's geology and seismicity. The design level of earthquake ground motion used shall, at a minimum, represent a 10 percent probability of exceedence in 50 years.

1.2.1 Items Not Included in Standards

These Standards do not include provisions for evaluating and if necessary mitigating the potential for damage to Federal buildings due to other hazards including:

- flooding due to failure of off-site facilities,
- non-seismic flooding,
- fire following earthquake,
- wind,
- blast, or
- volcanos.

These Standards also do not address:

- criteria for repair of damaged and deteriorated buildings, including damage caused by previous earthquakes, or
- standards for the preparation of post-earthquake preparedness plans.

1.3 Scope - Buildings

Except for buildings which require a seismic performance objective beyond Substantial Life-Safety because of agency mission requirements, the following buildings are exempt from these Standards:

a. buildings classified for agricultural use, or intended only for incidental human occupancy, or occupied by persons for a total of less than 2 hours a day,
b. detached one- and two- family dwellings that are located in areas having a governing acceleration coefficient less than 0.15 (within the United States, where A_v is less than 0.15 as delineated on Map 4 of the 1991 *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*),
c. special structures including, but not limited to: bridges, transmission towers, industrial towers and equipment, piers and wharves, and hydraulic structures,
d. one-story buildings of steel light frame or wood construction with areas less than 280 m^2 (3000 square feet),
e. fully-rehabilitated buildings which comply with these Standards in all four compliance categories (structural, nonstructural, geologic/site hazards, and adjacency),
f. post-benchmark buildings as defined in Table 1 which also comply with the nonstructural, geologic/site, and adjacency compliance categories,
g. pre-benchmark buildings which have been shown by evaluation to be life-safe in all four compliance categories,
h. buildings constructed for the Federal government whose detailed design was done after the date of the adoption of Executive Order 12699 (January 5, 1990) and that were designed and constructed in accordance with the ICSSC *Guidelines and Procedures for Implementation of the Executive Order on Seismic Safety of New Building Construction*,
i. leased buildings identified in Section 1.3.2 as exempt, or
h. Federally permitted or regulated privately owned buildings on Federal land.

1.3.1 Post-benchmark Buildings

A post-benchmark building is one that was designed and built after the adoption of seismic code provisions which are generally considered to provide acceptable life-safety protection. The determination of benchmark years is complex and varies with building location, age, structural system, and governing building code. An advisory table of benchmark years is provided in Table 1. Based on each agency's mission, facility locations, and construction history, each agency should develop benchmark years for its own use.

TABLE 1: ADVISORY BENCHMARK YEARS

		Model Building Seismic Design Provisions				
FEMA 178[1]	**BUILDING TYPE**	**BOCA**	**SBCC**	**UBC**	**ANSI**	**NEHRP**
1,2	Wood Frame, Wood Shear Panels	**	**	1949	**	**
3	Steel Moment Resisting Frame (MRF)	1987	1991	1976	1982	1985
4	Steel Braced Frame	1990	1991	1988	*	1991
5	Light Metal Frame	*	*	*	*	*
6	Steel Frame w/ Concrete Shear Walls	1987	1991	1976	1982	1985
8	Reinf. Conc. Moment Resisting Frame	1987	1991	1976	1982	1985
9	Reinf. Concrete Shear Walls w/o MRF	1987	1991	1976	1982	1985
10,7	Steel or Concrete Frame w/ URM Infill	*	*	*	*	*
11	Tilt-up Concrete	1987	1991	1973	1982	1985
12	Precast Concrete Frame	*	*	*	*	*
13,14	Reinforced Masonry	1987	1991	1976	1982	1985
15	Unreinforced Masonry (URM)	*	*	*	*	*

[1] The tabulated numbers refer to the 15 common building types as they are defined in FEMA 178.

* Indicates no benchmark year (no comprehensive seismic requirements for these buildings exist).
** Local provisions for wood construction need to be compared to 1949 UBC to determine benchmark year.

BOCA - Building Officials and Code Administrators, *National Building Code*. (BOCA adopted the NEHRP 1991 seismic provisions in a 1992 Addendum to their 1990 edition.)

SBCC - Southern Building Code Congress, *Standard Building Code*. (SBCC adopted the NEHRP 1991 seismic provisions in a 1992 Addendum to their 1991 edition.)

UBC - International Conference of Building Officials, *Uniform Building Code*.

ANSI - American National Standards Institute, A58.1, *Minimum Design Loads for Buildings and Other Structures*.

NEHRP - Federal Emergency Management Agency, *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*.

1.3.2 Leased Buildings

Non-federally owned buildings and portions of such buildings leased by the Federal Government are exempt from these Standards if both of the following apply:

- the leased space is less than 930 m^2 (10,000 square feet),

  and

- the Federal Government leases less than 50 percent of the total building square footage.

The following shall apply to all non-federally owned buildings and portions of such buildings leased by the Federal Government that are not exempt as stated above:

a. no new leases or lease renewals shall be made in buildings that do not comply with these Standards.

Exception: If no seismically conforming space is available, otherwise acceptable space with the best seismic resistance can be taken.

b. existing leases may be held without action until the lease expires.

Exception: For leases of buildings which, in the determination of the leasing agency, present an exceptionally high risk to occupants, appropriate administrative and/or legal action should be employed to end the lease or mitigate the risk as soon as feasible.

1.4 Summary of Standards

Application of the Standards identifies situations which trigger the application of the Standards, defines minimum compliance with the Standards, and identifies additional measures that must be included in each agency's seismic safety responsibilities for existing buildings.

The **Evaluation Requirements** of these Standards include requirements for the preliminary evaluation and detailed evaluation of structural, nonstructural, geologic/site, and adjacency hazards. Guidance on alternate analysis methods, and guidance in developing rehabilitation schemes to aid in mitigation decisions are also included.

The **Mitigation Requirements** of these Standards include the requirements for mitigation of seismic hazards, including standards for rehabilitation of structural, nonstructural, geologic/site, and adjacency hazards, and guidance on incremental or partial rehabilitation, alternative mitigation methods, and rehabilitation of historic buildings.

The **Performance Objectives Beyond Substantial Life-Safety** portion of these Standards includes guidance on which buildings may require seismic performance beyond life-safety and which evaluation and design standards may be appropriate in such cases.

2.0 APPLICATION OF THE STANDARDS

2.1 Situations Requiring Evaluation and Mitigation

At a minimum, a building shall be evaluated and unacceptable risks mitigated when any of the following occur:

a. a change in the building's function which results in a significant increase in the building's level of use, importance, or occupancy, as determined by the agency,
b. a project is planned which significantly extends the building's useful life through alterations or repairs which total more than 50 percent of the replacement value of the facility,
c. the building or part of the building has been damaged by fire, wind, earthquake or other cause to the extent that, in the judgement of the agency, structural degradation of the building's vertical or lateral load carrying systems has occurred,
d. the building is deemed by the agency to be an exceptionally high risk to occupants or the public at large, or
e. the building is added to the Federal inventory through purchase or donation after these Standards are adopted for use by the Federal government.

2.2 Compliance

A building is considered in minimum compliance with these Standards if the building is:

a. exempt from these Standards in accordance with Section 1.3,
b. determined by evaluation to be in compliance with these Standards in accordance with Section 3, or
c. unacceptable seismic risks have been mitigated in accordance with Section 4.

Refer to Section 5 for buildings which are considered in minimum compliance with these Standards as stated in 2.2a and 2.2b above, but which have been deemed by the agency as governed by higher performance objectives.

2.3 Qualifications of Evaluators, Designers, and Reviewers

In general, all evaluations, development of mitigation schemes, and design of rehabilitation work shall be prepared by an engineer qualified to perform the work by registration and/or experience. For independent peer reviews of alternative or innovative evaluation methods, analysis techniques or rehabilitation concepts required by Sections 3.3 and 4.4 of these Standards, an individual highly qualified in the field of earthquake engineering or a panel of such individuals should be selected by the agency. The detailed evaluation of potential geologic/site hazards must be conducted by a geotechnical engineer or engineering geologist qualified to perform the work by registration and/or experience.

2.4 Additional Requirements

As part of each agency's seismic safety responsibilities for existing buildings, the following measures shall be implemented as appropriate:

a. integration of seismic performance objectives higher than life-safety as necessary to carry out agency mission,
b. development of an agency-specific policy for leased buildings consistent with Section 1.3.2,
c. assurance that consistent measures of quality control are applied to all phases of evaluation, design, and construction, and
d. assurance that agency-specific standards and procedures for evaluation and mitigation of hazards are substantially equivalent to or more stringent than the minimum standards contained herein.

3.0 EVALUATION

The purpose of the evaluation is to determine whether life-safety risks exist in a building and, if so, what mitigation options are available. The evaluation covers all four compliance categories outlined in these Standards: structural, nonstructural, geologic/site hazards, and adjacency. FEMA 178, the NEHRP Handbook for the Seismic Evaluation of Existing Buildings, is considered to be the primary basis for this life-safety determination.

3.1 Preliminary Evaluation

A preliminary evaluation shall be conducted in accordance with this section for each of the four compliance categories. Acceptance criteria for each compliance category consist of passing all applicable checklist items. Failure in any one compliance category requires that a detailed evaluation be performed in the corresponding area, in accordance with Section 3.2. Note that the preliminary evaluation may be skipped if a detailed evaluation is performed.

3.1.1 Structural

A preliminary evaluation of a building's ability to satisfy the requirements of the structural compliance category shall consist of, as a minimum, completion of the FEMA 178 "Evaluation Statements for 15 Common Building Types" (in FEMA 178 - Appendix B) for the appropriate structural system. If a building's structural type does not match one of the 15 common building types, then the checklists for the two closest building types shall be used. If, after completion of the FEMA 178 checklist in the preliminary evaluation, any items are non-compliant, a detailed evaluation shall be performed of that item.

3.1.2 Nonstructural

A preliminary evaluation of a building's ability to satisfy the requirements of the nonstructural compliance category shall consist of, as a minimum, completion of the FEMA 178 nonstructural checklist and careful interpretation of the results. Judgement is needed to identify the true nonstructural life-safety risks. As a minimum, the following items, as covered in the FEMA 178 checklists, shall be considered potential life-safety risks:

a. unreinforced masonry partitions 2.5 m (8 feet) tall or more and hollow clay tile partitions, not already excluded by FEMA 178,
b. major mechanical equipment items suspended from the ceiling without bracing,
c. elevators with counter weights not adequately braced and secured,
d. inadequate connections of external cladding elements, glazing and veneer to the structure,
e. unbraced or unanchored parapets, cornices, ornamentation or appendages to the building structure,
f. nonstructural elements that may fail and impede egress, and
g. presence of hazardous materials.

Other nonstructural items listed in the FEMA 178 checklist may be seriously damaged, perhaps beyond repair, in a large seismic event but are not likely to constitute life-safety risks. If, after completion of the preliminary evaluation, any items are identified as potential hazards, a detailed evaluation shall be performed of that item.

3.1.3 Geologic/Site

A preliminary evaluation of the geologic and site hazards at a building site similar to the rapid evaluation procedure included in Appendix A of these Standards shall be completed. (Appendix A is excerpted from ATC-26-1 which was prepared for use by the U.S. Postal Service and is reproduced here by permission of the USPS.) Where preliminary evaluation finds that one or more of the geologic/site hazards are significant at a site, a detailed evaluation shall be performed of that hazard.

3.1.4 Adjacency

A preliminary evaluation of the adjacency conditions at a building site shall consist of, as a minimum, completion for the Federal and the neighboring building(s) of the FEMA 178 "Evaluation Statements for 15 Common Building Types" (in FEMA 178 - Appendix B) for the appropriate structural system of each building, and consideration of the issues identified in Section 3.4 of FEMA 178. In particular, the presence of falling hazards initiated by neighboring buildings, the possible existence of common structural elements such as party walls, and the possible impact of shorter adjacent buildings on the dynamic behavior of the taller building need to be considered. If found to exist, these conditions must be subjected to a detailed evaluation, as described in Section 3.2.4.

3.2 Detailed Evaluation

When preliminary evaluation shows that detailed evaluation is required, or when preliminary evaluation is omitted in favor of detailed evaluation in any or all compliance categories, such detailed evaluation shall be conducted in accordance with this section. Acceptance criteria for each compliance category varies. Failure in any one compliance category requires mitigation of that risk in accordance with Section 4.

3.2.1 Structural

As outlined in FEMA 178, Chapter 1, the evaluation procedures in the FEMA 178 "Procedures and Commentary" sections are to be used to evaluate each item found to be non-compliant when using the FEMA 178 preliminary checklists. Note that FEMA 178 - Sec. 1.3.3.3 suggests that an overall review of the results using good engineering judgement is critical to determining acceptability in many cases.

Other additional or alternate analysis techniques may be employed to clarify whether or not a building poses a risk to life-safety. A post-elastic evaluation that may be used to further judge a building's performance is included as Appendix B of these Standards (prepared for use by the United States Postal Service and included in ATC-26-1 as Appendix F).

3.2.2 Nonstructural

For non-structural items that constitute a life-safety risk (as described in 3.1.2), the specific procedures outlined in FEMA 178 for each item shall be followed for additional analysis. Detailed evaluation is not required for conditions that are not life-safety concerns.

3.2.3 Geologic/Site

A detailed evaluation of the geologic/site hazards that potentially pose a life-safety risk shall consist of completion of the detailed evaluation procedure included in Appendix A of these Standards (excerpted from ATC-26-1, which was prepared for use by USPS) or other site-specific geologic study. Note that existing geotechnical reports or additional geotechnical data such as supplemental subsurface investigations may be needed to complete this evaluation.

Liquefaction is a geologic/site hazard which can occur in areas with saturated, sandy soils. It is important to note that the presence of a liquefaction potential does not necessarily mean that the site poses a risk to life-safety. Good engineering judgement is needed to identify true geologic/site hazards which pose a risk to life-safety.

3.2.4 Adjacency

Appropriate detailed evaluation measures shall be employed to investigate adjacency issues identified as potential life-safety risks as described in Sec. 3.1.4. Possible damage due to pounding, the detrimental effects of the dynamic interaction of buildings of different heights, vertical irregularities, falling hazards initiated by neighboring buildings, common structural elements, and/or other potential adjacency problems shall be investigated as appropriate. As damage due to adjacent buildings can be difficult or impossible to mitigate, the engineer should be certain that the condition poses a significant life-safety risk before recommending that mitigation measures be taken.

3.3 Alternative Analysis Techniques

Alternative analysis methods and techniques which deviate from the specific requirements of these Standards or the documents referenced herein shall be permitted provided it can be shown that a level of Substantial Life-Safety is attained. When innovative analysis techniques are proposed for a specific evaluation of a Federal building, a peer review panel shall determine the adequacy of alternative analysis techniques proposed by the engineer (see Section 2.3).

3.4 Development of Mitigation Concepts

If a building is found to be deficient in any of the four compliance categories, mitigation concepts shall be developed and costs related with each scheme computed to aid in a decision.

Because of the difficulty in mitigating geologic/site hazards and/or adjacency conditions, rehabilitation costs may be prohibitively large. Mitigation methods other than building alteration may need to be considered.

4.0 MITIGATION

4.1 Requirements

All life-safety risks identified in a building shall be mitigated, either by rehabilitation in accordance with Section 4.2, or by some other acceptable method. Alternatives to rehabilitation include, but are not limited to the following:

a. removal of the building from an agency inventory by termination of lease agreements, sale with full disclosure, or demolition,
b. permanent evacuation of the building, or
c. reduction in occupancy of the building such that it becomes exempt in accordance with Section 1.3.

4.2 Minimum Standards and Scope for Rehabilitation

The rehabilitation of any building or site to mitigate seismic life-safety risks, as a minimum, shall be such that the requirements of FEMA 178, as specified in Section 3 of this document, are substantially satisfied after rehabilitation. All required new work shall conform to the agency's standard for new buildings except where the provisions of FEMA 178 are less stringent, such as for seismic demand.

4.2.1 Structural Hazards

The scope of the rehabilitation shall include the repair, removal, replacement, strengthening, or protection of structural elements which are identified as life-safety concerns using FEMA 178. In addition, the relative strengths, stiffnesses and ductilities of all the elements of the lateral load resisting system for the rehabilitated structure shall be analyzed by the engineer for satisfactory behavior in accordance with FEMA 178 - Sec. 1.3.3. Alternate methods of correction of structural hazards which provide an essentially equivalent level of protection may be allowed subject to the provisions of Section 4.4.

4.2.2 Nonstructural Hazards

The scope of the rehabilitation shall include the repair, removal, replacement, strengthening, or protection of nonstructural elements or conditions which the engineer has determined pose a risk to life-safety after a detailed evaluation in accordance with FEMA 178 and discussed in Sections 3.1.2 and 3.2.2. Required work shall conform to the appropriate provisions of the agency's standard for new buildings except where less stringent standards are allowed by FEMA 178.

4.2.3 Geologic/Site Hazards

The scope of the rehabilitation shall include all work necessary to correct the geologic/site hazards which pose a risk to life-safety. Alterations, additions, or reinforcement of the existing structure and foundation to resist the effects of geologic/site hazards shall conform to the appropriate provisions of the agency's standard for new buildings.

4.2.4 Adjacency Hazards

The scope of the rehabilitative work shall include all work necessary to correct the adjacency hazards which pose a risk to life-safety. The work shall conform to the standards cited above for structural and nonstructural elements. The agency shall inform the neighboring property owner of any work in the Federal building which may adversely affect the neighboring building. If an adjacency hazard is corrected, wholly or in part, by the neighboring property owner, the agency should confirm that all work conforms to these Standards in order to consider the risk completely mitigated.

4.3 Incremental/Partial Rehabilitation

Risk-reduction (as defined in Section 1.1.1) by incremental or partial rehabilitation of a building structure is acceptable as an interim step in a complete seismic mitigation process provided that at no time shall the building pose a greater risk to life-safety than in the un-rehabilitated state (except during the actual construction of rehabilitation measures).

4.4 Innovative Mitigation Methods

Innovative mitigation methods which deviate from the specific requirements of these Standards or the documents referenced herein shall be permitted provided it has been shown that a level of Substantial Life-Safety is attained. When new and untested rehabilitation techniques are proposed for a specific building, a peer review panel shall determine the adequacy of the mitigation techniques proposed by the engineer (see Section 2.3).

4.5 Historic Buildings

Historic buildings, in general, shall meet the same minimum life-safety objectives as all other buildings in the Federal inventory and as such, shall not be exempt from these Standards. However, understanding that historic buildings represent a considerable challenge to rehabilitate sensitively, considerable flexibility should be allowed to preserve essential historic features. Existing publications, such as the Secretary of the Interior's Standards and Guidelines for Archaeology and Historic Preservation, shall be used to guide agencies in preserving the historic fabric of these buildings. Alternative methods of evaluation and mitigation of seismic risks for historic buildings shall be allowed subject to the requirements of Section 4.4.

5.0 PERFORMANCE OBJECTIVES BEYOND LIFE-SAFETY

5.1 Identification of Conditions

Although the minimum objective of these Standards is to achieve Substantial Life-Safety in Federal buildings, there are many situations where higher performance objectives such as those discussed in Section 1.1.1 are warranted. The minimum standard, FEMA 178, is currently a consensus document to achieve Substantial Life-Safety when considering seismic risks created by the building and its subsystems. However, in buildings judged adequate by FEMA 178, post earthquake damage may still be extensive and in some cases may not be repairable, and building contents may not be protected. The large variety and many subtle differences in uses and occupancies in Federal buildings makes it incumbent on individual agencies to determine when a higher performance objective is appropriate. Such situations may include:

a. when the mission of the agency requires the building to be functional under post-earthquake emergency conditions,
b. when the contents of the building must be protected to prevent secondary life-safety risks from fire or hazardous material release,
c. when the protection of the building, its subsystems, or its contents is justified by economic considerations.

Each of these situations should be preliminarily identified, a higher performance objective established, appropriate evaluation techniques applied, and an appropriate mitigation program implemented.

5.2 Rehabilitation Standards Intended to Achieve Performance Beyond Substantial Life-Safety

Many Federal agencies and other organizations have identified use and occupancy categories that require special seismic protection and have adopted design standards to suit. These primarily apply in new buildings.

Table 2 lists several standards that have been used to achieve various performance objectives, along with the specific performance concern for which the standard was adopted. Example occupancies where use of the standard may be appropriate are also shown.

It must be noted that most of the standards listed in Table 2 have been developed for new buildings. Application of these standards to rehabilitation of existing buildings requires detailed consideration of the interaction between existing structural systems and new elements which may be added. Use of existing systems for lateral resistance may require use of lower demand-to-capacity ratios than allowed in FEMA 178 to assure satisfaction of performance objectives beyond life-safety.

Performance Objective	Previously Defined Standards	Specific Concern	Example Occupancies
Fully Functional	---	Building to be undamaged and to remain fully operational during earthquake	---
Immediate Occupancy	DOE-STD-1020-92 - Moderate & High DOD Tri-Services - Essential Bldgs.	Use of a building immediately following an earthquake and containment of hazardous materials	Facilities for storage of hazardous materials, weapons laboratories
	H-08-8 (VA) CBC - Hospitals 1991 UBC (I=1.5)	Use of a building immediately following an earthquake	Hospitals, fire & police stations, emergency and computer centers
Damage Control	DOE-STD-1020-92 - Low hazard & General Use	Protect occupants and prevent release of hazardous materials	Facilities for storage of hazardous materials, laboratories
	CBC - Schools	Protect occupants that are not fully able to help themselves	K-12 schools
	Developed for specific projects	Protection of valuable contents	Museums, high-technology laboratories, computer centers
	1991 UBC (I=1.0) 1992 Tri-Services Man. H-08-8 (VA) - MOBs GSA Seismic Design Manual	Resist a minor earthquake without damage Resist a moderate earthquake without structural damage but with some nonstructural damage Resist a major earthquake with damage but without collapse	New buildings and non-critical existing buildings for which damage control is desired because of cost-benefit, historical, aesthetic, or other considerations
	FEMA 222 - New Buildings	Minimize the hazard to life in all buildings	Same as above
Substantial Life-safety	FEMA 178 ATC-22/ATC-26-1 ATC-14	Protect occupants and general public	Offices, commercial buildings, apartment buildings, education (college), other non-critical occupancies

TABLE 2 - PERFORMANCE OBJECTIVE ADVISORY MATRIX

COMMENTARY

C1.0 INTRODUCTION

Several documents served as key references for development of these Standards. Interagency Committee on Seismic Safety in Construction (ICSSC) documents RP 2.1A (NIST 1992), dealing with new construction, and RP 3 (NIST 1989), an earlier guideline for existing buildings, provided a precedent and format for these Standards. As part of the development of these standards, the private consultants hired by NIST completed several studies and described their findings in reports to the ICSSC, the Task 1 Report and the Task 2 and 3 Report. These two reports provided valuable background information about ongoing agency and private sector seismic risk reduction programs, application to Federal buildings of previously resolved issues concerning seismic rehabilitation, a comparison of ongoing agency programs to FEMA 178, and a potential program that could be used to determine costs of seismic rehabilitation.

Substantial consideration was given to the current seismic hazard mitigation programs at the various agencies with a view to providing for their compatibility with these Standards. In addition, FEMA 237 and NIST GCR 92-617 provided constraints that were used in developing these Standards. FEMA 237 identifies issues associated with development of nationally applicable guidelines for seismic rehabilitation. NIST GCR 92-617 contains the proceedings of an ICSSC workshop held to resolve several of the FEMA 237 issues as they apply specifically to Federal buildings. These documents are listed in more detail in the Reference Document Summary.

C1.1 Objectives

The Standards establish Substantial Life-Safety as the minimum performance objective to be achieved in Federally owned or leased buildings that are subject to seismic evaluation and mitigation. At the ICSSC workshop described above, Substantial Life-Safety was identified as the appropriate minimum acceptable level of seismic safety for existing Federal buildings.

These Standards are not intended for use in judging the adequacy of past good-faith agency efforts at evaluation and mitigation; they are intended to establish appropriate minimums for actions taken after these Standards are formally adopted.

C1.1.1 Seismic Performance Objectives

Quite specific seismic performance objectives for certain buildings have been identified by several agencies (GSA, DOE) and others (State of California [see Table C1], Hewlett-Packard, Stanford University) and are discussed in detail in the Task 1 Report. Building codes for new construction also place requirements on certain occupancies that create de facto performance objectives that deal with issues over and above life-safety.

The seismic performance objectives described in Section 1.1.1 fall into three general categories: 1) those seeking to insure life-safety to occupants and the general public, 2) those seeking to control property damage, and 3) those seeking to maintain the ability of a building

TABLE C1

from California Seismic Safety Commission Report SSC 91-1

Earthquake Performance Objectives for Existing State Buildings

Earthquake Performance Objectives	Post-Earthquake Functions Within	Building Standards[1]	Occupancy Categories[2]							
			Hospitals, Essential Services	Hazardous Materials	Public Schools	Nursing, Prisons	University, Research	Offices, Courts	Other Occupancies	Historic, Non-essential
Fully Functional, no significant damage	Immediate	Nuclear Reg. Commission	✱	✱	✱	✱	✱	✱	✱	✱
Immediate Occupancy, minimal post-earthquake disruption, some non-structural cleanup required	Hours	Title 24 I = 1.50, 1.25	◆[3]	○[4]	○	○	○	○	○	○[5]
Repairable Damage, some structural and nonstructural damage, will not significantly jeopardize life	Days to Months	Title 24 I = 1, 1.15 Current UBC[6]	●	○[4]	○	○	○	○	○	○[5]
Substantial Life Safety, significant damage may not be repairable, will not significantly jeopardize life	Year(s)	75% of the 1988 UBC; ATC 14 & 22; or 1973 UBC[7]	●	◆[4]	◆	◆	◆	◆	◆	○[5]
Life Hazards Reduced, unrepairable damage very likely, some falling hazards, building may be a total loss, low life hazards.	No Limit	UCBC Appendix Ch. 1 for URM Bearing Wall Buildings	●	●	●	●	◆[8]	◆[8]	◆[8]	◆[5]
Very Poor Life Safety, collapse likely, unrepairable damage and total loss highly likely, significant life hazards	No Limit	None	●	●	●	●	●	●	●	●
Unsafe for Occupancy	No Limit	None	●	●	●	●	●	●	●	●
Unknown Performance	No Limit	None	●	●	●	●	●	●	●	●

Key:

◆ = Minimum Acceptable Earthquake Performance Objective
○ = Acceptable Earthquake Performance Objective
● = Unacceptable Earthquake Performance Objective
✱ = Typically does not apply, except to nuclear facilities

Abbreviations:

ATC—Applied Technology Council
I—Occupancy Importance Factor (pursuant to Ch. 23, Title 24)
Title 24 (Part 2, California Code of Regulations)—California Building Code
UBC—Uniform Building Code
UCBC—Uniform Code for Building Conservation
URM—Unreinforced Masonry

Footnotes:

1—Most building standards are not currently required by law for existing buildings, unless triggered by voluntary or mandatory strengthening, major alterations, additions, or changes of occupancy. This policy recommends that all existing state government buildings meet minimum earthquake performance objectives by the year 2000.
2—Emergency and recovery plans required for all occupancies.
3—Communications, emergency services, and acute care services shall be capable of functioning after earthquakes, as well as having immediate occupancy throughout the building.
4—Acceptable if chance of release of hazardous materials is remote.
5—Acceptable if anticipated earthquake damage is repairable, and the building also complies with the State Historical Building Code.
6—Applies to state leased buildings.
7—A uniform seismic retrofit building standard must be developed.
8—Acceptable for strengthened URM bearing wall buildings only.

to function immediately after an earthquake. Assuring totally unaffected functionality after a major earthquake is difficult and is not considered realistic within the realm of buildings intended to be covered by the Standards. However, to bracket the acceptable objectives on the high side, a performance objective of *Fully Functional* has been described. Similarly, to bracket the three most commonly used performance objectives on the low side, a range of nonspecific seismic improvements to buildings (e. g. parapet bracing) has been identified by the category of *Risk Reduction.*

The three performance objectives most applicable to Federal buildings, then, are *Substantial Life Safety, Damage Control,* and *Immediate Occupancy. Substantial Life Safety* refers to a level of damage that minimizes casualties from structural collapse, structural and nonstructural falling hazards, and panic from problematic evacuation from the building. An important concept associated with this objective is that economics of damage repair are not considered; the building need not be economically repairable to meet the objective. This performance objective was first formalized in ATC 14 and has been carried forward as the baseline acceptance level for FEMA 178.

Damage Control covers a broad range of performance objectives that reach in a continuous band from *Substantial Life Safety* to *Immediate Occupancy.* The purposes of limiting damage beyond what is minimally required for life safety are generally economic and related to the value of the building or its contents. An intermediate level sometimes referred to as *Repairable Damage* has been used in rehabilitation terminology, but that title is limiting and does not represent the full range of purposes for control of damage beyond that minimally required for life-safety.

Immediate Occupancy is a special case of damage control and often represents the high end of the range. This objective is set when the building is needed immediately after an earthquake so that whatever functions it supports can be maintained. The minimum requirement for such a building would be to pass the post earthquake emergency evaluation so that it would not be closed by the controlling building authority. Additional requirements related to the permissible level of damage to internal elements, equipment, and machinery would depend on the specifics of the function required in the post earthquake emergency period.

The most commonly referenced set of performance objectives previously published, and perhaps the first to be formalized, is contained in Table C1, "Earthquake Performance Objectives for Existing State Buildings" of the California Seismic Safety Commission report SSC 91-01.

Performance objectives, to be meaningful, must be related to a given event or level of shaking. The level assumed in the definitions in this document is the "code event" (level of shaking defined by the governing design document). In the design documents currently in common use in this country for new and existing buildings, that level is defined as shaking that has a 10% chance of being exceeded in 50 years. However, the relationship between shaking at this probability level and at lesser or greater probabilities varies greatly in different regions of the country. A more accurate definition of performance objectives intended to be applicable throughout the country would therefore require consideration of performance for several different earthquakes; such consideration is currently beyond the intended precision of these Standards.

Although the minimum long-term performance objective for all Federal buildings is Substantial Life-Safety, it is recognized that some agencies have ongoing programs which may prioritize the various actions needed to reduce overall seismic risk. Such programs could be considered in compliance with the intent of the Standards as long as all components of Substantial Life-Safety are considered when triggered by Section 2.1, and the agency documents its overall plan to reach the long-term goals of the Standards.

C1.1.2 Additional Objectives

Agencies may adopt higher performance levels; guidance on achieving higher performance is included in Chapter 5 of the Standards.

Identification and correction of seismic deficiencies that could impede an agency's ability to carry out its mission should be coordinated with the correction of life-safety hazards. Clearly, it would not be cost-beneficial to correct life-safety deficiencies in a building and discover that this performance level is inadequate for some other reason. However, post earthquake emergency expectations or performance requirements must be separated from normal operating needs to determine the appropriate level of seismic rehabilitation; a study of actual emergency operation requirements, the existence of potential back up facilities, and other considerations may indicate that extraordinary seismic performance is not essential.

The Standards make it clear that existing buildings need not be held to more stringent requirements than each agency applies to new buildings. This can occur, for example, when a site-specific spectrum representing free-field motion--which may be larger than equivalent mapped values, particularly when very near faults--is used for the evaluation of a building rather than the FEMA 178 base shear coefficients.

C1.2 Scope - Hazards

The four compliance categories identified — structural, nonstructural, geologic/site, and adjacency — are convenient groupings of sources of potential life-safety hazards. Element's of all four are included within the scope of FEMA 178, although in that document the potential deficiencies are not organized or identified specifically in these categories. In addition to obvious differences, each category has subtle characteristics that make separation convenient. Geologic/site hazards will likely only be determined by site specific studies. The adjacency category often will directly involve property not owned by the government and may therefore require legal or administrative intervention, rather than engineering solutions. It is also likely that future improvements in evaluation techniques will be easily categorized into one of these groups and can thus be modularly integrated into these Standards with a minimum of disruption. For example, the use of the geologic assessment developed in ATC-26 for the U.S. Postal Service, an expansion and improvement over FEMA 178, is easily incorporated into these Standards.

"False" answers to the FEMA 178 checklists indicate potential deficiencies; further analysis is needed to determine whether a substantial threat to life safety actually exists. For example, even though evaluation may show that liquefaction may occur on a site, the horizontal extent and continuity of the vulnerable layer, its thickness and depth, and the probable effect of the liquefaction on foundation settlements must be estimated before the nature of the threat to the structure can be determined. In accordance with FEMA 178, Section 1.3.3.3, structural "deficiencies" must be particularly reviewed for their actual anticipated contribution to life-safety risk.

Seismic contour maps 3 and 4 in the 1991 *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* are referenced in these Standards to represent varying levels of seismic hazard for all areas in the United States. These maps are reproduced in FEMA 178 at a smaller scale, without the contour lines depicting the acceleration coefficient of 0.15. The FEMA 178 maps may be used if the agency finds the level of detail is adequate for its needs. Site-specific studies may also be used to establish the seismicity of a site.

C1.2.1 Items Not Included in Standards

Although there are obvious interactions between seismic concerns and other natural or manmade threats to buildings, a multi-hazard approach is far beyond the scope of this document. However, before mitigation measures are taken for seismic deficiencies, it is strongly suggested that other potential hazards, particularly wind and blast, also be considered.

Further, it is beyond the scope of these Standards to address evaluation and mitigation criteria for damaged and deteriorated buildings, including those buildings damaged by earthquakes. However, any agency conducting an evaluation of a building damaged by any cause must investigate the condition of both the vertical and lateral force resisting elements to insure that these elements can perform dependably during an earthquake.

C1.3 Scope - Buildings

Buildings that may have higher performance objectives than Substantial Life-Safety should be identified prior to their elimination as exempted buildings, to assure that they are given adequate consideration. Performance expectations for even recently constructed buildings should be compared with required objectives; benchmark years suggested in Table 1 of the Standards (Section 1.3.1) may not be applicable to the higher performance objectives.

The list of buildings that need not meet these Standards--either because they do not present a measurable life safety risk, or because they do not fit within boundaries commonly placed on *building* standards and technology--was developed considering similar existing lists, and definitions already in use.

Item a. is based on one used in the Tri-Services Existing Building Manual as part of "Inventory Reduction". The exemption of small, nonessential buildings is enhanced by adding a definition of minimal occupancy as two hours per day (cumulative, for all occupants within 24 hours). The minimal occupancy definition is used in California under the Alquist-Priolo Act, which identifies zones that may be susceptible to surface fault rupture and therefore where special controls on construction are required.

Item b. is used in the NEHRP Recommended Provisions for new buildings (FEMA 222), and therefore those buildings should be equally excluded when considering existing buildings.

Item c. is simply a repeat of the exclusion of special structures used in FEMA 222 for new buildings. Excluding these structures from these Standards should not be interpreted as an indication that they do not present a seismic risk to life safety. Most or all of these structures should be considered under other seismic risk reduction programs and many should be considered essential. Lifeline buildings, or those buildings associated with, or containing lifeline systems, have been specifically included in the seismic standards for lifelines proposed for development by NIST. However, the schedule for completion and implementation of those standards is not yet clear, and until such a program is in place, such buildings should be considered as covered by these Standards.

Item d. is a combination of exclusions used by the U.S. Postal Service in ATC 26 and the Tri-Services Existing Building Manual. This exclusion results from the fact that steel light frame and wood buildings as defined in FEMA 178 have little if any history of creating life safety risks resulting from structural failure.

Item e. can refer to (1) buildings which have already been rehabilitated following the requirements of these standards, or (2) buildings which have been rehabilitated prior to the development and adoption of these standards. In deciding whether category (2) buildings are exempt, agencies should compare the evaluation and rehabilitation standards previsouly used with these Standards to show substantial compliance. Previous programs may have concentrated on structural deficiencies; in order for a previously rehabilitated building to be considered exempt, all four compliance categories must be considered satisfied.

Items f and g refer to benchmark years. Benchmark years are used to identify buildings which have been designed to standards which provide adequate structural seismic safety. Due to the constant refinements in building codes for new buildings, these years vary for different codes and different building types. Local modifications to code requirements and changes in zone maps should be considered when establishing benchmark years for agency use. (Also see Section C1.3.1.) A special case of post-benchmark buildings are the most recent ones, designed under E.O. 12699, which must attain seismic safety substantially equivalent to that in FEMA 222.

Item i refers to leased buildings. See Section C1.2.2 for further discussion of exemptions for leased spaces.

Item h. is included to make clear that the Standards do not apply to buildings which are situated on public land but which are not owned or leased by the Federal government, such as privately-owned ski area structures in National Forests.

C1.3.1 Post-benchmark Buildings

The establishment of benchmark years that will automatically qualify buildings as being structurally adequate is complex. Changes have been made to codes for new buildings in a piecemeal fashion which affect different building types in different ways. ATC 21 attempted to define appropriate benchmark years to facilitate the rapid screening/preliminary evaluation process. The table used for this purpose in ATC 21 has been updated for inclusion as an advisory table in these Standards. Many factors tend to indicate that the establishment of rigid benchmark years is inappropriate. These include the variation in treatment of certain building types such as unreinforced masonry in past building codes, the uneven adoption and enforcement of seismic provisions when presented in model codes as optional, and the existence of individual agency standards which were often more advanced than local codes. Advisory Table 1 should therefore be viewed as a set of conservative default benchmark years. The ultimate standard for acceptance is the least restrictive requirements of either FEMA 178 or the applicable seismic code for new buildings. Benchmark years for any previously used seismic provisions can be established by comparing resulting designs by building types with the acceptance standards. Care must be taken in such comparisons to consider all possible variations of the building type studied.

C1.3.2 Leased Buildings

A great deal of flexibility is necessary when dealing with leased spaces because of the wide variety of lease terms, the probable lack of control over the owner of the building, and the possible lack of alternative space.

These Standards identify certain leased spaces as exempt: the space must consist of less than 50 percent of the total space in the building and the space must be less than 10,000 square feet in area. The 50 percent rule stems from the Senate Committee Report on the 1987 NEHRP Reauthorization Act (Senate Report 101-446, August 30, 1990, p. 16), in which committee discussion indicates that it may be unrealistic to assume that the government could control the seismic safety of leased spaces comprising less than 50 percent of the total building area.

The second criterion related to 930 m^2 (10,000 square feet) was selected to match the size limit for accelerated and simplified lease procedure currently used by the GSA, the government's largest user of leased space. GSA routinely waives many of their leasing criteria for areas less than 10,000 square feet.

Both the 50 percent and the large area criteria must be met for exemption. This is to ensure that when significant portions or entire buildings are leased, even though they are not large, seismic safety is considered when leases are initiated or renewed.

C1.4 Summary of Standards

Self-explanatory.

C2.0 APPLICATION OF THE STANDARDS

C2.1 Situations Requiring Evaluation and Mitigation

"Active" components of a seismic hazard mitigation program specifically require some action to be taken, such as inventory, screening, evaluation or strengthening of buildings. "Passive" components refer to requirements which only become effective if "triggered" by (required because of) changes to the building's status. This section defines those situations which trigger a seismic evaluation and hazard mitigation of a Federal building.

The focus of passive triggers is on changes to the building which increase its life or value (significant remodelling) or will increase the risk level of the building (changes in occupancy). These types of requirements are included in many different codes and mitigation programs and have been discussed at length in a recent report (Hoover, 1992). A few Federal agencies have had several such triggers incorporated into their construction programs for some time. Although the philosophy of the use of passive triggers is to achieve safety similar to a new building when "renewing" an old building, such triggers also serve to gradually reduce the overall seismic risk presented by the existing building stock. In addition, since such triggered improvements will be done concurrently with significant non-seismic work, the cost and disruption attributable to the seismic rehabilitation is minimized.

In the private sector strict enforcement of such triggers has also served to effectively limit improvements to the existing building stock and at times has encouraged careful planning to avoid the triggers. Examples of this effect can be seen with building code provisions used in Massachusetts and for hospitals in California. It is important to note that the triggering value for renovations (50 percent of building value) has been used in past building codes.

The basic triggers listed in this section encourage consistent application of the "renewal" philosophy discussed above. Because of the efficiency of combining seismic rehabilitation with other work, additional triggers may be advantageous for each agency considering the characteristics of its own program. However, when such triggers occur in buildings in regions of low seismicity, structural adequacy of existing conditions should be carefully considered, possibly using post-yield evaluation techniques, before seismic rehabilitation is recommended, in order to achieve an economically reasonable approach.

A building presenting an "exceptionally high risk" may be discovered at any time, either in a systematic evaluation process, or by review of the building for other purposes. A plan to reduce such high risks should be developed immediately. One or more of the mitigation measures listed in Section 4.1 should be considered.

Item e. is intended to prevent additional unsafe buildings from being permanently added to the Federal inventory, by triggering a seismic evaluation and if necessary, mitigation, when they are acquired. It is not intended to apply to buildings temporarily under Federal ownership, such as those acquired through defaults or foreclosures on Federally insured loans or mortgages, or those in the assets of failed banks placed under Federal guardianship. Newly leased buildings are covered in Sec. 1.3.2.

C2.2 Compliance

Self-explanatory.

C2.3 Qualifications of Evaluators, Designers, and Reviewers

Qualified engineers should be used to evaluate seismic hazards for each of the four compliance categories for a specific building and to plan rehabiliation schemes necessary for mitigation. The qualifications of the individuals should match the scope and complexity of the assignment. Registration as a Professional Engineer should provide an individual with at least a familiarity with design and analysis of buildings for lateral loads. In addition, training and experience in seismic investigations should be required.

Those with a minimum amount of such background may be qualified for relatively small and simple buildings, particularly in areas of low seismicity. More highly qualified individuals may be required for complex buildings or for peer review. Such persons will likely have academic credentials beyond the bachelor level with courses in structural dynamics, inelastic analysis, and other topics in advanced earthquake engineering. They may have published technical articles on seismic issues of existing structures or be active in related professional organizations. Their project experience should relate specifically to seismic investigations of structures. They should be capable of providing personal references attesting to their successful completion of projects similar to that contemplated by the agency.

Where a detailed evaluation of geologic/site hazards is necessary, a specialist in geology should be used. The preliminary evaluation of geologic/site hazards can be performed by engineers without extensive geological experience.

C2.4 Additional Requirements

Items a. and b. are self-explanatory.

Item c., quality control, cannot be overlooked in a seismic hazard mitigation project. All phases of a project, including evaluation, design, and construction, must be monitored and evaluated to be successful. Guidance from documents like these Standards and FEMA 178 is needed in order to consistently identify and improve seismically hazardous buildings. However, earthquake engineering is not an exact science. Codes are constantly developing in an attempt to incorporate new research and to balance safety, building performance, and cost. Considerable engineering judgement is required to properly apply the provisions of these Standards to existing Federal buildings. Reviews of evaluations for consistency, construction documents for adequacy, and construction itself for compliance with drawings and construction standards are all essential to maximize effectiveness of the project.

Item d. is intended to serve as a generalized "grandfather" clause. Studies documented in the Task 1 Report indicate a wide variety of accomplishments and techniques in seismic hazard reduction by the various agencies. It is not the intent of these Standards to rewrite these procedures but to set common minimum standards for use by all agencies. Once these Standards are formally adopted for Federal use, each agency should be able to demonstrate that its existing programs meet or excede these minimum Standards. In fact, most of the evaluation and mitigation standards now in use have been shown to be equivalent or more stringent than FEMA 178 in the Task 2 & 3 Final Report.

C3.0 EVALUATION

The purpose of the evaluation is to identify conditions related to a specific building which pose seismic hazards. The Standard adopts FEMA 178 as the basic approach for the evaluation, although some modifications are specified. The engineer will generally conduct a preliminary evaluation in each of the compliance categories and a detailed evaluation only for those items identified in the preliminary study as potential life-safety risks.

C3.1 Preliminary Evaluation

The preliminary evaluations consist of responding to lists of questions contained in FEMA 178 related to a specific building and its physical characteristics. Negative responses indicate that a life-safety concern may exist and detailed evaluation of that particular concern is therefore required, in acccordance with Section 3.2 of the Standards.

C3.1.1 Structural

FEMA 178 includes generic structural checklists for use in the General Procedure (FEMA 178, Section 2.2.1). FEMA 178 also includes specific structural checklists for various structural systems for the Common Building Type Procedure (FEMA 178, Section 2.2.2). These checklists for specific building types are useful in order to quickly highlight potential problem areas that commonly occur; for example, it is pointed out in the checklist for infill buildings that the "nonstructural" masonry infill should be considered for its structural stiffness and lateral load carrying capability. The selection of the appropriate procedure is made by the engineer based on an initial review of the structure. Some statements in the procedure may be inappropriate for specific cases and can be dismissed by the engineer. The engineer is responsible for thoroughly reviewing the FEMA 178 document to become familiar with the entire procedure prior to beginning the evaluation.

C3.1.2 Nonstructural

Elements which are not part of the structural resisting system are evaluated in accordance with FEMA 178, Chapter 11. The procedure is similar to that for structural elements and utilizes a checklist approach. Nonstructural elements include a large number of items ranging from massive stone or concrete cladding panels to architectural furnishings, fixtures, and equipment. The life-safety implications of negative responses to the FEMA 178 nonstructural checklist must be carefully assessed by the engineer. Those which do not pose serious life-safety hazards should be eliminated from further consideration. The list given in the Standards are items considered life-safety hazards for the purposes of these Standards.

C3.1.3 Geologic/Site

FEMA 178 includes a section on the evaluation of foundations and geologic/site hazards. This consists of a series of true-false statements regarding the condition and capacity of foundations and geologic/site hazards. FEMA 178 is fairly general for both foundations and geologic/site hazards. For foundation-related problems, this generality is acceptable since the evaluation should be done by a Professional Engineer with adequate experience and well developed engineering judgment. With respect to geologic/site hazards, however, the

ATC-26-1 Rapid Evaluation Procedure for Geologic Hazards is a more complete method for evaluating potential geologic/site hazards and is suggested as a preliminary evaluation methodology. The Rapid Evaluation Procedure was developed for use by an engineer without extensive geological or geotechnical experience. The ATC-26-1 procedure also covers a broader range of potential hazards.

Typical geologic/site hazards include:

a) Surface fault rupture.
b) Soil liquefaction.
c) Differential compaction.
d) Landslides.
e) Tsunami-induced flooding.

The engineer must carefully consider the life-safety implications of any potential geologic hazard. For example, the warning time of tsunami-induced flooding may or may not be sufficient to allow evacuation of a building to avoid injury or death; the life safety implications of a landslide directly below a building and its foundation differ considerably from the impact of a landslide directly below a parking lot located on unstable fill.

C3.1.4 Adjacency

Adjacency hazards are those which arise from the proximity of two buildings. They include:

a) Pounding - The impacting of two adjacent buildings during an earthquake which causes damage that results in a life-safety concern.
b) Common Walls - Some buildings share common property-line walls that may pose a hazard during earthquakes.
c) Falling Hazards - Elements of buildings may fall on adjacent buildings during earthquakes.

FEMA 178 (Section 3.4) specifically covers potential hazards related to pounding. FEMA 178 assumes a hazard if the adjacent building is too close. Other potential adjacency hazards, however, are addressed only indirectly in FEMA 178. For example, common walls are only covered as structural elements of the building under investigation. At the preliminary evaluation stage, common walls should normally be considered a potential hazard in order to require analysis of seismic forces during the detailed evaluation stage.

Falling hazards posed by the building under evaluation are covered in the structural and/or nonstructural evaluation procedures. Risks to the Federal building posed by elements falling <u>from</u> adjacent buildings should be evaluated by the same procedure. This may be difficult to evaluate since access to the adjacent building for inspection may not always be possible.

The cooperation of the owner of the neighboring building will be required in order to fully complete an evaluation of adjacency hazards. Where such cooperation is not forthcoming, the evaluator should make reasonable, conservative assumptions about the building in order to complete the preliminary evaluation. If a potential life-safety hazard is identified, administrative or legal action may be needed in order to complete a detailed evaluation.

C3.2 Detailed Evaluation

The result of the preliminary evaluation in each of the compliance categories will be a list of potential seismic hazards. The purpose of the detailed evaluations is to investigate each potential condition to verify that a hazard actually exists. The procedure might include theoretical analyses, field or laboratory tests, and inspections. The engineer may conclude that some conditions do not constitute hazards and are therefore acceptable. Those conditions which are confirmed as hazards must be mitigated in accordance with the Standards.

C3.2.1 Structural

FEMA 178 specifies the procedures for detailed evaluation of potential structural hazards. Generally, an analysis comparing seismic demand to seismic capacity is made to determine the adequacy of individual structural elements. This approach is valuable, as it can identify the weak links in the lateral load resisting system. However, the engineer must interpret the results based on consideration of overall conditions. The procedure of FEMA 178 is an excellent guide, but it is not possible to anticipate the inevitable subtle differences posed by individual buildings in a pass/fail procedure. FEMA 178 (Section 1.3.3.3) recognizes this explicitly and requires the engineer to exercise experience and judgement before declaring a building a life-safety hazard.

A post elastic analysis can greatly enhance the understanding of the seismic performance of a building. The Standards include, as Appendix B, the U.S. Postal Service procedures for this purpose. This approach was originally developed for the Tri-Services Manuals, and has been reformatted for use in the proposed Postal Service program. It is reproduced here by permision of the U.S. Postal Service.

C3.2.2 Nonstructural

As noted above, the preliminary evaluation should identify for detailed analysis only those nonstructural items which could pose significant life-safety hazards. The detailed analysis of such elements in accordance with FEMA 178 parallels the requirements of the NEHRP Recommended Provisions for new buildings but allows the use of two-thirds of the demand force used for new construction. Elements which do not satisfy these criteria technically fail the detailed evaluation. It may be, however, that the consequences of the failure do not constitute a hazard. For example, the bracing for a piece of suspended electrical equipment may be judged to have insufficient strength using the FEMA 178 procedure, but the ductility of the system may be such that loss of vertical support is unlikely even if the bracing fails. In this case, the engineer may judge that no hazard exists.

C3.2.3 Geologic/Site

The detailed evaluation should generally cover the issues outlined in the geologic hazards evaluation procedure in ATC-26-1 (Appendix A of this document). Detailed evaluation requires specific information regarding site soils and geology. If this information is not readily available, subsurface investigations may be required.

Particular attention should be paid to estimating the probable consequences of liquefaction if such a potential should exist. For example, even though evaluation may show that liquefaction may occur on a site, the horizontal extent and continuity of the vulnerable layer, its thickness and depth, and the probable effect of the liquefaction on foundation settlements must be estimated before the nature of the threat to the structure can be determined. Step 3 of Section 6.3.3 of the Postal Service procedures outlines some of the effects that should be considered. The specific performance objective must be kept in mind when determining whether liquefaction potential adds significantly to the seismic risk in any one building.

C3.2.4 Adjacency

The detailed evaluation of potential adjacency hazards is essentially the same as for structural hazards. When assessing the hazard which may be associated with pounding, the engineer must first determine the magnitude of the potential pounding by calculating relative building displacements. The engineer must then consider the life-safety consequences of pounding before determining that a significant life-safety risk exists. For example, if the floor and roof diaphragms of the two buildings align vertically, it is unlikely that pounding would cause damage that would create a risk to life-safety unless one building was taller than the other.

C3.3 Alternative Analysis Techniques

The FEMA 178 procedures are well documented and fairly comprehensive. Nevertheless, other equivalent procedures exist. Agencies may have ongoing programs which achieve comparable results or could be easily modified to do so. In these cases, it is the intent of the Standards to allow alternatives that are shown to be substantially equivalent in results.

Specific situations in the context of a FEMA 178 evaluation may lend themselves to specialized analyses. This judgement should be initiated by the evaluating engineer. Independent peer review has been successfully utilized by many building owners to deal with the complex issues of seismic evaluation. The use of independent peer review is discussed further in Section C4.4.

C3.4 Development of Mitigation Concepts

FEMA 178 requires a written summary of the evaluation by the engineer. These Standards add the requirement that the engineer develop and provide cost estimates for remedial action. This is to provide the agency with information to aid in the consideration of mitigation alternatives. Some alternatives might not require rehabilitation of the building as discussed in C4.1.

C4.0 MITIGATION

C4.1 Requirements

The Standards require elimination of life-safety hazards, which may or may not include rehabilitation of the building itself. The intent is to eliminate life-safety hazards efficiently. In some cases the nature or extent of necessary rehabilitation can be so extensive that abandonment and relocation is a less costly alternative.

C4.2 Minimum Standards and Scope for Rehabilitation

The Standards generally require rehabilitation to the extent that the improved building would pass the minimum evaluation requirements. This characterization of the minimum standard for rehabilitation can be deceptive, however. FEMA 178 is not a design standard. Consequently, new work should comply with FEMA 222 or other agency standards for new construction except that seismic demand need not exceed levels of FEMA 178. In practice, this results in the use of the material sections of FEMA 222 for capacity design of new elements using FEMA 178 forces which are less than those for new buildings. All of the material detailing requirements of new construction apply to rehabilitation.

C4.2.1 Structural Hazards

In some instances, the direct correction of independent deficiencies may be sufficient. For example, the removal of dry rot or elimination of a structural pest infestation might result in satisfactory conformance in a wood frame building with no other deficiencies. The typical case, however, is more complex.

As discussed in Section 1.3.3 of FEMA 178, the performance of a structure in earthquakes depends upon the mutual interaction of the individual elements of the lateral load resisting system. Satisfactory performance normally results when the structure is capable of reaching a lateral load limit state without collapse. The relative strength and ductility of the individual elements of the lateral load resisting system define the limit state. Although FEMA 178 specifies procedures for assessing the adequacy of the individual elements and a general discussion of their interaction, the evaluating engineer is responsible for assessing the seriousness and relative importance of any deficiencies. In developing a rehabilitation scheme, the engineer must be mindful of the overall performance characteristics of the structure that result from changes to improve individual elements.

In some instances, the work required to completely satisfy the letter of the evaluation statements of FEMA 178 may be very costly. The engineer may propose alternative methods of evaluation or analysis and submit to peer review and approval. Additionally, new materials or structural systems not specifically addressed by current standards may be available. The Standards allow consideration of these as alternative methods.

C4.2.2 Nonstructural Hazards

During the evaluation, the Standards allow considerable discretion on the part of the engineer to interpret the life-safety implications of negative responses to evaluation statements for nonstructural elements. This discretion extends to the mitigation of any hazards. In many existing buildings, it is very costly to implement modern provisions for bracing nonstructural elements. In addition, some of the modern provisions exceed the life-safety goal of these Standards. The engineer may propose less expensive measures to accomplish an equivalent level of protection. These should be reviewed and approved by Peer Review under the provisions for alternative mitigation methods as described in Section C4.4.

C4.2.3 Geologic/Site Hazards

Specialized standards for the mitigation of seismic geologic/site hazards are not generally available. Considerable judgement and experience is required. It is very important that expert geologic and geotechnical advice be incorporated into any rehabilitation design for these hazards.

C4.2.4 Adjacency Hazards

There may be significant non-technical issues related to adjacency hazards such as legal and administrative policy considerations. If a technical solution to mitigate an adjacency hazard within the specific Federal building itself without modifying the adjacent building is considered unreasonable, the agency might try to convince or force the adjacent property owner to take action.

The nature of rehabilitation work to correct adjacency hazards will be highly site specific. In some cases it will be possible to correct hazards by work in the Federal building alone. For example, the agency might brace a parapet which poses a hazard to a neighboring building. The potential risk of a portion of a neighboring building falling into a Federal building might be mitigated by building a barrier on the Federal side. The danger of pounding of a Federal building might be reduced by strengthening the building to sustain the anticipated loads. Some adjacency hazards are best addressed by the joint efforts of the agency and the neighboring property owner. A common wall, for example, can be most effectively handled cooperatively.

Standards for the mitigation of adjacency hazards are equivalent to those for other structural hazards. FEMA 178, in its treatment of potential pounding, identifies a minimum separation between buildings. If a structure fails this requirement, rehabilitation measures to satisfy the minimum are implied. Any conditions relating to common walls and falling hazards would have to be treated in accordance with the procedures of FEMA 178. Conditions that fail the evaluation would have to be mitigated so that they pass. This means that a common wall, for example would have to exhibit an allowable demand/capacity ratio (or be strengthened so that it does) in the same manner as any other element not necessarily in common with an adjacent structure.

The requirement to notify the neighboring property owner recognizes that changes might reduce hazards in the Federal building but actually increase hazards for the neighboring building. For example, if a common wall is substantially strengthened and stiffened from the Federal building side, the unstrengthened portions of the neighboring building could be subject to increased torsion during a future earthquake. If adjacency hazards are mitigated jointly or solely by the neighboring property owner, the agency should attempt to ensure that the work meets the minimum requirements of the Standards.

C4.3 Incremental/Partial Rehabilitation

It may be efficient in some cases to complete some portion of a rehabilitation before all of the work is done. This is acceptable as long as the safety of the structure is not diminished at any time, except during actual rehabilitation construction. This requirement requires careful consideration of the performance of the structure after each increment of rehabilitation. The engineer must determine that partial work does not result in an undesirable lateral load limit state as discussed in Section C4.2.1 above.

C4.4 Innovative Mitigation Methods

Alternative methods of analysis, new materials and structural systems, or other non-complying techniques are generally allowed by building codes subject to some form of review and approval. Generally, the alternative methods must conform to the intent of the prevailing standard. This allowance is particularly important for the seismic rehabilitation of existing buildings due to large numbers of special conditions that inevitably arise. Many private and public institutions have established procedures for Peer Review. Some have standing panels; others hire reviewers specifically for projects when the need arises. Agencies should establish policies to ensure the independence and qualifications of the reviewers. The policy should also cover the general procedures to be followed by the engineer and the reviewers.

C4.5 Historic Buildings

The rehabilitation of historic buildings is a sensitive process. The design professionals must take care to protect the historical features and fabric of the building as much as possible. This reduces the flexibility and freedom to make alterations to the structure. Modern building standards, including FEMA 178 and FEMA 222, do not specifically cover the use of all archaic materials and systems. Often, engineers establish characteristics for these materials and systems by tests and available empirical data. The intention of these Standards is to provide essentially the same level of life-safety for historic buildings as for others without unreasonable impediment to the historic preservation process. Consequently, alternative mitigation methods (see Section C4.4) are allowed and encouraged when they can lessen the impact of the structural strengthening.

C5.0 PERFORMANCE OBJECTIVES BEYOND LIFE-SAFETY

C5.1 Identification of Conditions

The minimum acceptable seismic performance for Federal buildings is Substantial Life-Safety. Buildings meeting this criteria, however, may suffer significant damage and some may not even be salvageable. Better performance may be necessary for certain buildings in order for an agency to meet its post emergency mission, or may be deemed appropriate for other reasons, such as a high ratio of benefits to costs.

Determination of appropriate situations for evaluation and rehabilitation to performance objectives beyond life-safety is left to the individual agencies. Section 5.0 is therefore considered advisory.

The highest performance objective, "immediate occupancy", will require detailed consideration of critical lifelines, beyond the performance of the building itself, to assure achievement of this objective. The definition of this objective should include acceptance of realistic inconveniences in the post earthquake condition as long as the building can be used. Setting a performance objective of literally no damage and no effect on performance should be considered an extraordinary situation requiring special consideration of its feasibility in each case.

Buildings with mixed occupancies will require special consideration. A use of a small space for a critical function may not require the entire building to be strengthened to a high performance objective.

C5.2 Rehabilitation Standards Intended to Achieve Performance Beyond Substantial Life-Safety

The advisory matrix (Table 2 in the Standards) is built from information documented in the Task 1 Report. All matrix items are based on previously defined standards - none are new.

Table 2 contains information similar to the "Performance Objective Matrix" originally produced by the California Seismic Safety Commission (see Table C1 in this Commentary). However, considering the subtleties connected with performance objectives meant to apply to the wide variety of Federal buildings, the number of design standards already in use, and the advisory nature of the table, the format used in Table 2 of the Standards is slightly different. Table 2 couples standards with their probable performance level. In addition, appropriate standards for evaluation or rehabilitation can be determined either for the specific performance required or for the performance that may be appropriate for a given occupancy.

GLOSSARY

acceleration coefficient: A measure of seismicity expressed as an expected lateral ground acceleration as a percentage of gravity; examples include A_a and A_v in the *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*, and Z in the *Uniform Building Code.*

building: Any structure, fully or partially enclosed, used or intended for sheltering persons or property.

compliance categories: The four significant seismic risk categories which are addressed in these Standards: structural, non-structural, geologic/site and adjacency hazards.

These four categories are:

structural hazards: Structural elements of a building's vertical and lateral load carrying systems that could be damaged or fail during an earthquake.

non-structural hazards: Parts of a structure, other than the gravity or lateral-load carrying systems, that could fall and injure or kill a person during an earthquake. Includes parapets, exterior cladding or cornices, major mechanical equipment, tall hollow clay tile partitions, etc.

geologic/site hazards: Earthquake related hazards at a building site which occur during or after the earthquake. Includes surface faulting, landslides, liquefaction, and flooding due to tsunamis.

adjacency hazards: Hazards caused when adjacent buildings interact during an earthquake. Includes pounding, effects of one building buttressing another, falling hazards from an adjacent building, and the consequences of damage to common structural elements such as party walls (single bearing walls supporting two adjacent buildings constructed on separately defined parcels of land).

hazards: A hazard is a source of danger with potential to cause illness, injury or death to persons, or damage to a facility or to the environment (without regards for the likelihood or credibility of accident scenarios or consequence mitigation).

mitigation: Mitigation is the substantial reduction of life-safety risk from seismic hazards involving a building and/or building site. Examples include demolition, permanent evacuation, change in occupancy and rehabilitation.

performance objectives: A performance objective is a level of seismic functionality that a building owner or occupant expects of a structure. The minimum acceptable performance objective for application of these Standards is Substantial Life-Safety. Typical performance objectives include risk-reduction, substantial life-safety, damage control, immediate occupancy and full functionality.

These performance objectives are defined as:

risk-reduction: Performance objective where the earthquake damage state is greater than acceptable for life-safety but less than would have occurred in the building if no rehabilitative action had been taken. The extent of damage depends on the extent of the improvements made. As used in these Standards, the term "risk reduction" includes incremental strengthening as an interim measure in a total process aimed at achieving Substantial Life-Safety.

substantial life-safety: Performance objective where the 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. This is the minimum performance objective of these Standards. Compliance with FEMA 178 is assumed to achieve this level of performance.

damage control: Performance objective where the earthquake damage is controlled in order to protect some other feature of the building or its function beyond life-safety, for example, to control economic loss to the building itself, to prevent the release of toxic materials, or to protect building contents. The term "damage control" covers a range of performance objectives, from protection somewhat greater than that required for Substantial Life-Safety to somewhat less than needed for immediate occupancy.

immediate occupancy: Performance objective where the earthquake causes minor damage, facility disruption is minimal, and only some non-structural repairs and cleanup will be required. The facility is expected to remain occupied and be functional immediately after the earthquake event.

fully functional: Performance objective where the earthquake causes no damage to facilities and has no effect on building function. Achievement of such performance is beyond the scope and intent of these Standards.

post-benchmark building: A "post-benchmark" building is one which was designed and built after the adoption of seismic code provisions which are generally considered to provide acceptable life-safety protection. Specific benchmark dates vary with location, structural system, and governing building code. (See Table 1 for advisory benchmark years.)

rehabilitation: The repair, removal, replacement, strengthening, or protection of all structural elements which are identified as deficient during a building evaluation.

risk: The quantitative or qualitative expression of possible loss that considers both the probability that a hazard will cause harm and the consequences of that event.

REFERENCE DOCUMENT SUMMARY

This summary lists documents alphabetically by their colloquial name. The full name is given, along with a brief description of the content. For full citations, see the list of references following this summary.

ATC-14: Methods for Evaluating the Seismic Resistance of Existing Buildings:

ATC-14 was a first generation evaluation document which developed a procedure for the seismic evaluation of existing buildings based directly on the performance of buildings in past earthquakes. The procedure is limited to evaluating buildings for life-safety concerns.

ATC-22: A Handbook for Seismic Evaluation of Existing Buildings (Preliminary):

ATC-22 was a second generation evaluation document which built upon ATC-14 by refining the procedures, expanding the commentary information, and incorporating the strength design concepts of the NEHRP provisions for new buildings. The document format was modified into a handbook for easier use by evaluating engineers.

ATC-26-1: U.S. Postal Service Procedures for Seismic Evaluation of Existing Buildings (Interim):

A complete procedure for evaluating existing Postal Service facilities based on ATC-22 and other available methods including the post-yield techniques used in the Tri-Services manual.

ATC-26-4: U.S. Postal Service Procedures for Seismic Retrofit of Existing Buildings (Interim):

Presents guidelines for the seismic retrofit of existing buildings (15 building types) and nonstructural elements tailored to the Postal Service needs.

ATC-28: See FEMA 237.

BOCA: The BOCA National Building Code

The BOCA National Building Code is the standard of practice adopted for use in the majority of states in the Northeast United States and published by the Building Officials and Code Administrators International. The latest edition was published in 1993 and includes intermediate supplements.

CBC: California Building Code:

The California Building Code includes the seismic design requirements for California hospitals. It is composed of the 1991 edition of the UBC with special amendments. It is the intent of this code that hospitals designed to meet these provisions will remain functional in so far as practicable after the specified design level earthquake.

Denver Proceedings: Proceedings: ICSSC Issues Workshop, Development of Seismic Evaluation and Rehabilitation Standards for Federally Owned and Leased Buildings:

The ICSSC conducted an Issues Workshop to recommend policies related to these Standards. All recommendations were openly discussed and adopted by majority vote. The Proceedings include the issue statements as proposed, the subsequent discussion and the results of the balloting used to achieve consensus.

DOE Standard 1020-92: Natural Phenomena Hazard Design and Evaluation Criteria for Department of Energy Facilities:

A standard, based on UCRL 15910, which provides performance-based criteria for design of new structures, systems, and components (SSCs) and for evaluation, modification, or upgrade of existing SSCs so that DOE facilities meet selected performance goals when subjected to earthquakes, extreme winds, and flooding. The performance goals are based on a graded approach such that design, evaluation, and construction requirements of varying conservatism and rigor are established which reflect relative risk, environmental impact, importance, and cost of rehabilitating the SSCs.

FEMA Guidelines:

Guidelines for the seismic rehabilitation of buildings that are currently under preparation for FEMA by a joint effort of BSSC, ATC and ASCE. These guidelines are intended to be applicable nationwide and are expected to be done in 1997. ATC-33 is the project number of the portion of the project being done by ATC, which is the development of the Guidelines themselves.

FEMA 154: Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook:

A handbook for a rapid screening procedure based on a "sidewalk survey" style visual observation of buildings.

FEMA 172: NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings:

This companion document to FEMA 178 is a collection of techniques that can be used to correct seismic weaknesses, this document is the final consensus version of the seismic strengthening techniques originally developed for FEMA by URS/Blume in 1989.

FEMA 178: NEHRP Handbook for the Seismic Evaluation of Existing Buildings:

A third generation evaluation document built on the concepts and techniques established in ATC-14 and ATC-22. It was prepared by BSSC as a consensus document intended for the seismic evaluation of existing buildings in all parts of the United States.

FEMA 222/NEHRP Provisions: NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings: Part 1 - Provisions:

Seismic design technical criteria for new buildings developed by BSSC for FEMA. Current version is dated 1991. The previous version was named FEMA 95.

FEMA 237: Seismic Rehabilitation of Buildings - Phase 1 Issues Identification and Resolution:

Also known by its ATC project number ATC-28, FEMA 237 identifies and discusses issues that should be considered, resolved and included in the FEMA guidelines for the seismic strengthening of existing buildings.

H-08-8: Earthquake Resistant Design Requirements for VA Hospital Facilities:

Seismic design guidelines for new construction and rehabilitation projects prepared for the Department of Veterans Affairs. These guidelines were first adopted in 1973, have been updated on a regular basis and are currently under substantial revision to make them consistent with model building codes.

RP 2.1-A: Guidelines and Procedures for Implementation of the Executive Order on Seismic Safety of New Building Construction:

ICSSC RP 2.1-A provides Federal agencies with guidance in carrying out the provisions of Executive Order 12699, "Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction".

RP 3: Guidelines for Identification and Mitigation of Seismically Hazardous Existing Federal Buildings:

ICSSC RP 3 provides Federal agencies with guidelines for the mitigation of seismic hazards in their existing building inventories. This was a voluntary program published in 1989.

SBC: Standard Building Code:

The Standard Building Code is the standard of practice adopted for use in the majority of states in the southern United States. The current edition of the SBC was published in 1991 by the Southern Building Code Congress and includes intermediate supplements.

Task 1 Report: Evaluation and Strengthening Guidelines for Federal Buildings, TASK 1: Identification of Current Federal Agency Programs and Issues Identification for the Planned Guidelines:

A fact-finding report prepared during the development of these Standards by the contractors. Includes summaries of 7 Federal agency programs, a report on performance objectives currently in use by Federal agencies, and a discussion of the applicability of ATC-28 to the Federal effort.

Task 2 & 3 Report: Evaluation and Strengthening Guidelines for Federal Buildings, TASK 2: Assessment of Current Federal Agency Evaluation Programs and Rehabilitation Criteria and TASK 3: Development of Typical Costs for Seismic Rehabilitation:

A second background report prepared during the development of these Standards by the contractors. Task 2 involved the comparison of the main Federal agency evaluation and strengthening methodologies to those included in the NEHRP Handbooks (FEMA 178 & 172). Task 3 developed a plan for an optimum program to determine typical costs for seismic rehabilitation.

Tri-Services Manual:

Seismic Design Manual: TM5-809-10, NAVFAC P-355, AFM 88-3, Chapter 13: Seismic Design for Buildings:

A seismic design manual prepared by the Army, using a static load approach. The latest published edition was published in 1992.

Dynamic Analysis Manual: TM5-809-10-1, NAVFAC P-355-1, AFM 88-3, Chapter 13, Section A: Seismic Design Guidelines for Essential Buildings:

A seismic design manual for new, essential buildings, prepared by the Army using the dynamic load approach. Latest edition 1986.

Existing Buildings Manual: TM5-809-10-2, NAVFAC P355-2, AFM 88-3, Chapter 13, Section B: Seismic Design Guidelines for Upgrading Existing Buildings:

A manual prepared by the Army outlining a method for screening and evaluating existing buildings to determine their vulnerability to seismic events. Latest edition 1988.

UBC: Uniform Building Code:

The UBC is the standard of practice adopted for use in all states comprising the Western United States. The seismic provisions within the UBC were adapted from the Structural Engineers Association of California (SEAOC) "Blue Book". The current edition of the UBC was published in 1991 by the International Conference of Building Officials and includes intermediate supplements.

UCBC: <u>Uniform Code for Building Conservation</u>:

The UCBC is an optional companion document to the UBC and establishes overall life-safety requirements for existing buildings that undergo alteration or change in use. Within the context of seismic rehabilitation, it is predominantly used for unreinforced masonry structures and contains specific requirements for these structures in Appendix Chapter 1.

REFERENCES

Applied Technology Council, 1986. Methods for Evaluating the Seismic Resistance of Existing Buildings. Applied Technology Council Report ATC-14, Redwood City, California.

-------, 1989. A Handbook for Seismic Evaluation of Existing Buildings (Preliminary). Applied Technology Council Report ATC-22, Redwood City, California.

-------, 1989. Seismic Evaluation of Existing Buildings Supporting Documentation. Applied Technology Council Report ATC-22-1, Redwood City, California.

-------, 1991. U.S. Postal Service Procedures for Seismic Evaluation of Existing Buildings (Interim). Applied Technology Council Report ATC-26-1, Redwood City, California.

-------, 1991. U.S. Postal Service Procedures for Seismic Retrofit of Existing Buildings (Interim). Applied Technology Council Report ATC-26-4, Redwood City, California.

BOCA, 1993. The BOCA National Building Code. 1993 Edition, Country Club Hills, Illinois.

California Seismic Safety Commission, 1991. Policy on Acceptable Levels of Earthquake Risk in State Buildings. California Seismic Safety Commission Report SSC 91-01, Sacramento, California.

Degenkolb Associates, Engineers and Rutherford & Chekene, Consulting Engineers, 1992. Evaluation and Strengthening Guidelines for Federal Buildings: Task 1 Final Report. San Francisco, California.

-------, 1992. Evaluation and Strengthening Guidelines for Federal Buildings: Task 2 & 3 Final Report. San Francisco, California.

Departments of the Army, the Navy, and the Air Force, 1982. Seismic Design for Buildings. TM 5-809-10, NAVFAC P-355, AFM 88-3, Chap 13, Washington D.C.

-------, 1986. Seismic Design Guidelines for Essential Buildings. TM 5-809-10-1, NAVFAC P-355.1, AFM 88-3, Chap 13, Sec A, Washington D.C.

-------, 1988. Seismic Design Guidelines for Upgrading Existing Buildings. TM 5-809-10-2, NAVFAC P-355.2, AFM 88-3, Chap 13, Sec B, Washington D.C.

Federal Emergency Management Agency, 1985. NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings: Part 3 - Appendix. Developed by Building Seismic Safety Council for FEMA, FEMA 97, Washington, D.C.

-------, 1988. Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook. Applied Technology Council Report ATC-21, FEMA 154, Redwood City, California.

------, 1991. NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings. Developed by BSSC for FEMA, FEMA 172, Washington, D.C.

-------, 1991. NEHRP Handbook for the Seismic Evaluation of Existing Buildings. Developed by BSSC for FEMA, FEMA 178, Washington, D.C.

-------, 1991, NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings: Part 1 - Provisions. Developed by BSSC for FEMA, FEMA 222, Washington, D.C.

------, 1991. NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings: Part 2 - Commentary. Developed by BSSC for FEMA, FEMA 223, Washington, D.C.

-------, 1992. Development of Recommended Guidelines for Seismic Strengthening of Existing Buildings Phase 1: Issues, Identification and Resolution. Applied Technology Council Report ATC-28, FEMA 237, Redwood City, California.

General Accounting Office, 1992. FEDERAL BUILDINGS: Many Are Threatened by Earthquakes, but Limited Action Has Been Taken. GAO/GGD-92-62, Washington D.C.

General Services Administration, Public Buildings Service, 1990. Seismic Design Guidelines of the Facilities Standards for the Public Building Service, PBS-PQ100, Chapter 12. Washington D.C.

Hoover, Cynthia, 1992. Seismic Retrofit Policies: An Evaluation of Local Practices in Zone 4 and Their Application to Zone 3. Oakland, California.

ICBO, 1991. Uniform Building Code. 1991 Edition, Whittier, California.

ICBO, 1991. Uniform Code for Building Conservation. 1991 Edition, Whittier, California.

Southern Building Code Congress International Inc., 1991. Standard Building Code. 1991 Edition, Birmingham, Alabama.

State of California, 1989. California Building Code. California Administrative Code consisting of UBC '91 with 1991 California amendments, Whittier, California.

United States Department of Commerce, National Institute of Standards and Technology, 1989. Guidelines for Identification and Mitigation of Seismically Hazardous Existing Federal Buildings. Report developed by the Interagency Committee on Seismic Safety in Construction (ICSSC) Subcommittee 1, sponsored by FEMA, ICSSC RP-3, NISTIR 89-4062, Gaithersburg, Maryland.

-------, 1992. Guidelines and Procedures for Implementation of Executive Order on Seismic Safety of New Building Construction. Report developed by the Interagency Committee on Seismic Safety in Construction (ICSSC) Subcommittee 4, sponsored by FEMA, ICSSC RP-2.1A, NISTIR 4852, Gaithersburg, Maryland.

-------, 1992. Proceedings: ICSSC Issues Workshop, Development of Seismic Evaluation and Rehabilitation Standards for Federally Owned and Leased Buildings. Report developed by the Interagency Committee on Seismic Safety in Construction (ICSSC) Subcommittee 1, NISTIR GCR 92-617, Gaithersburg, Maryland.

United States Department of Energy, 1992. Natural Phenomena Hazards Design and Evaluation Criteria for DOE Facilities. Department of Energy DOE-STD-1020-92, Washington D.C.

United States Department of the Interior, 1983. Secretary of the Interior's Standards and Guidelines for Archaeology and Historic Preservation. Federal Register Volume 48, Number 190, September 29, 1983, pp. 44716-44740. National Technical Information Service order number PB85-238806, Springfield, VA.

United States Department of Veterans Affairs, 1986, Seismic Design Guidelines. Veterans Administration Seismic Design Guidelines H-08-8, Washington D.C.

-------, 1986, Seismic Design Guidelines - Site Evaluation Survey for Existing Facilities. Veterans Administration Seismic Design Guidelines H-08-8, Appendix A, Washington D.C.

-------, 1986, Seismic Design Guidelines - Scope of Work for Phase I and Phase II Evaluations of Existing Facilities. Veterans Administration Seismic Design Guidelines H-08-8, Appendix B, Washington D.C.

APPENDIX A

Assessment of Earthquake-Related Geologic Phenomena
(Chapter 6, ATC-26-1, United State Postal Service)

The information presented in this appendix was prepared by the Applied Technology Council (ATC) for specific use by the U.S. Postal Service, and issued in report number ATC-26-1. It is reprinted here with the permission of the U.S. Postal Service. While the information is believed to be correct, ATC, the U.S. Postal Service, the Interagency Committee on Seismic Safety in Construction, and the National Institute of Standards and Technology assume no responsiblity for its use by others. Users of information from this publication assume all liability arising from such use.

6. Assessment of Earthquake-Related Geologic Phenomena

6.1 Introduction

On an overall basis, the most significant earthquake-related consideration for buildings is ground shaking. The methodology for evaluating the vulnerability of Postal Service buildings and equipment to ground shaking is presented in Chapters 3 and 5. At some sites, however, other earthquake-related geological phenomena may be significant, including surface fault rupture, soil liquefaction and its associated consequences, differential compaction, landsliding, and flooding. Where these phenomena have occurred during past earthquakes, they have caused catastrophic damage to structures in many cases. Thus, these geological phenomena need to be considered in evaluating the expected performance of Postal Service buildings in future earthquakes.

To complete this evaluation, additional knowledge of site conditions must be obtained, either from existing sources or through an exploration program. Obtaining existing information about sites from various sources will often allow a qualified geotechnical engineer, geologist, or seismologist to make a determination of the site conditions without performing an intensive investigation. In some cases, however, it may be necessary to obtain knowledge of the site conditions through a specific subsurface exploration and testing program. Thus, the design engineer must use judgment when requesting a detailed evaluation of a particular site. By working with the seismologist, geologist, or geotechnical engineer, it is possible to determine the most expedient method of obtaining site information.

This chapter presents the methodology that can be used for rapid and detailed evaluations of earthquake-related geological phenomena. This methodology, presented in Sections 6.3 and 6.4, is prefaced by a brief description of the phenomena.

6.2 Description of Earthquake-Related Geologic Phenomena

6.2.1 Surface Fault Rupture

Fault rupture during moderate-to-large-magnitude earthquakes (Richter magnitude equal to or greater than 6) may involve breakage of the ground surface along the traces of surface exposure of the fault. The movements may range from a fraction of an inch to several feet or more, depending on earthquake magnitude and other factors. Surface fault rupture can be catastrophic to structures situated directly astride the rupture zone.

For purposes of evaluating the risk to existing buildings from surface fault rupture, a fault is considered to be active and capable of surface rupture if the fault exhibits any of the following characteristics:

- Has had documented historical earthquakes or is associated with a well-defined pattern of microseismicity.
- Is associated with well-defined geomorphic features suggestive of recent faulting.
- Has experienced surface rupture (including fault creep) during approximately the past 10,000 years (Holocene time).

6.2.2 Soil Liquefaction

Liquefaction is a soil behavior phenomenon in which a soil loses a substantial amount of strength due to high excess pore-water pressure generated by strong earthquake ground shaking. Recently deposited (geologically) and relatively unconsolidated soils and artificial fills located below the ground water table, are susceptible to liquefaction. Sands and silty sands are particularly susceptible soils. Silts and gravels are also susceptible to liquefaction, and some sensitive clays have also exhibited liquefaction-type strength losses. Potential consequences of liquefaction include loss of foundation bearing strength, flotation of lightweight embedded structures, differential settlement, landslides including lateral spreads and flow slides, and increased lateral pressures against retaining walls.

6.2.3 Differential Compaction

Dissipation of excess pore-water pressures generated in soils by strong ground shaking causes volume decreases within the soil (termed consolidation or compaction) that are manifested at the ground surface as settlement. Differential settlements may occur due to spatial variations in soil characteristics and variable foundation loads.

Compaction may occur in both liquefied and non-liquefied zones below the ground water table, with larger contributions to settlement expected to result from liquefied soil. Compaction may also occur in loose unsaturated soils above the ground water table. For most circumstances, ground shaking-induced settlement is not life-safety-endangering except in its most severe forms when associated with liquefaction.

6.2.4 *Landsliding*

In addition to landsliding that may be facilitated by the loss of soil strength accompanying liquefaction, slope movements may occur in the absence of liquefaction. Such movements are associated with ground shaking-induced inertia forces that cause a temporary exceedance of the strength of the geologic materials in a slope. The concern here is that a building may be located within a zone of seismically induced slope failure or located below a slope whose failure may send soil and debris into the structure. In addition to the intensity of ground shaking, factors which affect slope stability include slope angle, slope height, soil type, joints and bedding, ground water conditions, and past instability.

6.2.5 *Flooding*

Earthquake-induced flooding of a site can be caused by seiches, tsunamis, and dam, levee, and water storage tank failures. Seiches are undulations of the surface of a body of water such as a bay, lake, or reservoir, set up by harmonic interaction of the water body with seismic waves transmitted through the earth's crust. Seiches can be caused by earthquake occurrences either in the region of a site or thousands of miles away. Seiche waves may reach several feet in height and can be damaging to facilities located at or very near the shoreline. The occurrence of seiches large enough to cause flooding and damage is not common. Tsunamis, on the other hand, occur with greater frequency and can have greater damage potential than seiches. Tsunamis are ocean waves generated by vertical seafloor displacements associated with large earthquakes. As with seiches, tsunami waves at a site may be produced by local or distant events; but for tsunamis, wave heights may reach tens of feet at some coastal locations. Failures of dams, levees, and water storage tanks also pose a flooding danger to downstream construction.

6.3 Methodology for Rapid Evaluation of Earthquake-Related Geologic Phenomena

6.3.1 *General*

These rapid evaluation procedures are intended for all USPS buildings except: those buildings in NEHRP Seismic Map Areas 1 and 2, as well as any buildings judged by the USPS to be of sufficiently low importance that such evaluations are not needed.

The minimum qualification for rapid evaluation of the geological conditions is registration as a civil engineer. The methodology developed will allow registered civil engineers with minimal geotechnical experience to conduct the evaluation. In general, it is expected that this will result in a more conservative assessment. However, it is possible for the rapid evaluations to be carried out by experienced geotechnical engineers and geologists. This will facilitate the assessments and result in conclusions with less conservatism at a greater level of confidence. In this case, the methodology presented can easily be used as guidelines for the geotechnical specialist. The decision as to who should conduct the rapid evaluation may be made on a case-by-case basis, and may depend on the evaluation of the structure.

A rapid evaluation should include a check of whether any of the above described geologic phenomena has occurred historically at a site or in the near vicinity. (With respect to landsliding, this should include a check as to whether the site is located within a pre-existing active or ancient landslide, whether induced by earthquakes or other causes.) Such information, if available, could generally be obtained through a state, county, or city geologist. If it is known that a geologic phenomenon has occurred historically, then a detailed evaluation (Section 6.4) is required. If there is no evidence or knowledge of historic occurrences, then the rapid evaluation methodology presented below should be followed.

In some regions and communities of the United States, earthquake-related geologic phenomena have been mapped and rated on a regional basis by federal or state agencies, such as the urban-area mapping conducted under the U.S. Geological Survey's NEHRP. Such mapping (which can generally be obtained through a state, county, or city geologist) should be consulted where available, and, if an area is mapped as having a low risk with respect to any specific geologic phenomenon (e.g., liquefaction), then the risk for that area can be regarded as insignificant for purposes of a rapid evaluation. In the absence of such mapping, or to supplement the mapping, the step-by-step methodology outlined in the following sections can be used.

6.3.2 Surface Fault Rupture

Step 1. Select the appropriate NEHRP Seismic Map Area designation from the NEHRP Seismic A_a coefficient map. If the NEHRP Seismic Map Area is 4 or lower, do not continue the evaluation for surface fault rupture; the likelihood of active faults rupturing to the ground surface during earthquakes is considered to be low for these NEHRP Seismic Map Areas.

Step 2. For NEHRP Seismic Map Areas 5, 6, or 7, the evaluator should contact the state, county, or city geologist to identify the locations of any known active faults with earthquake-related surface rupture potential in the vicinity of the facility. (Note: It is anticipated that government geologists in the states affected can provide the input needed for a reasonable evaluation.) In California, the effort is simplified through the availability of the "Alquist-Priolo" maps, which define those zones within the state in which surface fault rupture is a significant risk. In general, since surface fault ruptures are localized along the traces of active faults, few USPS buildings are expected to be exposed to them. See Section 6.2.1.

6.3.3 Liquefaction

A rapid evaluation of a facility site for liquefaction considers the opportunity for liquefaction to occur (i.e., ground shaking potential), the susceptibility of site soils to liquefaction, and the possible consequences of liquefaction occurrence. As outlined below, either (1) a combination of opportunity and susceptibility that results in a low likelihood of liquefaction-induced ground failure, or (2) anticipated non-life-endangering consequences of liquefaction provide the criterion for eliminating a site from further consideration.

Step 1. Assess the geologic, soil, and groundwater conditions at the site. Information may be obtained from: 1) geologic maps and reports; 2) logs of geotechnical borings drilled at the site and/or on adjacent sites, which are typically contained in foundation engineering reports prepared for a facility; 3) logs of water wells drilled on-site or nearby. Information can be obtained for many areas from geologists of regional U.S. Geological Survey offices, state geological agencies, or local government agencies. The potential for groundwater rise due to seasonal or annual variations should be considered when evaluating groundwater depth.

Step 2. Assess the likelihood of liquefaction-induced ground failure. In general the likelihood can be considered low and the risk as not significant if any of the following conditions are met:

a. The soils within a depth of 30 feet below the groundwater table are not of a type that are susceptible to liquefaction; clays, clayey silts, and bedrock are not susceptible, whereas sands, sandy silts, and gravels may be susceptible. However, a check should be made with a geologist to verify that the region is free of highly sensitive clays that could be susceptible to liquefaction-type behavior.

b. The soils below the groundwater table are older than very late Pleistocene in geologic age (i.e. older than about 20,000 years). It is noted that older soils have been found to have liquefied in certain regions, specifically soils as old as a few hundred thousand years in South Carolina.

c. If soils below the groundwater table are very late Pleistocene (10,000–20,000 years old) and the groundwater level is at least 20 feet below the ground surface; or if the soils below the groundwater table are Holocene (less than 10,000 years old) and the groundwater level is at least 30 feet below the ground surface.

Step 3. Consider the consequences of liquefaction. The occurrence of liquefaction would generally not pose a significant life-safety risk if *all* of the following criteria are satisfied. If these criteria are satisfied, the liquefaction risk is categorized as not significant whether or not it has been characterized as significant from Step 2 above. Thus, in some cases, this step by itself may result in an assessment that the risk is not significant.

a. Check for the potential for building bearing capacity failure or excessive differential settlement:

- *Shallow-foundation-supported structures*—risk is not significant if the vertical distance between the foundation base and the ground water table is equal to or greater than four times the width of the largest foundation, and the soils within that distance are clays or clayey silts.

- *Deep-foundation-supported structures*—risk is not significant if pile or pier foundations derive their bearing support from soils that are clays or clayey silts, or are older in geologic age than very late Pleistocene.

b. Check for the potential for structure flotation:

- Risk is not significant if the ground water table is at least ten feet below the deepest basement level.

c. Check for the potential for lateral spreading-type movements:

- Risk is not significant if the average topography, over any horizontal distance of 300 feet or more, within 2,000 feet of the building has a slope of 0.3° or less and if there is not a free face (e.g., creek channel) within 2,000 feet toward which lateral movements could occur.

6.3.4 Differential Compaction

In general, differential compaction should not pose a significant risk if the liquefaction potential risk is found to be insignificant.

6.3.5 Landsliding

The potential for landsliding lateral movements associated with liquefaction is covered within the evaluation methodology for liquefaction. The following methodology addresses the potential for other, non-liquefaction-associated forms of slope instability.

Step 1. Identify the NEHRP Seismic Map A_a coefficient designation for the facility location. Note: for selected counties, the map area designation should be reviewed using Table 4-3. If the NEHRP Seismic Map Area designation is 3 or lower do not continue the rapid evaluation for landslides; earthquake-triggered landslides are not considered to present significant risk for these NEHRP Seismic Map Areas.

Step 2. If the facility is located above or below a slope but is a horizontal distance of at least three times the slope height from the toe of the slope, the facility is not considered to be at risk due to landslides. Exceptions include sites located near shorelines and underlain by soft soils.

6.3.6 Flooding

Although flooding due to seiche or due to the failure of upstream dams, levees, or water tanks could pose a danger to some facilities, these instances would be relatively rare and the likelihood of their occurrence and their consequences are difficult to evaluate. Therefore, it is not required to evaluate potential flooding due to seiche or to dam, levee, or water tank failure.

A tsunami poses a more likely cause of flooding. In cases where significant flooding potential is found to exist, it would likely affect areas much larger than specific USPS building sites and it may be unfeasible to mitigate the risk to a specific site in many cases. The following methodology may be used to evaluate a tsunami risk.

Step 1. Tsunami waves are a potential danger only to facilities located near coastal waters, or bays and sounds connected to coastal waters. If the facility is not located near such coastal waters, do not continue with the evaluation; tsunamis are not considered to be significant for these facilities.

Step 2. Identify the appropriate tsunami zone for facilities in coastal zones from Figure 6-1.

Step 3. Select the approximate potential maximum tsunami wave height from Table 6-1 for the identified zone.

Table 6-1 Tsunami Wave Heights.

Zone	Wave Height (Feet)
1	5
2	15
3	30
4	50

Step 4. If the ground surface elevation of the facility above sea level is greater than the tsunami wave height from Step 3, there is no significant risk to the facility from the tsunami.

6.4 Methodology for Detailed Evaluation of Earthquake-Related Geologic Phenomena

6.4.1 General

A Detailed Evaluation of earthquake-related geologic phenomena should be performed:
a) when the rapid geotechnical evaluation finds that one or more of the phenomena is likely to be significant for the site; or b) when a Detailed Structural Evaluation is performed. In the latter case, a detailed or supplemental geotechnical evaluation should be performed even if the rapid geotechnical evaluation has not disclosed risks. The rationale for this requirement is that it would be highly undesirable to be unaware of a significant site geologic phenomenon in cases where detailed structural evaluations for ground shaking may lead to decisions for major structural retrofits. Conducting a detailed geotechnical evaluation will lead to increased confidence in the evaluation of risks associated with geologic phenomena for such buildings. Similarly, in cases where a decision to retrofit is made directly based on a rapid structural evaluation (i.e., in those cases that bypass a detailed structural evaluation), the rapid geotechnical evaluation should be confirmed by a detailed geotechnical evaluation during the retrofit design stage.

Detailed Evaluations of earthquake-related geologic phenomena should be conducted by qualified geotechnical engineers, geologists, and seismologists as appropriate.

6.4.2 Scope

The results of rapid evaluations should be used in planning the scope of Detailed Evaluations. In many cases, a Detailed Evaluation will involve relatively minor efforts using existing data (prior geotechnical reports for the site and/or adjacent areas, geologic publications and maps, etc.). Supplemental subsurface investigations (borings, trenches, etc.) are required only when needed to arrive at conclusions regarding the presence of the phenomena and their severity.

The evaluation should address each of the phenomena described in Section 6.2. Key aspects to be considered include:

Surface Fault Rupture—The investigation should be focused toward assessing whether existing faults cross the site, assessing their location with respect to buildings on the site, and evaluating the recentness of their activity. Criteria for defining an active fault having potential for surface fault rupture are given in Section 6.2.1. In cases where active faults traverse an existing building, assessments should be made of the potential displacement associated with surface fault rupture.

Liquefaction—An investigation for liquefaction can take many forms. A review of current methods for assessing liquefaction potential is given by the National Research Council (1985). One acceptable method is to use blow count data from the Standard Penetration Test conducted in soil borings. This method is described in publications by Seed and Idriss (1982), and Seed et al. (1985). Because this field is still in a state of development, the evaluator also needs to be cognizant of new data and publications.

Evaluations of liquefaction potential should include an assessment of the local geology, and site behavior during historical earthquakes should be reviewed when such information is available. This requirement should be followed for assessments of differential compaction, slope stability, and flooding, as well as liquefaction. The consequences of liquefaction should be addressed; potential consequences are described in Section 6.2.2.

Differential Compaction—The amount of differential settlement beneath the building that could result from this phenomenon should be estimated.

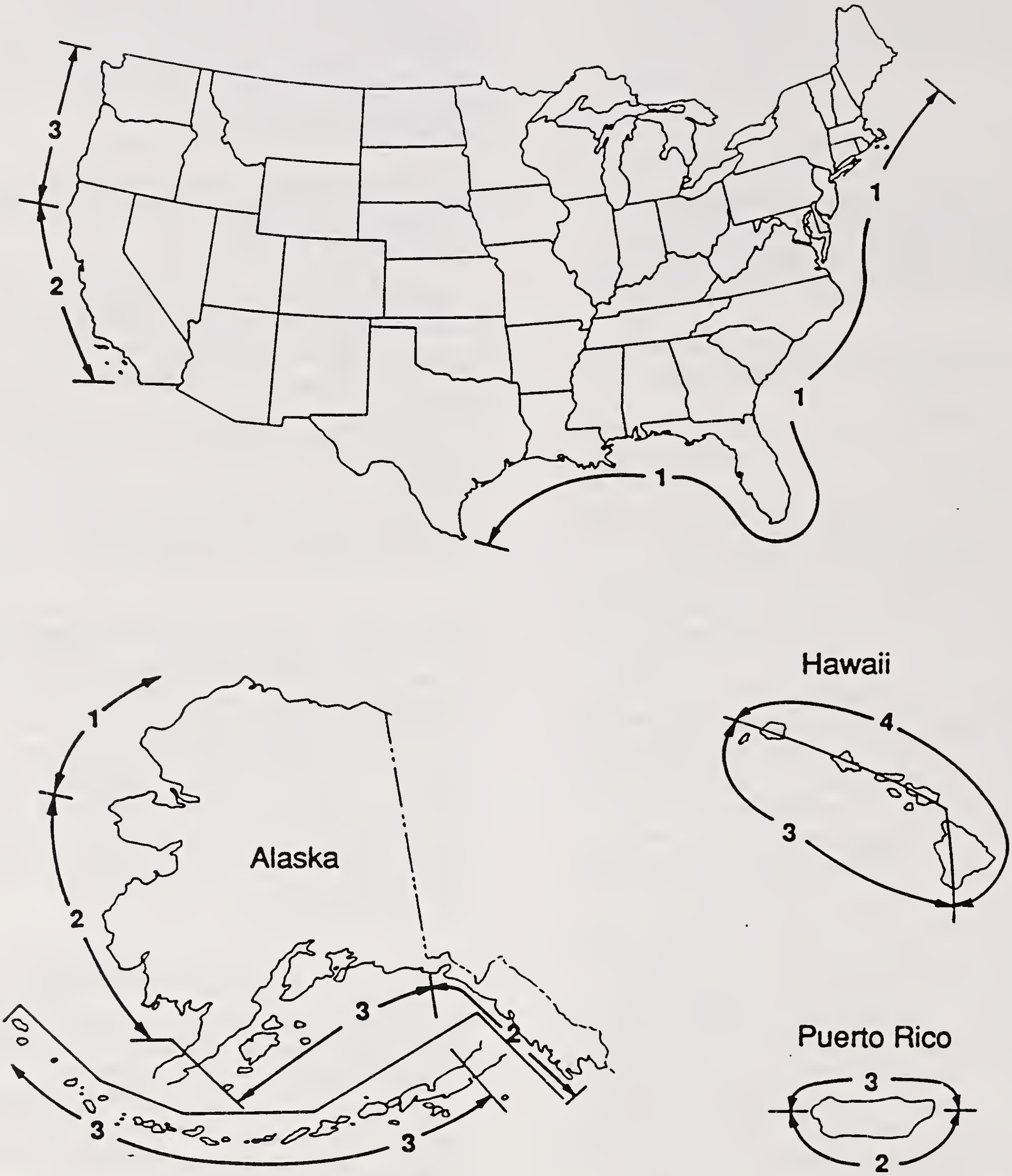


Figure 6-1 *Tsunami zone map.*

Slope Instability—If seismically induced slope deformations at the site are possible, the evaluation should address the potential for deformations large enough to adversely affect the structure. Simplified approaches (e.g., "Newmark" methods) are often sufficient to assess the amplitude of slope deformations.

Flooding—The potential for and extent of flooding associated with the phenomena identified in Section 6.2.5 should be evaluated.

In some cases, it is necessary to estimate earthquake site ground motions (in particular, peak ground acceleration and, in some cases, duration of strong shaking) in carrying out an assessment of geologic phenomena. Except as otherwise required below, site peak ground accelerations may be assumed equal to the values interpolated from the NEHRP Seismic Map A_a coefficient. Note: for selected counties, acceleration values in Table 4-3 should be used. If a probabilistic ground motion analysis is carried out for the site, the site peak ground acceleration should not be lower than that having a 10% probability of exceedance during a 50-year time period. For any site located in a region where active faults have been identified, the site peak ground acceleration should not be lower than the mean value of peak acceleration estimated (using appropriate attenuation relationships) for maximum earthquakes on the active faults. Potential soil amplification effects on ground motions should be considered in selecting values of peak acceleration. Estimates of the duration of strong shaking should generally be based on the assumption of the occurrence of maximum earthquakes in the site region.

If the detailed evaluation discloses any phenomena that present significant risks to the building, then the evaluation should also identify feasible remedial measures that might be used to mitigate the risk at the site. For example, underpinning or grouting might be feasible remedial measures for a building founded directly on liquefiable soils.

6.4.3 Required Documentation

Investigations of geologic phenomena should be documented. The geologic report should as a minimum contain the following:

(1) List of phenomena investigated, which must include the five mentioned in Section 6.4.2

(2) Description of the methods used to evaluate the site for each phenomenon

(3) Results of any investigations, borings, etc.

(4) Summary of findings

(5) Identification of risk mitigation measures, if required with cost estimates for the mitigation measures

APPENDIX B

Post-Yield Approach
(Appendix F, ATC-26-1, United State Postal Service)

The information presented in this appendix was prepared by the Applied Technology Council (ATC) for specific use by the U.S. Postal Service, and issued in report number ATC-26-1. It is reprinted here with the permission of the U.S. Postal Service. While the information is believed to be correct, ATC, the U.S. Postal Service, the Interagency Committee on Seismic Safety in Construction, and the National Institute of Standards and Technology assume no responsiblity for its use by others. Users of information from this publication assume all liability arising from such use.

Appendix F: Post-yield Approach

Sections F.1 through F.7 provide details of the SDG procedures that are first discussed in Section 4.4.1. Section F.8 provides further information on the Special Procedure for unreinforced masonry buildings that is first discussed in Section 4.4.2.

F.1 Evaluation

F.1.1 Evaluation Criteria and Methods

The structure will be evaluated for its ability to resist the combined effects of the seismic forces prescribed herein and the applicable gravity loads within the prescribed lateral distortion limits.

A. *Load combinations*. The demands on the structure will be equal to the combined effects of the dead (D), live (L^*), and seismic (E) loads shown in Formulas F-1 and F-2:

$$\text{Demand} = D + L^* + E\,, \qquad \text{(F-1)}$$

$$\text{Demand} = D + E\,, \qquad \text{(F-2)}$$

where the live load (L^*) is equal to a realistic estimate of the actual live load. The value of L^* may be as low as 25% of the code-prescribed live load.

In Formula F-1, the earthquake load is to be used in the same sense as the gravity loads; in F-2, the earthquake load is to be used in the opposite sense to the dead load.

B. *Lateral displacements and drift limits*.

1. *Drifts*. Interstory drifts shall not exceed 0.015.

2. *Building separations*. Under the conditions of these requirements, some contact between buildings is acceptable if it can be shown that the effects of pounding will not cause loss of function, instability of the affected portion of the structure, or risk to life-safety. For example, if all the floors of adjacent buildings are in vertical alignment with each other, then the pounding associated with the post-yield conditions might cause only some minor local damage to the material in contact. However, if the floor of one building is in alignment with mid-height of columns in the adjacent building, pounding could cause column instability due to buckling and P-Δ effects. If contact is to be avoided, the minimum separation between buildings will be governed by the combined maximum displacements of the adjacent buildings. The maximum story displacements, at respective locations, may be combined by the square-root-of-the-sum-of-the-squares to determine the minimum separation.

3. *P-Δ effects*. The secondary effects of the lateral displacements (Δ) combined with the gravity forces (P) will be investigated. The P-Δ effects in a given story are due to the eccentricity of the gravity loads above the story. If the story drift due to the lateral forces is Δ, the bending moments in the story would be augmented by an amount equal to Δ times the gravity load above the story. The ratio of the P-Δ moment to the lateral-force story moment can be designated as a stability coefficient. If the stability coefficient is less than 0.10 for every story, then the P-Δ effects can be considered insignificant. If, however, the stability coefficient exceeds 0.10 for any story, then the P-Δ effects for the whole building must be determined by a rational analysis.

C. *Overturning*. The structure shall be designed to resist the overturning effects of the seismic loading. In some portions of the structure, the resulting forces may cause uplift at the foundation interface, thus creating an apparent condition of instability. However, structures designed for force levels substantially less than those experienced during actual earthquakes have not exhibited this behavior. Although the state of the art of earthquake engineering has not been able to establish a consistent recommendation for evaluating this condition, it is generally acceptable that buildings can be subjected to rocking on their bases, that the resulting displacements do not approach an incipient overturning condition, and that the maximum displacement is limited by the short time interval between load reversals. When the design engineer determines that uplift conditions exist, two basic choices exist: (1) tie down the foundation to prevent uplift; or (2) do not provide any additional restraint on the potential uplift. The decision requires some judgment of the engineer. If the foundation is tied down, the resulting forces on the structure will generally be increased in the event of a large earthquake because of the added rigidity of the overall structural system. If uplift is allowed to occur, the resulting seismic forces may actually be reduced because of increased energy absorption and the nonlinearity of the base rocking; however, the redistribution of loads to other portions of the foundation may cause some distress in the structure or at the foundation. When uplift is allowed to occur, the designer should provide justification for the assumed redistribution of loads and for the adequacy of the structure and foundation.

D. *Horizontal torsional moments*. Elements that are intended to resist torsion should be located at or near the periphery of the building to maximize torsional rigidity. When this cannot be accomplished or when there are large horizontal eccentricities, the structure must be analyzed for potential torsional instability.

   1. If the torsional component is a substantial amount of the total design force on any element (e.g., one-third of the total), then torsional stability should be evaluated.

   2. Review the mathematical modeling assumptions and calculations to evaluate the validity of the modeling techniques. Determine if uncertainties in assumptions would increase or decrease the torsional characteristics.

   3. Investigate the consequences of the worst-case conditions.

   4. Evaluate the feasibility of revising the lateral-force-resisting system to minimize the effects of horizontal torsional moments.

E. *Structural materials and details*. Structural elements and connections will be evaluated for their ability to sustain the implied ductility demands of the post-yield analysis procedures.

F.1.2 Evaluation Methods

The two methods available for the post-yield analysis: Method 1, the Elastic Analysis Method, and Method 2, the Capacity Spectrum Method, are outlined below; details of the methods are given later in this appendix.

F.1.3 Outline of Method 1: Elastic Analysis Method

This section outlines the Elastic Analysis Method that evaluates overstresses of individual elements. Design examples are illustrated in the SDG (U.S. Army, 1986), Appendix I.

A. Perform a modal analysis of the structure using the appropriate response spectrum.

B. Calculate the forces on all of the structural elements, using the load combinations of Formulas F-1 and F-2. These forces will be defined as the demand forces and denoted with subscript D (e.g., M_D, V_D, F_D).

C. Calculate the yield or plastic capacities of all the structural elements in the same force units used in paragraph (2) above. These forces will be defined as the capacity forces and denoted with the subscript C (e.g., M_C, V_C, F_C).

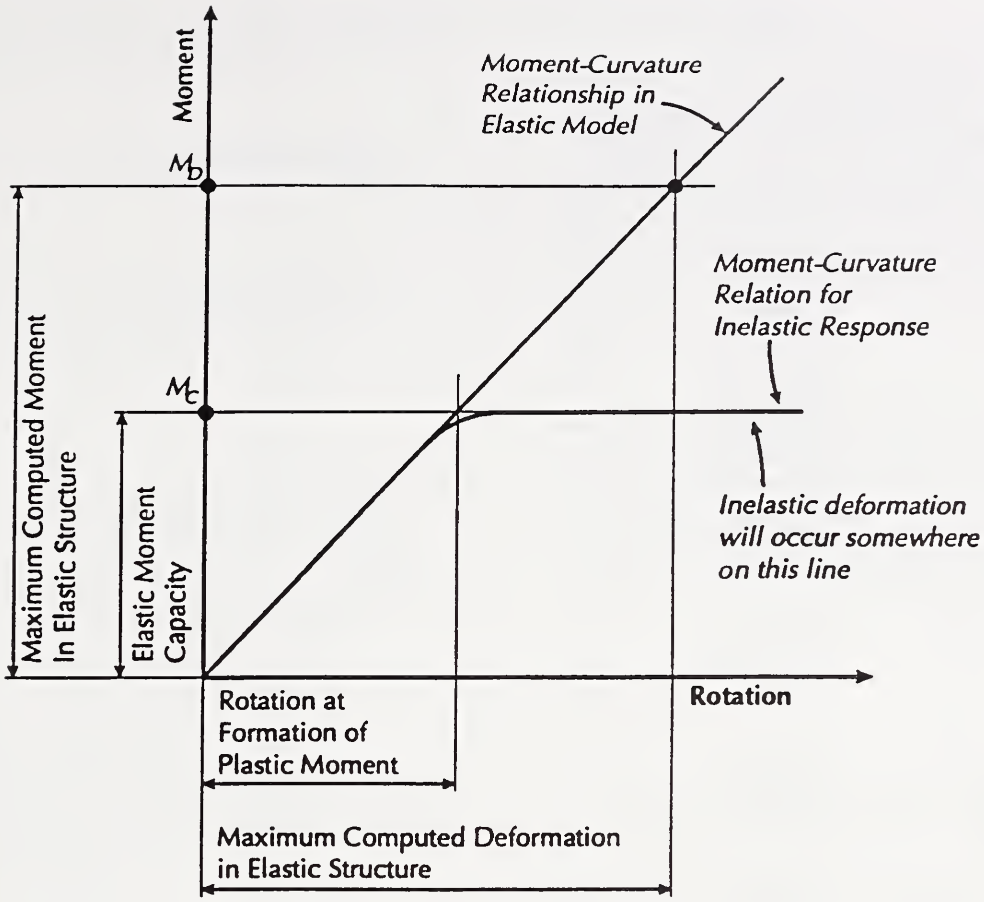
D. Calculate the ratio of the demand forces to the capacity forces of all the structural elements. These ratios are called the inelastic demand ratios. A graphical illustration for flexural members is shown in Figure F-1.

E. Review the inelastic demand ratios. Compare the values to the limits set forth in Table F-1. If any of the following conditions exist, the structure must be analyzed in accordance with Method 2 or the deficiencies must be corrected by a redesign of the critical elements:

   1. Exceeding the inelastic demand ratios of Table F-1.

   2. Unsymmetrical yielding, on a horizontal plane, that will decrease the torsional resistance.

   3. Hinging of columns at a single story level that will cause a collapse mechanism.



$$\text{Inelastic Demand Ratio} = \frac{\text{maximum computed moment in elastic model}}{\text{elastic moment capacity}} = \frac{M_D}{M_C}$$

Figure F-1 *Definition of inelastic demand ratios for flexural members.*

4. High values at discontinuities in vertical elements that could cause instability or fracture.

5. Unusual distributions of inelastic demand ratios.

F. Engineering judgment is required for the structural evaluation of the post-yield analysis. If the review of the inelastic demand ratios satisfies the requirements of paragraph (E) above, it may be assumed that the inelastic drift is adequately approximated by the elastic analysis.

F.1.4 Outline of Evaluation Method 2: Capacity Spectrum Method

This section outlines a step-by-step method for approximating the inelastic capacity of the structure. This capacity is compared to the demands of the response spectrum.

A. Perform a modal analysis using the spectrum of Formula 4-1. Use the resulting element forces (moments, shears, axial forces) to determine the level of the excitation that causes first major yielding of the structure.

B. Revise the structure by modifying its stiuffness or resistance characteristics to represent plastic hinges wherever elements are within 10% of their yield capcities.

C. Apply additional forces to the revised structure in a succession of modal analyses, until an additional group of structural elements reaches their yield capacities.

D. Repeat the above until the combined results reach an ultimate limit (e.g., a collapse mechanism, instability, or excessive distortions).

Table F-1 Inelastic Demand Ratios.

a. Systems conforming to the NEHRP Provisions

Building System	*Element*	*Inelastic Demand Ratio*
Steel		
Ductile Moment Steel Resisting Frames	Beams	3.0
	Columns*	1.75
Braced Frames	Beams	2.0
	Columns*	1.75
	Diagonal Braces †	1.5
	K-Braces ‡	1.25
	Connections	1.25
Tie Rods	Tension only	1.25
Concrete		
Ductile Moment Steel Resisting Frames	Beams	3.0
	Columns*	1.75
Walls:		
(1) Single Curtain of Reinforcing	Shear	1.5
	Flexure	2.0
(2) Double Curtain of Reinforcing	Shear	1.75
	Flexure	3.0
Diaphragms	Shear	1.75
	Flexure	2.0
Masonry Walls	Shear	1.5
	Flexure	2.0
Wood	Trusses	2.0
	Columns*	1.75
	Shear Walls & Diaphragms	3.0
	Connections (other than nails)	2.0

b. Systems not conforming to the NEHRP Provisions

Building System	*Element*	*Inelastic Demand Ratio*
Concrete Frames	Beams	1.75
	Columns*	1.25
Unreinforced Concrete Walls	Shear	1.25
	Flexure	1.0
Unreinforced Masonry Walls	Shear	1.25
	Flexure	1.0

* Axial loads shall not exceed the elastic buckling capacity in any case.
† Full panel diagonal braces with equal number acting in tension and compression for applied lateral loads.
‡ K-bracing and other concentric bracing systems that depend on compression diagonal to provide vertical reaction for tension diagonal.

Note: These values are applicable only to buildings of seismic risk exposure group I as defined in the NEHRP Provisions (BSSC, 1988).

E. Convert the results into a capacity curve based on the periods and spectral accelerations for the fundamental mode of vibration.

F. Graphically compare the demand of the response spectrum to the capacity of the structure.

G. Approximate the lateral deformations and compare to the drift limits given in the evaluation criteria above.

F.2 Ground Motion

The post-yield approach uses response spectra that are developed by the procedures given in this section. The ground motion has a 10% probability of exceedance in 50 years. The spectra are to be considered as the minimum seismic loading criteria. Where there are exceptional site conditions such as close source proximity or highly responsive soil columns, or if the configuration or use of the structure is very different or special, the evaluation should be based on a site-specific response spectrum developed by a geotechnical engineer.

F.2.1 Site Severity

A_a and A_v for a given site location can be obtained from the acceleration contour maps of Figures 4-1, 4-2, 4-3, and 4-4, and Table 4-3.

F.2.2 Site Soil Type

The site soil profile type will be determined and identified as S_1, S_2, S_3, or S_4 according to Table 4-1.

F.2.3 Design Response Spectra

With the known values of A_a and A_v, the 5%-damped acceleration response spectrum is given by Formulas 4-1 and F-3.

For T less than or equal to 4 seconds:

Formula 4-1:

$$S_a = \frac{1.22}{T} A_v S \,,$$

but need not exceed $2.5A_a$.

For T greater than 4 seconds:

$$S_a = \frac{4.88}{T^2} A_v S \,. \qquad \text{(F-3)}$$

Values for S are given in Table 4-1. These formulas for S_a are equivalent to the constant acceleration, velocity, and displacement levels shown on the tripartite, logarithm-scale graph in Figure F-2. A specific example is shown for $A_a = A_v = 0.40$ in Figure F-3. (Linear-scale graphs for S_a are given in Figures F-4 and F-5.)

F.2.4 Damping

All of the design spectra given by Formulas 4-1 and F-3 are for structural damping equal to 5% of critical damping. These spectra may be converted to other damping ratios by the factors given in Table F-2. Linear interpolation may be used to provide factors for intermediate damping values.

F.2.5 Examples

Example 1

1. Site location is Las Vegas, Nevada. The building is of reinforced concrete.

2. Soil type is S_2; $S = 1.2$ from Table 4-1.

3. Find map contour values:

   Figure 4-1, $A_a = 0.07$
   Figure 4-3, $A_v = 0.16$

4. For the basic spectrum, Table F-3 specifies damping of 5%, and from Table F-2 the multiplying factor is 1.00. For the post-yield spectrum to be used in Method 2, the damping from Table F-3 is 10% and the multiplying factor for this spectrum from Table F-2 is 0.80. The basic 5%-damped spectrum is obtained from Formula 4-1 for $T \leq 4$ seconds:

$$\begin{aligned} S_a &= (1.22/T)\, A_v S \\ &= (1.22)\,(0.16)\,(1.2)/T \\ &= (0.23/T) \end{aligned}$$

   with a limiting value of

$$\begin{aligned} S_a &= 2.5\, A_a \\ &= 2.5\,(0.07) \\ &= 0.18 \end{aligned}$$

5. The 5% and 10% damped spectra are shown in Figure F-4.

Table F-2 Damping Adjustment Factors, β.

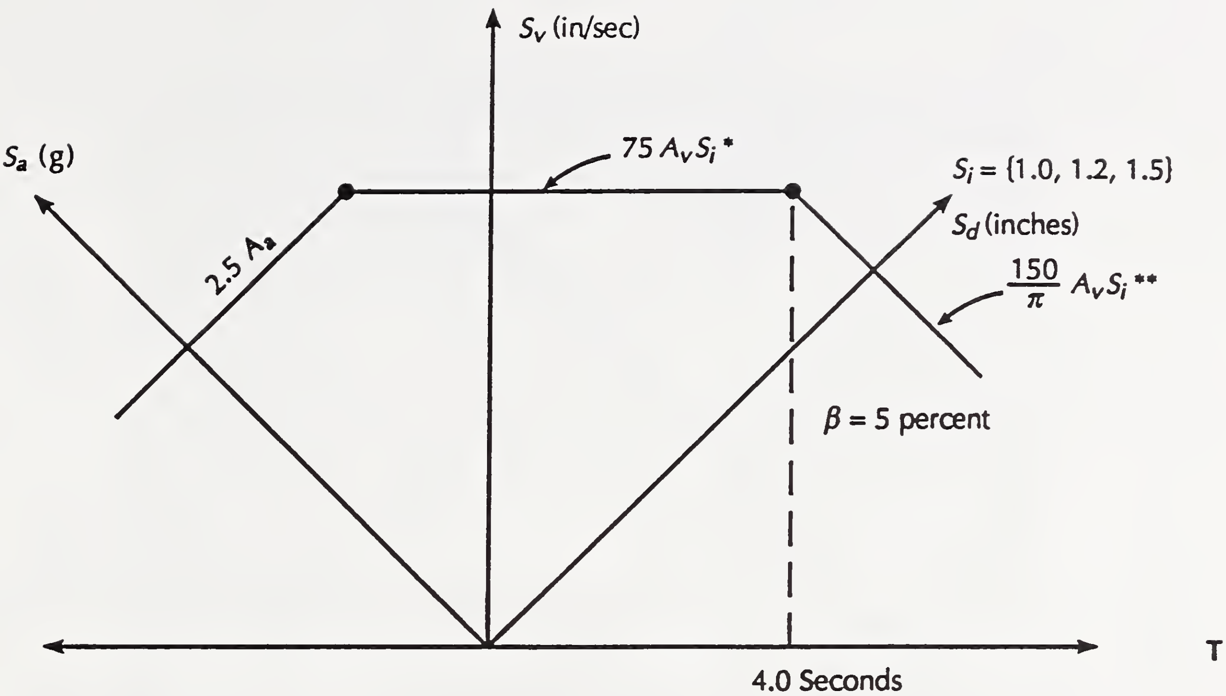
β Percent	*Multiplying Factor for the 5 Percent Spectrum**
2	1.25
5	1.00
7	0.90
10	0.80
15	0.70
20	0.60

* Multiply the value of S_a in Formulas 4-1 and F-3 by these factors to obtain a percent of critical damping other than 5%.

Table F-3 Damping Values for Structural Systems.

Structural System	*Elastic-Linear*	*Post Yield*
Structural Steel	5%	7%
Reinforced Concrete	5%	10%
Masonry Shear Walls	7%	12%
Wood	10%	15%
Dual Systems	(1)	(2)

1. Use the value of the primary, or more rigid, system. If both systems are participating significantly, a weighted value, proportionate to the relative participation of each system, may be used.
2. The value for the system with the higher damping value may be used.



$$* \quad S_v = S_a \times \frac{T}{2\pi} \times g$$
$$= \left(\frac{1.22}{T} A_v S_i \right) \times \frac{T}{2\pi} \times 386$$
$$= 75 A_v S_i$$

$$** \quad S_d = S_a \times \left(\frac{T}{2\pi} \right)^2 \times g$$
$$= \left(\frac{4.88}{T^2} A_v S_i \right) \times \left(\frac{T}{2\pi} \right)^2 \times 386$$
$$= \frac{150}{\pi} A_v S_i$$

Figure F-2 Tripartite spectrum

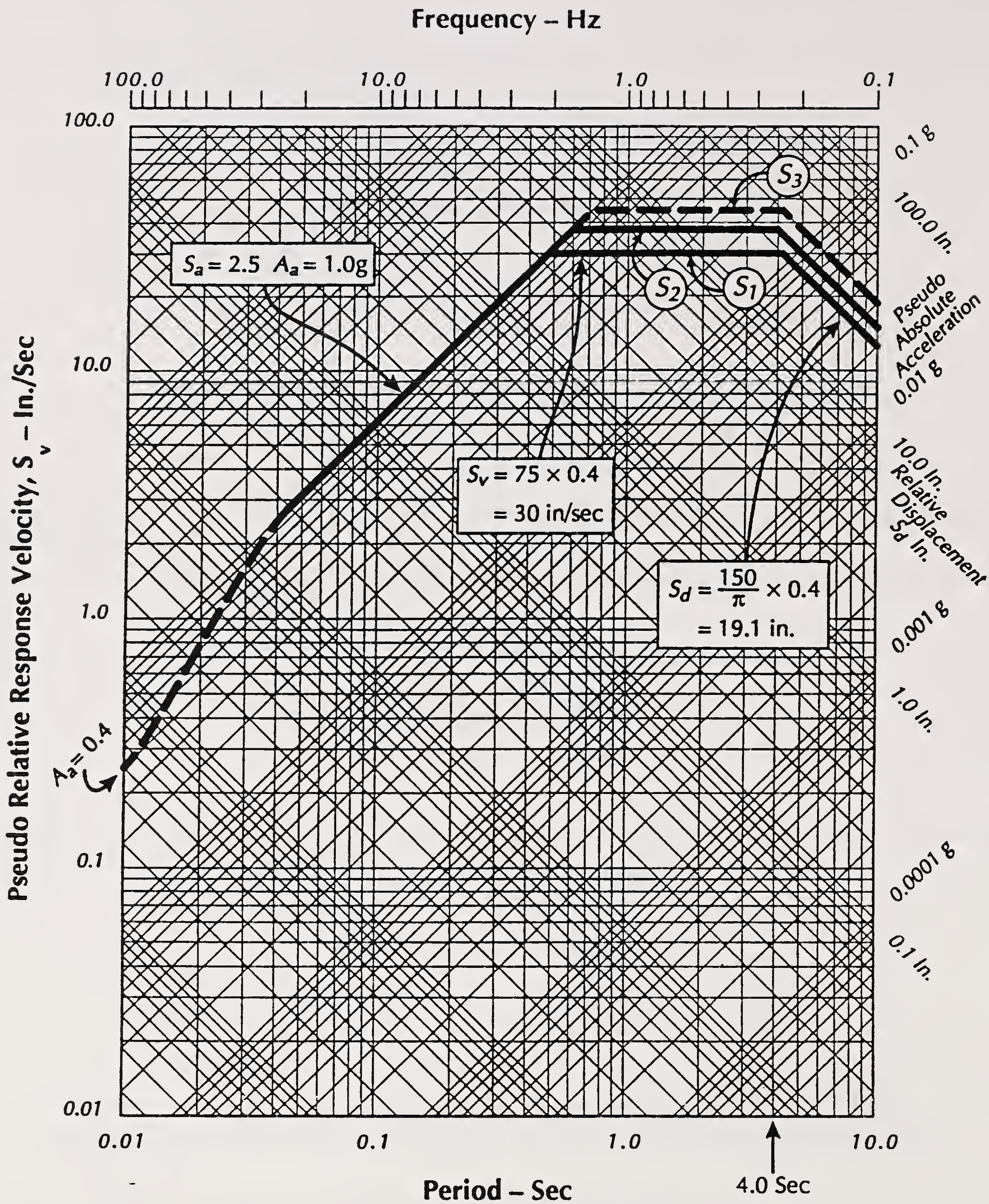


Figure F-3 *Spectra for $A_a = A_v = 0.40$, and $\beta = 5\%$.*

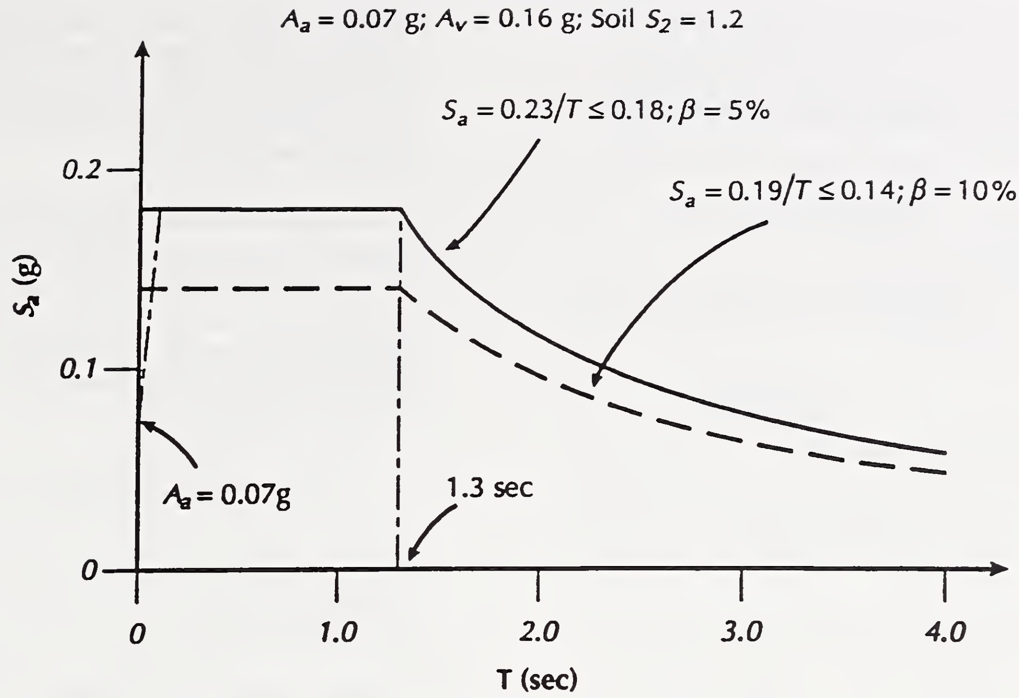


Figure F-4 *Las Vegas, Nevada, site spectra for soil type* S_2.

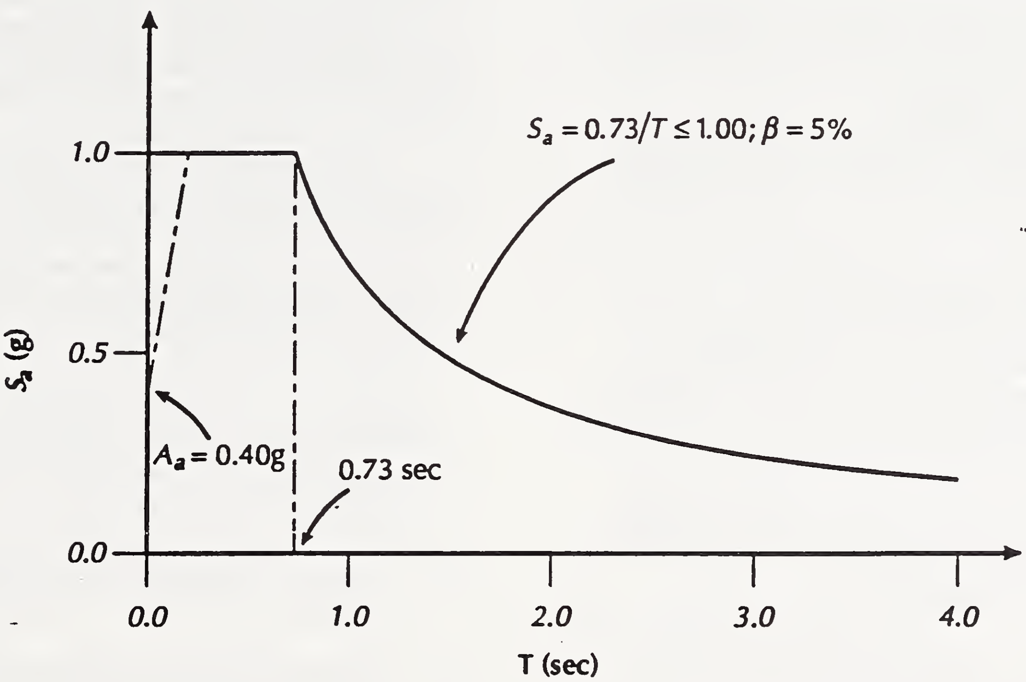


Figure F-5 *Emeryville, California, site spectra for soil type* S_3.

Example 2

1. Site location is Emeryville, California.

2. Soil type S_3; $S = 1.5$ from Table 4-1.

3. Find map contour values:

   Figure 4-1, $A_a = 0.40$
   Figure 4-3, $A_v = 0.40$

4. The 5%-damped spectrum is given by Formula 4-1 for $T \leq 4$ seconds:

$$\begin{aligned} S_a &= (1.22/T)\, A_v S \\ &= (1.22)\,(0.40)\,(1.5)/T \\ &= (0.73/T) \end{aligned}$$

   with a limiting value of

$$\begin{aligned} S_a &= 2.5\, A_a \\ &= 2.5\,(0.40) \\ &= 1.00 \end{aligned}$$

5. This spectrum is shown in Figure F-5.

F.3 Details of Method 1

This section provides the details of Method 1, the Elastic Analysis Method.

F.3.1 Modal Analysis

The modal analysis follows the procedures of Section F.5.

F.3.2 Mathematical Modeling

Some modification to the pre-yield elastic modeling assumptions may be made for the post-yield procedure. (See the discussion of modeling in Section F.6.)

A. Allowances may be made to account for the reduced section properties of cracked or partially cracked concrete.

B. Allowances may be made for flexibility at beam-column joints.

C. Unless the floor slab system is integrated into the design of the beams and girders, composite action need not be considered.

D. The effects of nonseismic frames should be evaluated. These effects would usually be ignored in the mathematical model unless they provide redundancy for the overall lateral-force-resisting system.

E. The effects of nonstructural elements are generally not included in the mathematical model to calculate periods, displacements, and member forces. However, the possible detrimental effects of rigid nonstructural elements must be considered in the overall evaluation of the structure.

F. The modification of modeling assumptions can result in post-yield models having periods of vibration 25% to 50% longer than those obtained from linear-elastic pre-yield models or code formulas.

F.3.3 Stresses and Load Combinations

The loads on the structural elements resulting from the modal analysis must also be combined with the gravity loading, using Formulas 4-6 and 4-7. Only the actual dead load need be considered, and the design live load may be reduced to a value that is consistent with actual live loads that are likely to be in place at the time of a severe earthquake. This reduced gravity loading is justified on the basis of the probability that it is unlikely that both maximum live loads and maximum earthquakes will occur at the same time.

F.3.4 Inelastic Demand Ratios

The Method 1 evaluation procedure assumes that a number of lateral-force-resisting elements will be stressed beyond their elastic limit yield capacities.

A. *Demand.* The calculated forces on the structural elements are obtained from an elastic analysis. Therefore, these are the force demands if the structure had remained elastic.

B. *Capacity.* The capacity is defined as the strength of the element at the point of yielding.

C. *Demand/Capacity Ratio.* The ratio of the demand to the capacity (i.e., the inelastic demand ratio) is an indication of the ductility that may be required for the structural element to withstand the seismic forces. As the first elements of the overall structure begin to yield (i.e., inelastic demand ratio exceeds 1.0), forces will be redistributed to other elements of the lateral-force-resisting system. The limiting values of inelastic demand ratios for structural elements are prescribed in Table F-1. The limiting values have been established as acceptable limits for a structural system that has a reasonable amount of redundancy and is not subjected to premature vertical or torsional instability or to a premature mechanism at a single story level. Possible weak links in the overall structural system are detected by investigating the distribution of the inelastic demand ratios that exceed a value of 1.0. Conditions to be evaluated are listed in the following paragraphs.

D. *Unsymmetrical yielding on a horizontal plane.* This provision is used to check for the possibility of torsional instability, as discussed in Section F.1.1, paragraph D. For example, if all the inelastic demand ratios on the north side of the structure are greater than 1.0, and all the ratios on the south side are less than 1.0, a potential for torsional instability exists. Yielding of the north side will reduce the stiffness of that side of the building relative to the south side; thus the center of rigidity moves to the south. If this condition increases the horizontal eccentricity of the building, torsional moments increase geometrically and the potential for collapse is present.

E. *Hinging of columns at a single story.* This provision is used to check for the possibility of an unstable soft story. For example, if inelastic demand ratios are equal to about 1.5 at the tops and bottoms of 80% of the columns for the first story of a multi-story building and inelastic demand ratios for columns at every other story are less than 1.0, the potential for instability at the first story exists. Because the columns are yielding only at the first story, all the inelastic energy will have to be absorbed at that level. This subjects the first story to the possibility of excessive interstory displacements.

F. *Unusual distributions of inelastic demand ratios.* This is a more general case of paragraphs (D) and (E), above. This provision is used to check the efficiency of the overall lateral-force-resisting system. If a limited number of structural elements have large inelastic demand ratios and the remainder of the elements have ratios less than 1.0, it might be prudent to consider some structural modifications to reduce the potentially high demands on a small number of structural elements.

F.4 Details of Method 2

This section provides the details of Method 2, the Capacity Spectrum Method.

F.4.1 Introduction

The procedure requires the construction of two curves. One curve represents the capacity of the structure to resist lateral forces and the other curve represents the demand of the ground shaking.

A. The capacity curve is developed in a two-step process. First, force (F or V) versus deflection (δ) relationships are calculated. For each stage of yielding of the overall building there is a force-deflection relationship. These are plotted in a curve such as the one shown in Figure F-6. This capacity curve is then converted into a spectral acceleration curve (S_a vs T).

B. *The demand curve* is represented by a composite of two spectra: one for the structure at service level, acting in a linear elastic range with 5% damping, and the other for the structure acting in the inelastic (post-yield) range with higher damping determined from Table F-2.

C. *Reconciliation.* The capacity curve and the demand curve are plotted on the same graph; their intersection is considered to be the reconciliation between demand and capacity.

D. *Example.* The procedure is discussed in the paragraphs that follow, and it is illustrated by a sample building of six stories, 66 feet in height.

F.4.2 General Procedures for Constructing the Capacity Curve

The capacity curve is a simplified global representation of the building capacity. As localized yielding occurs (e.g., bending at the end of a girder), the overall (or global) characteristics of the building are modified. If the localized yielding is at a critical structural element, the

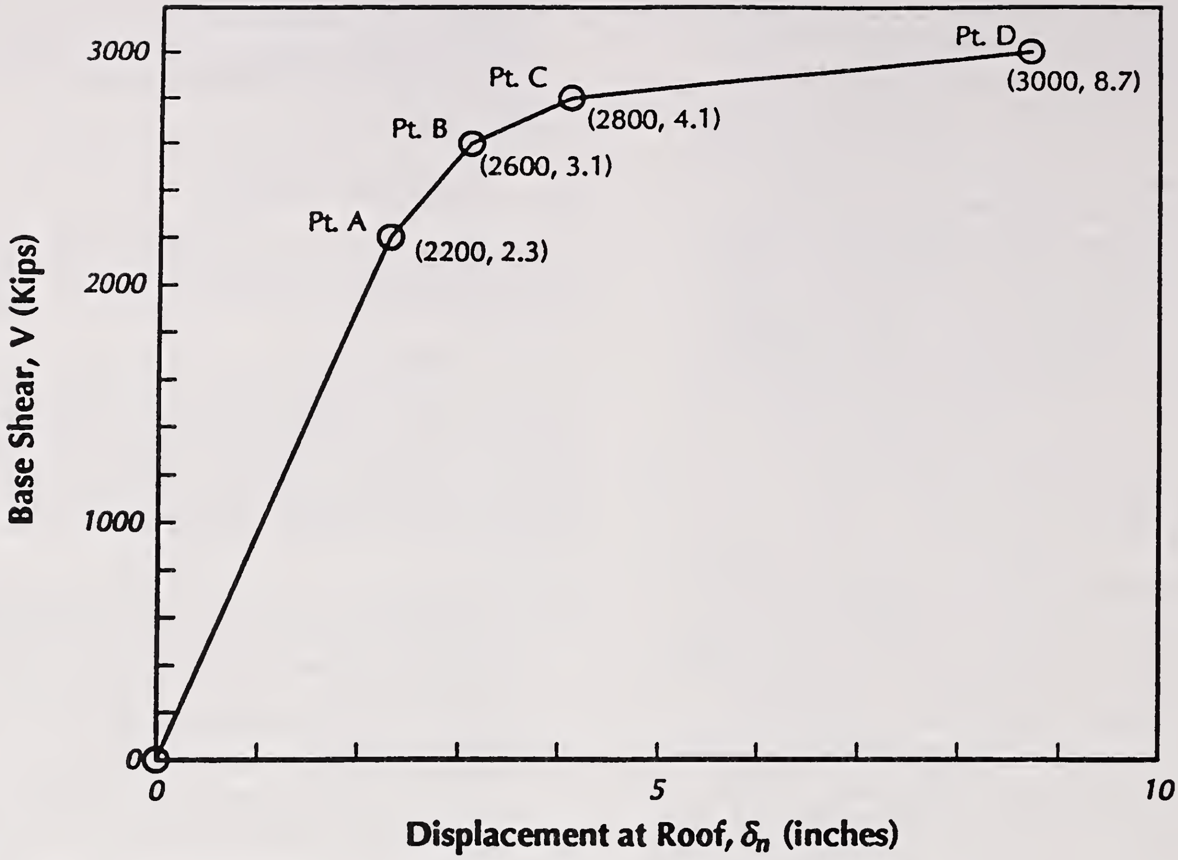


Figure F-6 *Force-displacement capacity curve for six-story building.*

global characteristics may change significantly. Conversely, if the localized yielding is at a redundant location, the change to the global characteristics may be insignificant. For single-story buildings and low-rise buildings up to about 5 stories, the modal analysis procedure for constructing the capacity curve can generally be limited to the fundamental mode of vibration. For taller buildings, effects of higher modes of vibration may become significant; thus a multi-mode analysis may be required. The capacity curve is developed by a step-by-step procedure, using superposition where the structure is laterally distorted to a limiting value and frozen in that position, local yielding elements are relaxed, and the structure is laterally distorted to a new value. The procedure is repeated until an ultimate limit is reached. The capacity curve is constructed by superpositioning straight lines. The period and stiffness characteristics are determined from the secant modulus drawn from the origin to the various points on the force-displacement curve.

F.4.3 Single-Mode Capacity Curve

If it is determined that only the fundamental mode is required (i.e., higher modes are insignificant), the shape of the ground motion response spectrum is not required for the construction of the capacity curve. The following procedure can be set up in tabular form:

A. Determine the elastic capacity (EC) for each structural element (e.g., negative and positive moment capacities at each end of each girder, interaction diagrams at $\varphi = 1.0$ for each column, and shear and moment capacities of shear walls at various key locations). These capacities are defined as the strength of the element at the point of yielding.

B. Determine the net capacity available for earthquake loading in each element using the load combination criteria of Formulas F-1 and F-2. For example, Formula F-1 is for negative moments and Formula F-2 is for positive moments at ends of girders. Note that the net earthquake capacity is reduced by gravity loads when they are in the same sense as seismic loads and the net earthquake capacity is increased by dead loads when in the opposite sense.

$$\text{Net earthquake capacity} = EC - D - L^* \quad \text{(F-4)}$$

$$\text{Net earthquake capacity} = EC + D \quad \text{(F-5)}$$

C. Perform a modal analysis using the spectrum defined by Formula 4-1. Obtain member forces (moments, shears, axial forces).

D. Divide the net earthquake capacities for each element by the corresponding earthquake forces. This gives the local elastic capacity ratio for each element. Find the element with the lowest ratio, or the group of elements whose ratios fall within 10% of the lowest. These elements will yield first: they define the global elastic capacity ratio for the structure.

E. Establish the point of initial major yielding, the first point on the capacity curve, by multiplying the base shear and lateral roof displacement by the global elastic capacity ratio for the structure. This point is represented as point A by $V = 2{,}200$ Kips and $\delta_n = 2.3$ inches for the sample six-story building characterized in Figure F-6.

F. Determine the first post-yield segment of the capacity curve. The structure is essentially frozen at the point of initial major yielding. The balance of net capacity in each element still available for additional earthquake loading is tabulated. Elements that are at or near (e.g., within 10%) their yield capacities are modeled as plastic hinges (e.g., beam elements might have their moments of inertia reduced to 5% of their elastic values). Lateral forces proportional to the fundamental mode shape are applied to the revised mathematical model. For the sample six-story building, the base shear of the applied forces was 1,000 Kips. The resulting forces on the elements were compared to the balance of net earthquake capacities, and lateral displacements were calculated. It was determined that 40% of the applied loads will form a new group of yielding elements. A second point on the capacity curve was determined at $V = 2{,}600$ Kips and $\delta_n = 3.1$ inches (2,200 Kips at point A plus 40% of 1,000 Kips and 2.3 inches at point A plus 40% of 2.0 inches), represented by point B in Figure F-6.

G. Determine sequential post-yield segments on the capacity curve by repeating the procedure in E above (e.g., points C and D in Figure F-6, using revised mode shapes and mathematical models).

H. The procedure is repeated until a failure mechanism, instability, or excessive deformations occur. Rotational ductility demands can be approximated by using M/EI diagrams of the yielding girders, taking into account the reduced EI's used in the yielding mathematical model. Ductility demands for flexure should not exceed 2 times the Inelastic Demand Ratios of Table F-1, and for all other conditions they should not exceed the values shown in Table F-1. Interstory displacements are determined by superposition of the lateral story displacements of the sequential models. For the sample six-story building, the ultimate global capacity of the structure is represented by point D at $V = 3{,}000$ Kips and $\delta_n = 8.7$ inches in Figure F-6.

I. Determine lateral displacements and drift demands. The capacity curve is converted to S_a and T coordinates and superimposed on the response spectrum curve. If the curves do not intersect, irreparable damage or collapse of the structure is anticipated. If the curves do cross, the intersection can be used to approximate the response of the structure. For the sample six-story building, the force-deflection curve of Figure F-6 is converted into the spectral curve of Figure F-7. A table for calculations is set up as shown in Table F-4. Spectral values are calculated for each of the yield points, A, B, C and D. Base shear (V) and roof deflection (δ_n) are entered from Figure F-6. The quantity V/W is calculated. The spectral qualities are determined by Formulas F-6, F-7 and F-8. The modal roof participation factor (PF_N) and the effective modal weight (α) are calculated from Formulas F-10 and F-11.

$$S_a = \frac{V/W}{\alpha}, \quad \text{(F-6)}$$

$$S_d = \frac{\delta_N}{PF_N}, \quad \text{(F-7)}$$

$$T = 2\pi\sqrt{\frac{S_d}{S_a g}}. \quad \text{(F-8)}$$

Table F-4 Conversion of V and δ_n to S_a and T.

Point	V (kips)	δ_N (in)	V/W	PF_N	α	Sa (g)	S_d (in)	T (sec)
A	2200	2.3	0.22	1.30	0.78	0.280	1.77	0.80
B	2600	3.1	0.26	1.28	0.80	0.325	2.42	0.87
C	2800	4.1	0.28	1.28	0.80	0.350	3.20	0.97
D	3000	8.7	0.30	1.26	0.83	0.361	6.90	1.40

See Figure F-8 (V and δ_N) See Figure F-9 (Sa and T)

The values of S_a and T are used to construct the spectral capacity curve of Figure F-7. Use the 5%-damped demand curve for the elastic capacity ($T < 0.80$ sec) and the 10%-damped demand curve for the ultimate capacity ($T > 1.4$ sec). A transition curve is drawn between $T = 0.80$ sec and $T = 1.4$ sec. The intersection of the capacity and demand curves is about $S_a = 0.35$g and $T = 1.0$ seconds. The lateral story displacements at this intersection are calculated from Formula F-9.

$$\delta_N = PF_n S_a (T/2\pi)^2 g \qquad \text{(F-9)}$$

$$= 1.28 \times 0.35 \times (1/2\pi)^2 \times 386$$

$$= 4.38 \text{ inches}$$

The roof displacement equals about 4.4 inches for a six-story building, 66 feet high. Maximum interstory displacements can be obtained from a composite deflected shape estimated from the sequential incremental analysis done above. For the sample building, the average interstory drift is 0.73 inches. The maximum interstory drift, which is at the second story, equals 1.1 inch or 0.0083 times the story height. Thus, it satisfies the requirements of drift (i.e., less than 0.015) as prescribed in the evaluation criteria.

J. The results of this procedure give an estimate of the inelastic response of a building to a severe earthquake. In general, it will result in lower force levels and larger displacements than the results of Method 1. Neither procedure is necessarily more accurate than the other; however, an evaluation of both procedures should give the designer enough insight to determine the weak links of the structural system, evaluate the potential for instability, and suggest needs for possible structural modifications.

F.4.4 Multi-Mode Capacity Curve

If it is determined that the higher modes are significant, a multi-mode analysis is required. The procedure for constructing the multi-mode capacity curve is the same as the procedure for the single-mode capacity curve, with the following exceptions:

A. Same as paragraph F.4.3A concerning the single-mode capacity curve.

B. Same as paragraph F.4.3B.

C. Same as paragraph F.4.3C, except that the corresponding earthquake loads are determined by a multi-mode analysis.

D. Same as paragraph F.4.3D, except that only the fundamental mode component of the base shear and lateral roof displacement are multiplied by the global elastic capacity ratio. For example, assume the data for the seven-story building in Table F-5 and Figure F-8 represent the initial major yielding for the structure. The multi-mode base shear is 2,498 Kips, but the fundamental mode component is 2,408 Kips. The multi-mode roof displacement is 0.229 feet and the fundamental mode roof displacement is 0.228 feet. Although 2,498 Kips represents the forces used to determine the initial major yielding in the building, the values of 2,408 Kips and 0.228 feet represent the "point A" used in the capacity spectrum (i.e., such as shown for the six-story building in Figure F-6).

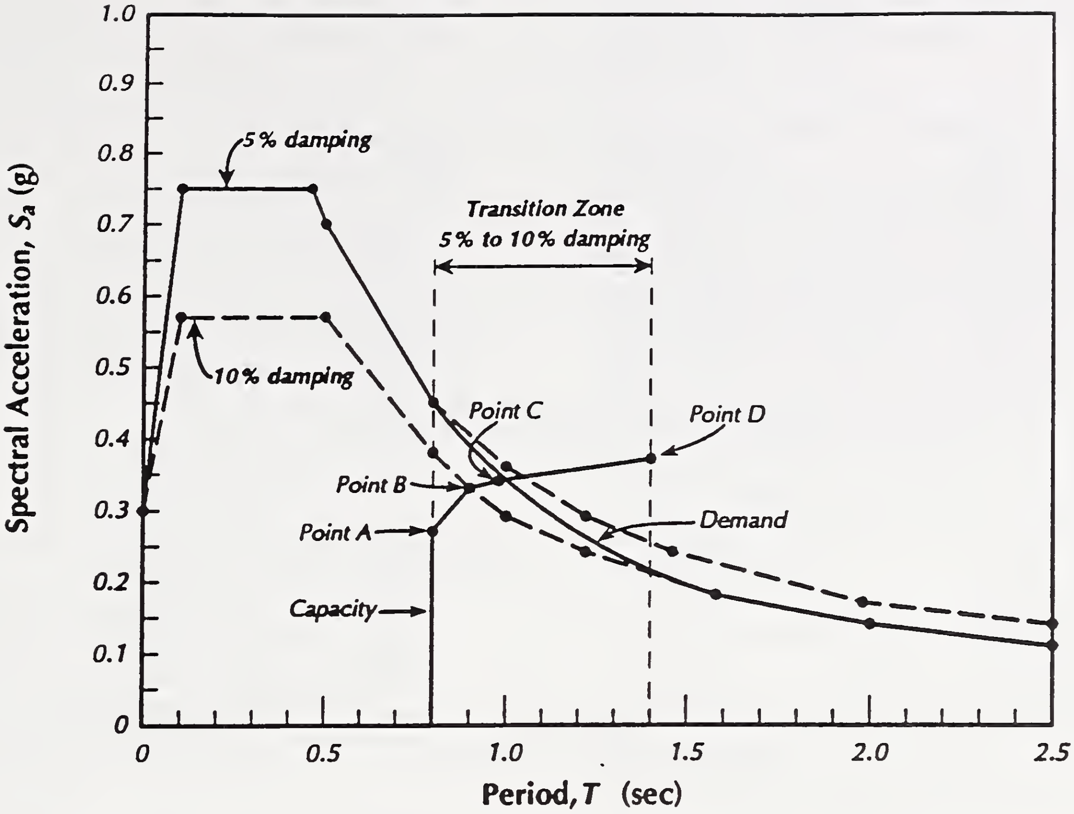


Figure F-7 *Capacity spectrum method curve for six-story building.*

Table F-5 Seven-Story Building—Transverse Direction—Summary of Modal Analysis.

Story		*Forces (kips)*				*Shears (kips)*				*OTM (k–ft)*			
Level	*Wt kips*	*1*	*2*	*3*	*SRSS*	*1*	*2*	*3*	*SSRS*	*1*	*2*	*3*	*SSRS*
roof	1410	508	–330	170	629	508	–330	170	629	0	0	0	0
7	1460	494	–188	–10	529	1002	–518	160	1139	4420	–2871	1479	5474
6	1460	443	19	–166	473	1445	–499	–6	1529	13137	–7378	2871	15338
5	1460	371	216	–163	459	1816	–283	–169	1846	25709	–11719	2819	28394
4	1460	282	329	–6	433	2098	46	–175	2106	41508	–14181	1349	43884
3	1460	185	319	156	400	2283	365	–19	2312	59761	–13781	–174	61330
2	1830	125	267	219	367	2408	632	200	2498	79623	–10605	–339	80327
ground		0	0	0	0					112131	–2073	2361	112175

	Acceleration (g)				*Displacement (ft)*				*Insterstory Drift (ft)*				
Story	*1*	*2*	*3*	*SRSS*	*1*	*2*	*3*	*SSRS*	*1*	*2*	*3*	*SSRS*	*Δa/hs*
roof	.360	–.234	.121	.446	.228	–.016	.003	.229	.014	.007	.003	.016	.0018
7	.338	–.129	–.007	.362	.214	–.009	.000	.214	.022	.010	.003	.024	.0028
6	.303	.013	–.114	.324	.192	.001	–.003	.192	.031	.009	.000	.032	.0037
5	.254	.148	–.112	.315	.161	.010	–.003	.161	.039	.005	.003	.039	.0045
4	.193	.225	–.004	.297	.122	.015	.000	.123	.042	.000	.002	.042	.0048
3	.127	.219	.107	.275	.080	.015	.002	.081	.037	.005	.001	.037	.0043
2	.068	.146	.120	.201	.043	.010	.003	.044	.043	.010	.003	.044	.0033
ground	0	0	0	0	0	0	0	0					

US Army Corps of Engineers

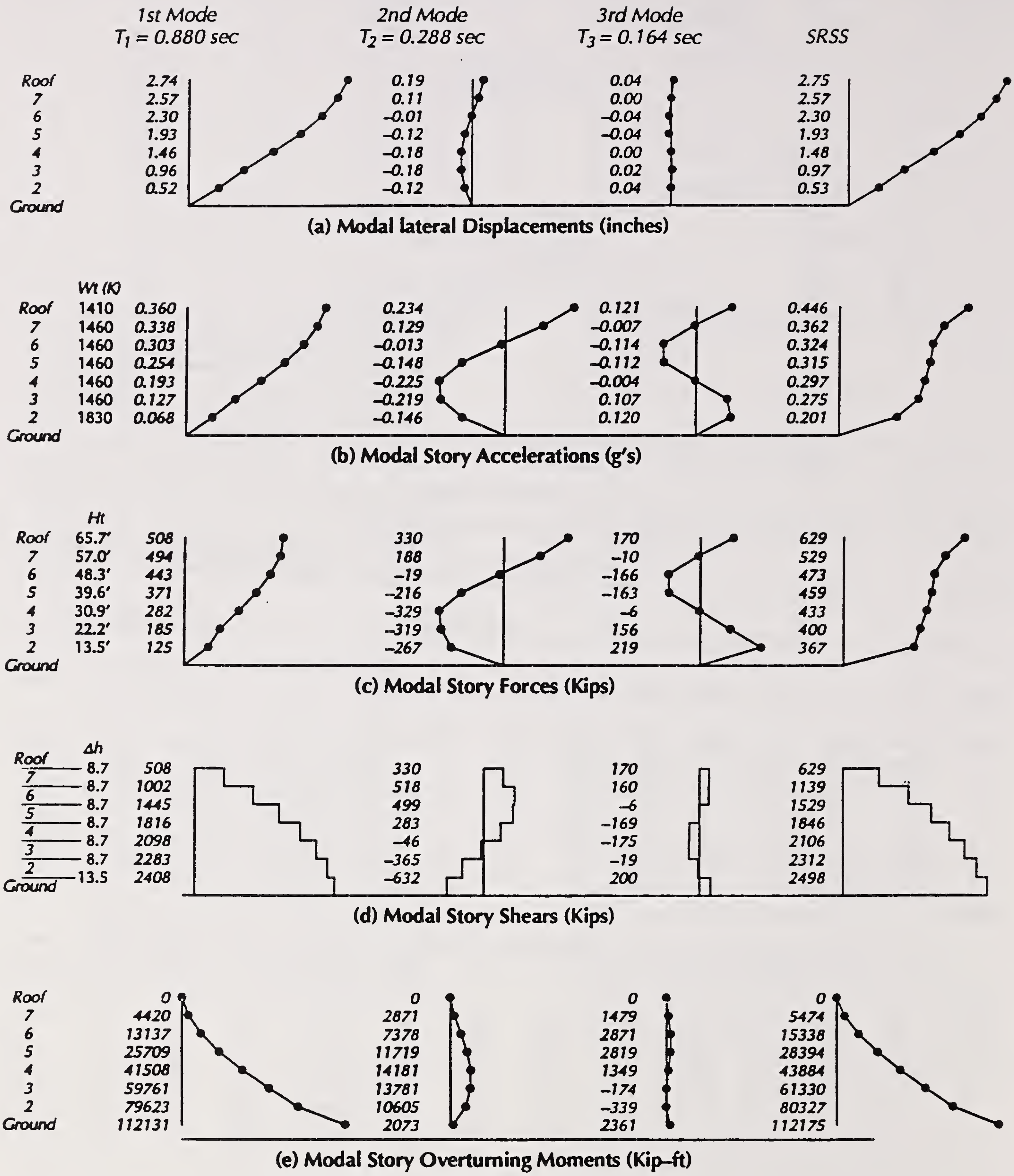


Figure F-8 Sample modal analysis of a seven-story building.

E. Same as paragraph F.4.3E, except that the lateral forces are applied by means of a multi-mode response spectrum analysis. If the response spectrum with a peak ground acceleration of 0.10 g is applied to the revised mathematical model and it is determined that 40% of the resulting multi-mode forces will form a new group of yielding elements, the second segment of the capacity curve is determined by using 40% of the fundamental mode component of base shear and lateral roof displacement. This is the same as finding the spectral acceleration for the first mode period on a response spectrum that has a peak ground acceleration of 0.04 g (i.e., 40% of 0.10 g). First-mode spectral acceleration and period can be converted to base shear and roof displacement in the same manner as shown for the six-story building in Table F-4. As in paragraph F.4.4D above, the forces in the elements are determined by the multi-mode analysis, but the capacity spectrum is represented by the fundamental mode component.

F. Same as paragraph F.4.3F, with the exceptions noted in paragraph F.4.4E above.

G. Same as paragraph F.4.3G, except that the interstory displacements determined by superposition of fundamental modes represented in the capacity curve must be increased proportionally to represent the multi-mode analysis. For example, the interstory drifts between the sixth and seventh stories in Table F-5 are 0.024 feet for the multi-mode analysis and 0.022 feet for the fundamental mode. Therefore, interstory displacements determined by superpositions of the sequential fundamental modes will be increased by a factor of 0.024/0.022 = 1.09. Between the third and fourth floors, the values are the same and no correction is required.

H. Same as paragraph F.4.3H, except that the lateral displacements that represent the first-mode component must be increased proportionally to also represent the multi-mode components. For example, roof displacements will be increased by a factor of 0.229/0.228 = 1.004.

I. Same as paragraph F.4.3I above.

Variations of the procedures outlined above for constructing a capacity curve are acceptable with justification.

F.5 The Method of Modal Analysis

For a building that is regular and essentially symmetrical, a two-dimensional model (a vertical plane with vertical and horizontal movement within the plane) will generally be sufficient for the modal analysis of the structure in each of its two horizontal components of motion. When a structure is not symmetrical in plan, has discontinuities in the vertical or horizontal planes, has large length-to-width ratios, has flexible horizontal diaphragms, or has other irregularities, a three-dimensional model will be required for the modal analysis.

F.5.1 Two-Dimensional (2-D) Models

The modal analysis procedure for two-dimensional models is outlined in paragraphs (A) through (I) below. Variations of this procedure may be acceptable with proper justification and approval.

A. *Mathematical model.* The building will be modeled as a system of masses lumped at each floor level, each mass having one degree of freedom, that of lateral displacement in the direction under consideration. The stiffness of the lateral-force-resisting system will be determined by established methods in accordance with the modeling guidelines given in Section F.6.

B. *Mode shapes and periods of vibration.* The analysis will include, for each major axis, all significant modes of vibration, with a minimum of three modes for buildings with six or more stories. The relative significance of higher modes will be determined by the values of modal participation factors and modal spectral accelerations. See the discussion of computer programs in Section F.7. The natural periods and mode shapes will be computed by established methods of structural mechanics.

C. *Modal story participation factor.* This factor will be calculated for each mode using Formula F-10:

$$PF_{xm} = \left(\frac{\sum_{i=1}^{n} \frac{w_i}{g} \varphi_{im}}{\sum_{i=1}^{n} \frac{w_i}{g} \varphi_{im}^2} \right) \varphi_{xm} , \quad \text{(F-10)}$$

where

PF_{xm} = modal participation factor at level x for mode m
w_i/g = mass assigned to level i
φ_{im} = amplitude of mode m at level i
φ_{xm} = amplitude of mode m at level x
n = level n

It should be noted that some references define the "modal participation factor" as the quantity within the brackets in Formula F-10 above. Also, in some references, φ is normalized to 1.0 at the uppermost mass level and other references will normalize the value of $\Sigma(w/g)\varphi^2$.

D. *Modal base shear participation factor.* The effective modal weight (or modal base shear participation factor) will be calculated for each mode using Formula F-11:

$$\alpha_m = \frac{\left(\sum_{i=1}^{n} \frac{w_i}{g} \varphi_{im} \right)}{\sum_{i=1}^{n} \frac{w_i}{g} \sum_{i=1}^{n} \frac{w_i}{g} \varphi_{im}^2} \varphi_{xm} , \quad \text{(F-11)}$$

where

α_m = modal base shear participation factor for mode m ($\alpha_m = C_{bm}/S_{am}$ where C_{bm} is the modal base shear coefficient and S_{am} is the modal spectral acceleration)

E. *Modal story lateral forces.* The lateral forces for mode m are calculated using Formula F-12:

$$F_{xm} = PF_{xm} S_{am} w_x , \quad \text{(F-12)}$$

where:

F_{xm} = story lateral force at level x for mode m
w_x = weight at or assigned to level x
S_{am} = spectral acceleration for mode m from the design response spectrum (as a ratio of the acceleration of gravity, g)

F. *Modal base shear.* The total lateral force corresponding to mode m is calculated using Formula F-13:

$$V_m = \alpha_m S_{am} W , \quad \text{(F-13)}$$

where

V_m = Total lateral force for mode m
W = Total dead load of the building and applicable portions of other loads

G. *Modal shears and moments.* Story shears and overturning moments for the building and shears and flexural moments for the structural elements will be computed for each mode separately by linear analysis in conformance with the story forces determined by Formula F-12.

H. *Modal deflections and drifts.* Modal lateral story displacements will be calculated using Formula F-9:

$$\delta_{xm} = PF_{xm} S_{dm} = PF_{xm} S_{am} \left(T_m / 2\pi \right)^2 g ,$$

where

δ_{xm} = lateral displacement at level x for mode m
S_{dm} = spectral displacement for mode m calculated from the response spectrum
T_m = modal period of vibration

The modal interstory drift in a story, δ_{xm}, will be computed as the difference of the displacements, δ_{xm}, at the top and bottom of the story under consideration (i.e., where $\Delta_{xm} = \delta_{(x+1)m} - \delta_{xm}$).

I. *Combinations of modal values.* The combined effects of the individual modal actions (shears, moments, axial forces, etc.) and deformations (lateral story displacements, interstory drifts, etc.) for the structure and the members will be obtained by taking the square-root-of-the-sum-of-the-squares (SRSS) of the values of all significant modes. These total values are subject to modification by other provisions (e.g., torsional, orthogonal).

F.5.2 Three-Dimensional (3-D) Models

When a 3-D analysis of a building is used or is required, some modifications to the procedure outlined for 2-D models will be necessary. These modifications will be most significant for structures with large eccentricities, for structures that do not have an orthogonal axis of symmetry, and for structures where the forces are applied from a direction that is not parallel to one of the major axes of the building.

A. At each floor level, there will be three degrees of freedom. The primary displacement will generally occur in the component parallel to the direction under consideration. There will also be a displacement component normal to the direction under consideration and a rotation component about the vertical axis of the building. When the floor diaphragm is not rigid, the horizontal flexibility will be considered.

B. A minimum of nine modes will be required in order to include three horizontal modes in each of the principal directions and three torsional modes. The possible coupling effects of the various components of motion will also be investigated.

C. Modal story participation factors in Formula F-10 will be adjusted for 3-D effects.

D. Modal base shear participation factors in Formula F-11 will be adjusted for 3-D effects.

E. Modal story lateral forces will have three components: primary forces in the direction under consideration, forces normal to the direction under consideration, and a torque due to rotational motion.

F. Modal base shears will have three components consistent with E above.

G. Modal shears and moments will be determined from three components consistent with E and F above.

H. Modal displacements and drifts will vary within the horizontal plane of each floor level as well as along the vertical axis.

I. The total forces and deformations for the structure and the members will be obtained by an approved method to account for a rational combination of the modal values.

F.5.3 Applications of Modal Analysis

Modal analysis requires a design response spectrum and modal periods, mode shapes, and participation factors. The accuracy of these elements and the degree of sophistication required for the analysis depend on the size and complexity of the building. The following paragraphs present some of the basic cases.

A. *Single-story building.* Unless the building is unusual or irregular in plan, the modal analysis procedure essentially becomes equivalent to a static design procedure.

   1. The period of vibration will generally be in the range of 0.1 to 0.2 seconds, thus placing it at the peak of the response spectrum for a maximum value of S_a. Note that the peak of the response spectrum is assumed to extend back to $T = 0$ for the fundamental mode as noted in the SDG (U.S. Army, 1986), Figure 5-4. (Also see Figures F-4 and F-5.) In general, even a very rigid structure with a short natural period of vibration will respond at a slightly longer period due to soil-structure interaction.

   2. For a single-story building, the base shear participation factor will be equal to unity (e.g., $\alpha = 1.0$). Therefore, the base shear coefficient will be equal to the spectral acceleration, S_a.

   3. The total lateral force on the building, for each direction of motion, will be equal to the spectral acceleration times the weight of the building ($V = S_a \times W$) in accordance with Formula F-3.

B. *Low-rise buildings up to about 5 stories.* Unless the building is unusual or irregular in elevation or plan, the modal analysis can generally be limited to the fundamental mode of vibration. Although the use of a computer program will generally be more efficient and will generally give more accurate results, the single-mode analysis can be done by hand calculations.

1. Estimate the fundamental period of vibration using Formula 4-3 or 4-4, assume a straight-line mode shape, and calculate or estimate the story weight.

2. Calculate the model participation factors PF_x and α. Approximate the spectral acceleration, S_a, for the estimated period using the response spectrum.

3. Calculate the story forces, F_x (refer to SDG (U.S. Army, 1986) Appendix E, design example E-1, for the procedure).

4. Calculate the deflected shape of the structure. This can be done by hand calculations (though somewhat difficult and time-consuming) or with the aid of a computer program.

5. Use the calculated deflected shape as a new estimate for the mode shape and repeat paragraphs (2) and (3) above.

6. If the story forces of paragraph (5) compare favorably with the original values of paragraph (3) (e.g., within about 10%), assume the deflected shape of paragraph (4) to be acceptable. If not, repeat paragraph (4) to calculate the deflected shape for the revised story forces.

7. Calculate the period of vibration using Formula 4-5. A quicker method is by means of the following equation, using the forces and displacements calculated above:

$$T = 2\pi\sqrt{\frac{\delta_n w_n}{F_n g}}, \qquad \text{(F-14)}$$

   where δ_n, w_n, and F_n are the displacement, weight, and force at the roof. This equation can be derived from Formulas F-9 and F-12.

8. If the period of vibration calculated in paragraph (7) above is substantially different than the value assumed in paragraph (1) above, repeat paragraph (2) and adjust the forces and displacements in proportion to the new value for S_a.

C. *Moderate-rise buildings from 5 to 15 stories.* For buildings over 5 stories, some of the effects of higher modes of vibration may be significant. In lieu of a detailed analysis, the dynamic characteristics can be approximated. Table F-6 shows the general modal relationships for a fairly uniform seven-story reinforced concrete frame building. For a fourteen-story building, a modal analysis could be approximated as follows:

1. Estimate the fundamental period of vibration.

2. Approximate periods for the second through fifth modes of vibration using the ratios shown in Table F-6 (e.g., second mode period equals 0.327 time the fundamental mode period).

3. Approximate the model shapes by using the shapes shown in Table F-6 and interpolating for the taller structure (e.g., for the second mode, assume 1.00 for the roof and 0.550 for the 13th story; estimate the 14th story at 0.775).

4. The participation factors can be taken directly from Table F-6 or new values can be calculated from the mode shape by using Formulas F-10 and F-11.

5. Determine the spectral accelerations, S_a, for each modal period from the response spectrum.

6. Calculate story forces for each of the modes as shown in the SDG (U.S. Army, 1986) Appendix E, design example E-1. The results for the 7-story building are summarized in Table F-5 and are illustrated graphically in Figure F-8.

7. Calculate the deflected shape of the building separately for each mode of vibration. This will generally require the use of a computer. Compare the deflected shapes to the mode shapes approximated in paragraph (3), above. (Note: some computer programs will perform paragraphs (1) through (7), above, directly.) If the shapes are similar, continue with the analysis. If there are significant differences in mode shapes, a modification of paragraphs (4) through (7), above, may be required.

Table F-6. General Modal Relationships.

Mode		1	2	3	4	5
Period (seconds)		0.880	0.288	0.164	0.106	0.073
Ratio of Period to 1st Mode Period		1.000	0.327	0.186	0.121	0.083
Participation Factor at Roof		1.31	−0.47	0.24	−0.11	0.05
Base Shear Participation		0.828	0.120	0.038	0.010	0.000
Mode Shape at Story Levels (Normalized)	Roof	1.000	1.000	1.000	1.000	1.000
	7	0.938	0.550	−0.059	−0.852	−1.749
	6	0.839	−0.056	−0.942	−1.080	0.194
	5	0.703	−0.631	−0.921	0.526	1.674
	4	0.535	−0.961	−0.034	1.259	−1.068
	3	0.351	−0.933	0.883	−0.080	−1.139
	2	0.188	−0.625	0.990	−1.150	1.310
	1	0	0	0	0	0

8. Calculate the periods of vibration using Formula 4-5. An alternate method is to use Formulas F-9 and F-12 and solve for T_m for each mode at several story levels as follows:

$$T_m = 2\pi \sqrt{\frac{\delta_{xm} w_x}{F_{xm}\, g}}. \qquad \text{(F-15)}$$

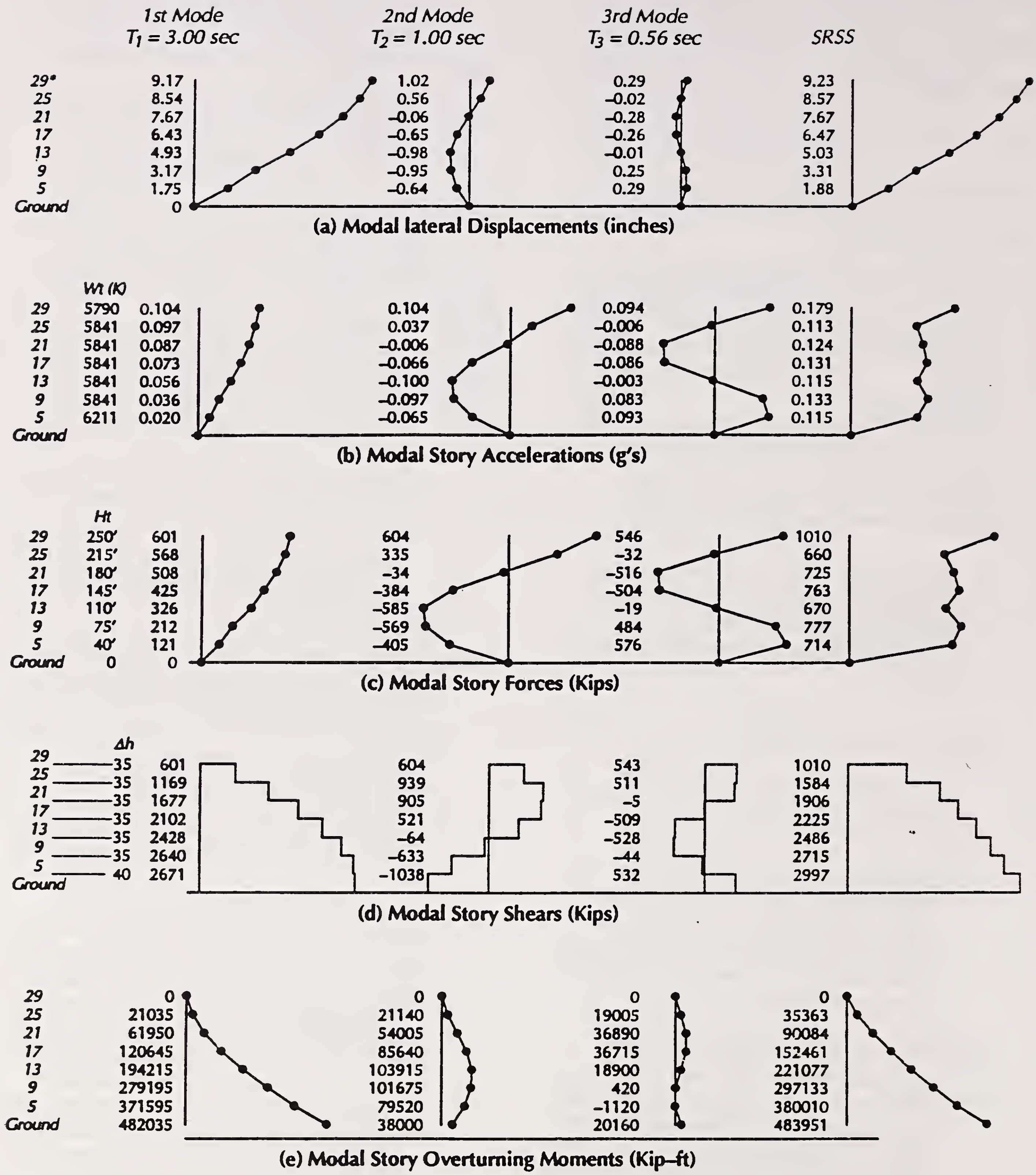
If the mode shapes are reasonably accurate, the calculated value of T_m will be the same at each story.

9. If the calculated periods of vibration are substantially different than the values assumed in paragraphs (1) and (2), above, repeat paragraph (5) and adjust the modal forces and displacements in proportion to the new values of S_a.

10. Compare the responses of the higher modes of vibration to the actions of the fundamental (e.g., refer to Figure F-8 and design example E-1). This includes story shears, story accelerations (i.e., story force divided by story weight), story overturning moments, and interstory displacements. If all the higher-mode responses are small relative to the fundamental mode, they can generally be omitted from the analysis. If in no case the square-root-of-the-sum-of-the-squares (SRSS) of all the modes is more than 10% greater than the fundamental mode, it can be assumed that the higher modes are negligible in the overall design.

D. *High-rise buildings*. As buildings get taller, the higher modes of vibration become more significant, relative to the fundamental modes (See Figures F-8 and F-9 for examples). These buildings generally require the use of computer programs that can calculate the dynamic characteristics (e.g., periods, mode shapes, and participation factors), as well as the member stresses and story displacements.

E. *Irregular buildings*. Buildings that have vertical discontinuities, that are irregular in plan, that have large horizontal eccentricities (center of mass significantly distant from the estimated center of rigidity), or have other irregularities will generally require the aid of computer programs to determine the dynamic characteristics, member stresses, and story displacements. When horizontal eccentricities exist, the analysis must be in three dimensions to account for the twisting deformations and the lateral deformations normal to the direction of the seismic forces. Refer to Section F.7 below, for use of three-dimensional computer programs.



- * Story 29 represents the roof, floors 29 and 28, and one-half of floor 27. Other story designations represent the reference story plus one-and-one-half stories above and below.

Figure F-9 *Sample modal analysis of a thirty-story building.*

F.6 Mathematical Modeling of Structural Components

The basic assumptions and considerations for pre-yield analyses are given below. (For post-yield analyses they may be modified as discussed in Section F.3.) The results of a lateral-force analysis can be very sensitive to the assumptions made for the stiffness of the structural elements when constructing a mathematical model of the structure. As the stiffness is overestimated, the period of vibration shortens and the displacements reduce. However, a shorter period may possibly attract higher forces. When the stiffness is underestimated, periods lengthen, lateral displacements increase, and lateral forces may be reduced. When the relative rigidities of various lateral-force-resisting elements are not accurately modelled, the analyses of the 3-D structural model will not accurately predict the torsional characteristics of the structure. The effects of nonstructural elements, as well as structural elements not part of the lateral-force-resisting system, can have a significant effect on the response of the overall structure to earthquake ground motion. Therefore, it is important to account for possible inaccuracies in the mathematical model. When there are uncertainties, an attempt should be made to envelope the possibilities to assure good performance of the structure in case of an earthquake. The stiffness characteristics may vary with amplitude of lateral motion; thus the model used for a code design level analysis may vary from the model that represents the yield level capacity or the ultimate post-yield capacity. For an elastic analysis, the following factors should be considered:

A. Gross concrete section properties are considered appropriate for modeling the stiffness of reinforced concrete members.

B. The effects of column widths and beam depths on the rigidity of frames should be evaluated. This is particularly important for concrete frames or for steel frames with relatively deep members and short spans or low story heights.

C. The effects of the floor slab system acting compositely with the frame beams or girders should be considered. Although the composite action may have an insignificant effect in resisting negative moments, it provides a significant contribution to the effective beam moment of inertia for positive moments and increases the stiffness of the beams acting as members of a rigid frame. In most cases, the beams will be modeled as prismatic members and engineering judgment will be required to determine an effective portion of the floor system to be modeled compositely with the beams. This composite action is used in the model to calculate the dynamic characteristics, but should be reevaluated for member design to resist negative moments.

D. The effects of structural elements that are not included in the lateral-force-resisting system will be evaluated. This may include flat-slab and column systems and structural steel frames with standard connections. The effects of these elements on the stiffness of a building with shear walls or braced frames may properly be ignored, but they may have a significant effect on the stiffness of a building with a moment frame lateral-force-resisting system. In the latter case, the moment frames will be designed to resist 100% of the lateral forces, but the modeled stiffness of the frames will be adjusted to reflect the additional stiffness of the above elements, including any torsional effects due to asymmetry in the location of elements.

E. The effects of relatively rigid nonstructural elements, such as masonry partitions, will be evaluated. If the stiffness of these elements is significant as compared to the stiffness of the assumed lateral-force-resisting system, the analyses will probably indicate an unacceptable D/C ratio for these partitions. Isolation of the partitions from the structural system by means of expansion joints at the sides and top of the element should be included in the retrofit design.

F. Evaluate the effects of assumptions for modeling shear walls of various cross-sections, for example, the relative stiffnesses of an L-shaped wall and a wall that consists of a single plane, and also, the relative stiffness of a shear wall system and a moment frame system.

F.7 Computer Programs

F.7.1 Two-Dimensional Computer Programs

The analyst must be familiar with all of the features and limitations of computer programs used for the design and analysis of buildings. A two-dimensional computer program essentially places all the lateral-force-resisting structural frames and shear walls within a single vertical plane and analyzes for lateral motion within that vertical plane. In a sense, all of the lateral-force-resisting column lines of the building are linked end-to-end. The two-dimensional analysis does not allow for any rotation about a vertical axis of the building (i.e., ignores horizontal torsion) and does not allow lateral sidesway normal to the direction of the applied force. The two-dimensional computer programs are applicable to buildings that are generally symmetrical in plan and are not subject to torsional deformation.

A. *Features and limitations.* There are a variety of two-dimensional computer programs, each having certain features and limitations, such as the following:

 1. *Dynamic characteristics.* Some computer programs will calculate member forces and joint displacements but do not calculate the periods or mode shapes of the structure. These programs can be used for low- to moderate-rise buildings where only the fundamental mode of vibration is required. The fundamental period and mode shape can be calculated and the effects of higher modes can be approximated by procedures outlined in Section F.5.3. However, computer programs are available that will calculate periods and mode shapes for all the modes of vibration.

 2. *Axial, shear, and flexural deformations.* Some computer programs are limited on the degrees of element deformations. Beams are generally considered as flexural elements. Some computer programs also account for shear deformation. Shear and flexural deformations are generally accounted for in column elements, but not all programs account for axial deformation. Axial column deformation can be significant in high-rise buildings; however, caution must be used when it is applied to gravity loads because of the sequence of construction. Shear walls are generally analyzed for shear and flexural deformations.

B. *Number of modes and use of participation factors.* In general, the first three modes of vibration in each horizontal direction of a building are sufficient for the modal analysis. For tall buildings or for buildings with vertical irregularities, a greater number of modes may have to be analyzed. A review of the participation factors for the first three modes will give a good indication if more are required. The sum of the participation factors (PF_{xm}) for all the modes at a particular story (x), as calculated from Formula F-10, equals unity. Also, the sum of all the modal base shear participation factors, (α), as defined in Formula F-11, will equal unity. Therefore, if the sum of the participation factors for the first three modes is within 10% of unity, it can generally be assumed that all the major modes have been included. For an example, refer to Table F-6. The sum of the participation factors at the roof for three modes equals 1.08 (i.e., 1.31 – 0.47 + 0.24) and the sum of the base shear participation factors is equal to 0.986 (i.e., 0.828 + 0.120 + 0.038). Both 1.08 and 0.986 are within 10% of the value of 1.0.

C. *Check static equilibrium.* Some computer programs present the results for each individual mode and others only present the results in modal combinations. Once the modes have been combined, it is not possible to check the statics for the overall building or for localized areas, such as at a beam-column joint. Therefore, static checks must be made prior to making the modal combination. Spot checks at a variety of locations should always be made to assure that static equilibrium is maintained. These checks are made not only to confirm the validity of the computer program, but also to alert the designer to possible irregularities or to the possibility of data input errors.

D. *Torsion.* The two-dimensional computer analyses do not account for torsional motion due to horizontal eccentricities. However, the effects of horizontal eccentricities or the requirement for accidental torsion can be approximated by hand calculations in conjunction with the results obtained from the computer analysis. The horizontal torsional moment can be calculated from the product of the story shear and the assumed eccentricity. The torsional moment can then be distributed to the lateral-force-resisting elements in proportion to the product of their relative rigidities and distances from the center of rotation (*Kd*) divided by the torsional moment of inertia (ΣKd^2). The forces obtained from the computer can then be proportioned upward to account for the additional forces due to torsion. The minimum torsional eccentricity should be equal to 5% of the maximum building dimension. A rational alternative to this requirement is to calculate accidental torsions by using eccentricities that result by moving the center of mass of each story 5% of the maximum building dimension to either side of its calculated position. An example is included in the SDG (U.S. Army, 1986) Design Example E-2.

E. *Flexible horizontal diaphragms*. Two-dimensional computer programs assume that the diaphragms are infinitely rigid. In some buildings, the horizontal diaphragms may exhibit some flexibility relative to the vertical lateral-force-resisting elements. For very flexible diaphragms, the forces should be distributed to the vertical lateral-force-resisting elements by means of tributary areas. When a limited amount of flexibility is anticipated, the forces on the less rigid elements of the rigid diaphragm model should be increased to account for possible additional forces due to tributary area distribution. Some judgment decisions are required. When there is difficulty in determining the proper distribution of forces, a three-dimensional analysis that accounts for diaphragm flexibility may be required.

F.7.2 Three-Dimensional Computer Programs

Three-dimensional computer programs become much more complex than the two-dimensional programs, and more care must be taken to fully understand their features and limitations. Three-dimensional programs can account for rotation about a vertical axis and horizontal movement in any direction. Some programs, usually those using finite element procedures, can allow for flexibility in the horizontal diaphragm. Section F.7.1 in general also applies to three-dimensional programs. Additional comments, which apply to three-dimensional programs, follow:

A. *Features and limitations*. There are a variety of three-dimensional computer programs, each having certain features and limitations, such as the following:

1. *Three-dimensional compatibility*. Some three-dimensional computer programs were developed as extensions of two-dimensional programs. The three-dimensional features are determined by combining the components of two-dimensional analyses. In some cases, where a structural element is part of both a transverse and longitudinal lateral-force-resisting system, compatibility of common actions from both directions of force is not maintained (e.g., axial forces and vertical deformations in a column common to two intersecting systems are not truly compatible).

2. *Horizontal eccentricities*. Additional care must be taken in preparing the data for three-dimensional computer programs. Torsional characteristics of a building are sensitive to the size and location of the story weights and the rigidity properties of lateral-force-resisting elements on the horizontal story plane. In some computer programs, mass moments of inertia are required. In other programs, the masses are distributed on the horizontal planes. Assumptions used in modeling a variety of shear walls and frames can be critical in the evaluation of torsional properties and horizontal eccentricities; therefore, methods to envelope the uncertainties should be investigated. (See the discussion of modeling in Section F.6 and torsion in F.7.1 Paragraph D.)

3. *Modal combinations*. Because the computer programs allow for three degrees of freedom (longitudinal, transverse, and rotational), combining the modes in three-dimensional analysis becomes substantially more complex than combining modes for two-dimensional analysis. In some cases, the use of the square root of the sum of the squares (SRSS) can give erroneous results, especially when the loads are applied in a direction not parallel to the major axes. Therefore, other procedures for combining the modes are required. The analyst must be aware of the procedures and pitfalls that may be inherent in the computer

program being used in relation to the building being analyzed.

B. *Modes and participation factors*

1. *Mode identification*. In three-dimensional analyses, it is sometimes difficult to identify the characteristics of the various modes of vibration. For a regular building, the first three modes will generally include the fundamental modes that represent primary motion in the translational transverse direction of the building, the translational longitudinal direction of the building, and the rotational torsional action of the building. The first nine modes listed in order of decreasing lengths of period will generally include the first three modes of each of those directional motions. However, for unusual buildings, the sequence of the modes may be highly irregular. For example, a building with very low torsional rigidity will have torsional modes with long periods of vibration; thus the translational modes may not be identified until after several torsional modes are calculated. Another example is in buildings with flexible diaphragms. If the diaphragms are more flexible than the overall structure, the modes for each of the flexible diaphragms will be calculated before the primary building modes are identified. Each of these examples would indicate that the building may have some undesirable characteristics or that there may be an error in the modeling of the building. Modes can be identified by plotting the mode shapes in three-dimensional representations.

2. *Participation factors*. The concept of participation factors also becomes more difficult to interpret in three-dimensional analyses; therefore, the guidelines given in Section F.7.1, paragraph B to identify the number of modes required for analysis may not be applicable for buildings with unusual three-dimensional characteristics. For each direction of applied earthquake forces there will be a major component in the direction of motion, a translational component normal to the direction of applied forces, and a rotational component. The participation factors, based on the mode shapes (φ) in the direction of applied motion, will not add up to 1.0, as occurs in the two-dimensional programs, because of the contribution of the other components of motion. If the base shear participation factors (α) do not add up to within 90% of unity, then all of the values of the modal analysis will be increased proportionately to satisfy the 90% requirement.

F.8 Unreinforced Masonry Bearing Wall Buildings

F.8.1 Characteristic Behavior

Certain buildings with bearing walls of unreinforced masonry and floors of wood have a unique response to earthquake ground motions. The walls are relatively heavy and rigid and the floors are relatively light and flexible. The shear walls and the floor are not fully coupled: the walls impart the ground motion to the ends of the diaphragms, and the responses of the shear walls and the diaphragms are quite different. Under certain conditions the diaphragms may yield in shear, and the piers of shear walls with openings may rock rather than bend. Both the yielding and rocking may have significant effects on the response. If the walls are adequately connected to the roof and floors, the behavior described here may take place, and the performance in an earthquake may be better than might be expected. Current building codes, having been written with other types of buildings in mind, do not recognize this behavior.

F.8.2 The Methodology

The methodology as originally developed for these buildings used a post-yield approach: the building was considered at its limit of stability. However, for application in the Los Angeles hazard-reduction program the methodology was converted to a working-stress level: the earthquake demands were reduced and the allowable stresses were established at a fraction of the ultimate values. Because the working-stress level has been retained in subsequent versions of the methodology, and because the provisions are prescriptive, the post-yield basis is not apparent.

For help in understanding the methodology, the engineer is referred to a discussion of the methodology and a list of pertinent references in the Draft Commentary published by the California Seismic Safety Commission (SSC, 1990). The engineer may also consult the final SEAOC Commentary when it becomes available.

F.8.3 UCBC Evaluation Procedure

The UCBC provisions (ICBO, 1991b) involve the following evaluation procedures: the anchorage of masonry walls is checked for its ability to keep the walls from falling away from the rest of the building under out-of-plane forces; the stability of the walls under out-of-plane forces and the possible need for lateral bracing are checked by reference to height/thickness ratios; the diaphragm displacement is checked by a procedure which involves calculation of a demand/capacity ratio (DCR) and reference to a chart of DCR vs. diaphragm span; depending on DCR-span conditions, the allowable height/thickness ratios may be increased where cross walls are effective; the stability of walls for in-plane forces is checked by a procedure that allows rocking of piers when they are strong enough and have enough superimposed load.

F.8.4 Limitation on the UCBC Procedure

The engineer is encouraged to consider the use of the methodology, for it recognizes the dynamic characteristics and the ultimate behavior of the building type. When the conventional static code approach is used on these buildings, the result is often a brute-force strengthening that is inappropriate. For example, adding plywood to resist large diaphragm shears tends to make the diaphragm stiffer and prevent the shear yielding that is an essential part of the desired response. However, several considerations call for caution in the use of the procedure:

1. The UCBC provisions (ICBO, 1991b) are subject to revision. The first set of revisions was proposed even before the document was officially accepted by ICBO. Further revisions will be proposed as engineers gain experience with the provisions.

2. The provisions are prescriptive: the evaluating engineer should review the procedure and be satisfied that it is applicable to the building and that it will provide a level of safety equivalent to that of the conventional procedure.

3. The procedure is not applicable in all cases, but the developers and reviewers of the provisions have been unsuccessful in developing a satisfactory set of limitations that would prevent misapplication.

4. The engineer should keep in mind the fact that retrofitting changes a building, making it different than it was.