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Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards

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Prepared for

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Foreword

Guidelines presented herein were developed under the auspices of Lawrence Livermore National Laboratory (LLNL) under contract to the Assistant Secretary for Environment, Safety, and Health, Office of Safety Appraisals of the U.S. Department of Energy (DOE). They were prepared by the DOE Natural Phenomena Hazards Panel. The DOE Project Manager was Mr. James R. Hill. Dr. Robert C. Murray was the LLNL Project Manager.

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Abstract

The Department of Energy (DOE) and the DOE Natural Phenomena Hazards Panel have developed uniform design and evaluation guidelines for protection against natural phenomena hazards at DOE sites throughout the United States (UCRL-15910). The goal of the guidelines is to assure that DOE facilities can withstand the effects of natural phenomena such as earthquakes, extreme winds, tornadoes, and flooding. The guidelines apply to both new facilities (design) and existing facilities (evaluation, modification, and upgrading). The intended audience is primarily the civil/structural or mechanical engineers conducting the design or evaluation of DOE facilities.

DOE Order 6430.1A, *General Design Criteria Manual*, was revised in 1989. This current version of Order 6430.1A references these guidelines (UCRL-15910) as an acceptable approach for design evaluation of DOE facilities for the effects of natural phenomena hazards. UCRL-15910 provides earthquake ground acceleration, wind speed, tornado wind speed and other effects, and flood level corresponding to the design basis earthquake (DBE), design basis wind (DBW), design basis tornado (DBT), and design basis flood (DBFL) as described in Order 6430.1A. Integrated with these natural phenomena loadings, UCRL-15910 provides recommended response evaluation methods and acceptance criteria in order to achieve acceptably low probabilities of facility damage due to natural phenomena.

The design and evaluation guidelines presented here control the level of conservatism introduced in the design/evaluation process such that earthquake, wind, and flood hazards are treated on a reasonably consistent and uniform basis. These guidelines also seek to ensure that the level of conservatism in design/evaluation is appropriate for facility characteristics such as importance, cost, and hazards to people on and off site and to the environment. For each natural phenomena hazard covered, these guidelines generally consist of the following:

1. Usage categories and performance goals.
2. Hazard probability from which natural phenomena hazard loading on structures, equipment, and systems is developed.
3. Recommended design and evaluation procedures to evaluate response to hazard loads and criteria to assess whether or not computed response is permissible.

Performance goals are expressed as the annual probability of exceedance of some level of facility damage due to natural phenomena. The appropriate performance goal for a facility is dependent on facility characteristics such as mission dependence, cost, and hazardous functions. Different guidelines are provided for four usage categories each with a specified performance goal. Usage categories and performance goals range from general use (normal building code provisions) to highly hazardous use (approaching nuclear power plant provisions).

The likelihood of occurrence of natural phenomena hazards at each DOE site has been evaluated by the DOE Natural Phenomena Hazard Program. Probabilistic hazard models are available for earthquake, extreme wind/tornado, and flood. Alternatively, site organizations are encouraged to develop site-specific hazard models utilizing the most

recent information and techniques available. In any case, to achieve a specified performance goal, hazard annual probabilities of exceedance are specified with design and evaluation procedures that provide a consistent and appropriate level of conservatism.

In this document, performance goals and natural hazard levels are expressed in probabilistic terms, and design and evaluation procedures are presented in deterministic terms. Design/evaluation procedures conform closely to common standard practices so that the procedures will be easily understood by most engineers. Performance goals are expressed in terms of structure or equipment damage to the extent that: (1) the facility cannot function; (2) the facility would need to be replaced; or (3) personnel are endangered.

The guidelines presented in this document contain information needed for the first two steps in a natural phenomena risk assessment: characterization of the hazard and procedures for structural evaluation. The remaining steps in estimating risk extend to consequences beyond the levels of facility damage (e.g., off-site release of hazardous materials, general public safety, or environmental damage). These remaining steps require a systems evaluation of the facility and surrounding area and are beyond the scope of this document.

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1 Introduction

1.1 Overview of the DOE Natural Phenomena Hazards Project

Lawrence Livermore National Laboratory (LLNL), under contract to the Assistant Secretary for Environment, Safety and Health, Office of Safety Appraisals (OSA) of the U.S. Department of Energy (DOE), has developed uniform design and evaluation guidelines for protection against natural phenomena hazards for facilities at DOE sites throughout the United States. This work provides guidance and criteria for design of new facilities and for evaluation, modification, or upgrade of existing facilities so that DOE facilities safely withstand the effects of natural phenomena (earthquakes, extreme winds, and flooding). This goal is being achieved by the Natural Phenomena Hazards Program, illustrated in Figure 1-1.

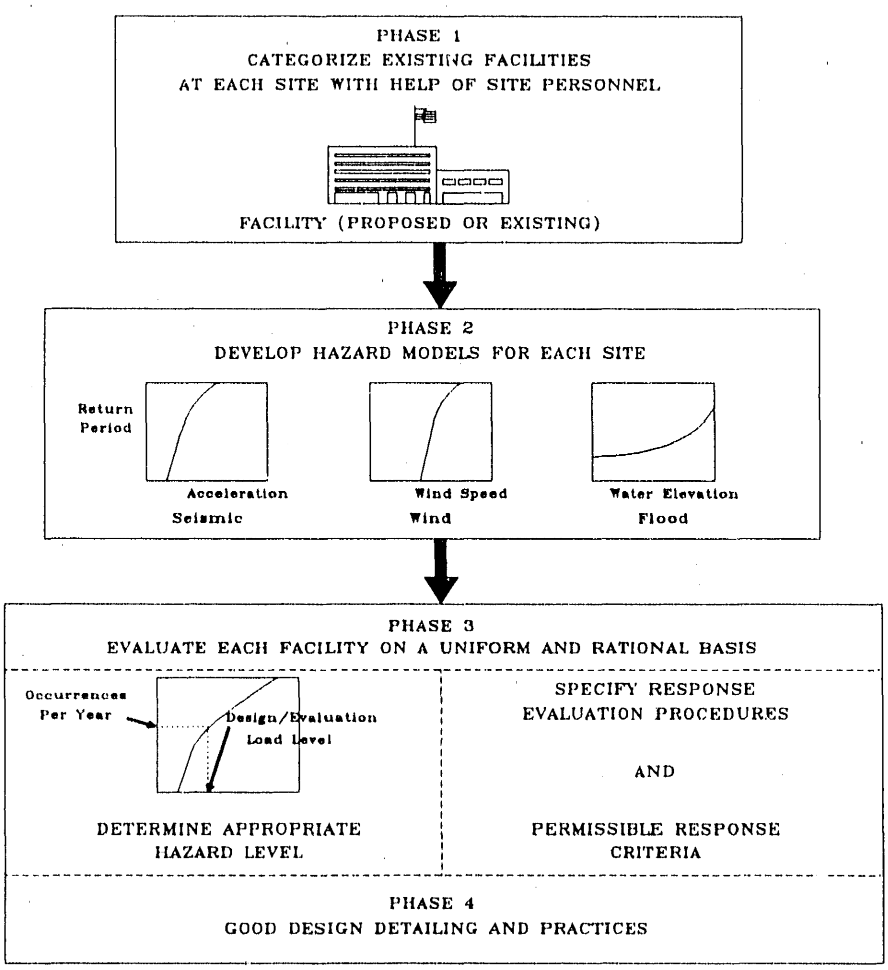


Figure 1-1. Flow Diagram of the Natural Phenomena Hazards Project

The Program consists of the following phases:

1. Selecting specific DOE sites to be included in the project and identifying existing critical facilities at each site.
2. Evaluating the likelihood of natural phenomena hazards at DOE sites. Phase 2 developed hazard models for earthquake, extreme wind/tornado, and flood for each DOE site.
3. Preparing design and evaluation guidelines that utilize information on the likelihood of natural phenomena hazards for the design of new facilities and the evaluation, modification, or upgrade of existing facilities.
4. Preparing manuals describing and illustrating good design or upgrading practice for structures, equipment, piping, etc. for earthquake and wi..d/tornado loadings. Also, conducting supporting studies on specific problem areas related to the mitigation of natural phenomena hazards.

Several phases of the Prog, am have been completed. The first phase - selecting DOE sites and identifying critical facilities - was completed many years ago. The development of probabilistic descriptions of earthquake and wind hazards at 25 DOE sites across the country has also been completed. The seismic hazard definitions have been published in LLNL report UCRL-53582, Rev.1 (Ref. 1). The wind/tornado hazard definitions have been published in LLNL report UCRL-53526, Rev.1 (Ref. 2). Note that seismic hazard estimates have been changing rapidly since Reference 1 was completed in 1984. A number of ongoing studies, not currently available, will provide the basis for upgrading Reference 1 in the future. However, Reference 1 represents the best currently available information on seismic hazard at all DOE sites.

Ongoing flood screening evaluations establish which sites have a potential flood hazard. As part of the screening analysis, preliminary probabilistic flood hazard descriptions are developed. To date, flood evaluations have been completed for the eight Albuquerque Operations Office sites and for the Richland Operations Office site. The results were published in LLNL report UCRL-53851 (Ref. 3). From screening analysis results, flooding can be eliminated for some sites as a design consideration. For those sites in which flooding is a significant design consideration, probabilistic definitions of the flood hazard will be refined by additional investigation.

The design and evaluation guidelines presented in this document are derived from the third phase of this project. These guidelines, together with the manuals on structural details and studies on problem areas, should enable DOE and site personnel to design or evaluate facilities on a uniform and rational basis for the effects of natural phenomena hazards. Phase 4 activities include: (1) a wind design practice manual (Ref. 4); (2) preparation of a seismic design practice manual is now being planned; (3) supporting studies have been published on seismic bracing of suspended ceilings (Ref. 5) and on seismic upgrade and strengthening guidelines for equipment (Ref. 6).

1.2 Overview of the Design and Evaluation Guidelines

The guidelines presented in this document provide relatively straightforward procedures to evaluate, modify, or upgrade existing facilities or to design new facilities for the effects of natural phenomena hazards. The intent is to control the level of conservatism in the design/evaluation process such that: (1) earthquake, wind, and flood hazards are treated consistently; and (2) the level of conservatism is appropriate for facility characteristics such as importance, cost, and hazards to people and the environment.

The guidelines include:

1. Usage categories and performance goals for each category.
2. For each category, hazard probability from which hazard loads are developed.
3. For each category, recommended design and evaluation procedures to evaluate facility response to hazard loads.
4. For each category, criteria to assess whether or not computed response is permissible.

Note that these guidelines do not cover practice and procedures for facility design or upgrading detailing; these matters will be covered by separate documents.

The first step in constructing these guidelines is to establish performance goals. These are expressed as the annual probability of exceedance of some level of facility damage due to natural phenomena hazards. The appropriate performance goal for a facility is dependent on facility characteristics such as mission dependence, cost, and hazardous functions. Four usage categories ranging from general use to highly hazardous use have been defined, along with a corresponding performance goal. Performance goal probability levels for each category are consistent with current design practice for both general use and high-hazard use facilities.

To achieve a performance goal, hazard annual probabilities of exceedance are specified along with design and evaluation procedures, with a consistent level of conservatism. Performance goals and hazard levels are expressed in probabilistic terms; design and evaluation procedures are presented deterministically. Design/evaluation procedures recommended in this document conform closely to common standard practices so that most engineers will readily understand them. The intended audience for these guidelines is the civil/structural or mechanical engineer conducting the design or evaluation of facilities. These guidelines do not preclude the use of probabilistic or alternative approaches if these approaches meet the specified performance goals.

The framework under which these guidelines have been developed allows for their use in an overall risk assessment, as shown in Figure 1-2.

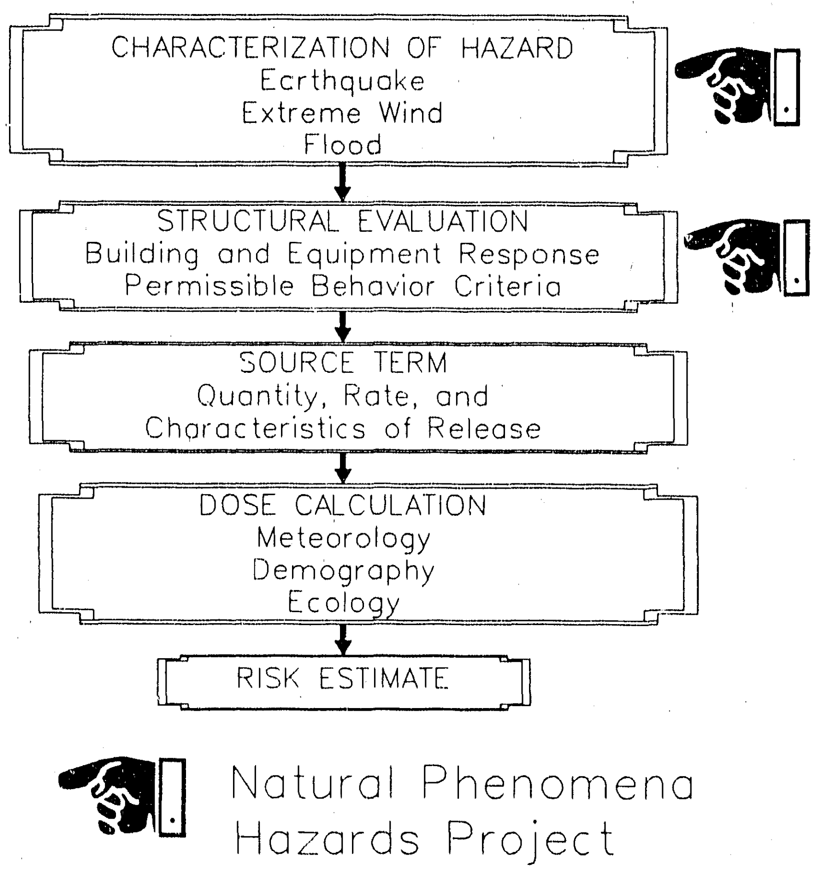


Figure 1-2. Flow Diagram For Assessment Of Risk From Natural Phenomena Hazards

These guidelines contain information needed for the first two steps in a natural phenomena risk assessment: (1) characterization of the hazard and (2) procedures for structural analysis. The remaining steps in estimating risk are not covered in this document. Performance goals are expressed in terms of structure or equipment damage to the extent that: (1) the facility cannot function; (2) the facility would need to be replaced; or (3) personnel are endangered. The performance goals in this document do not refer to the consequences of structure or equipment damage beyond those just described. For example, this document does not attempt to set performance goals in terms of off-site release of hazardous materials, general public safety, or environmental damage.

Existing criteria for the design and evaluation of DOE facilities are provided by the General Design Criteria Manual, DOE Order 6430.1A (Ref. 7). DOE Order 6430.1A has recently been revised, and material from these guidelines are referenced by the revised Order as an acceptable approach for the design or evaluation of DOE facilities for the effects of natural phenomena hazards. DOE 6430.1A requires that facilities be designed for design basis events including natural phenomena hazards, fire, accidents, etc. Design basis events due to natural phenomena hazards, as defined in 6430.1A, include earthquakes (DBE), winds (DBW), tornadoes (DBT), and floods (DBFL). This document (UCRL-15910) provides earthquake ground acceleration, wind speeds, tornado wind speeds and other effects, and flood levels corresponding to these events for usage in design and evaluation of facilities. UCRL-15910 is an integrated approach combining definition of loading due to natural phenomena, response evaluation methods, and acceptance criteria.

The remainder of this chapter defines some of the terminology used in this report and briefly describes the seismic, wind, and flood hazard information from References 1, 2, and 3. Chapter 2 covers aspects of these design and evaluation guidelines common to all natural phenomena hazards. In particular, usage categories and performance goals are discussed in Chapter 2. Chapter 3 provides general discussion of the effects of natural phenomena hazards on facilities. Specific design and evaluation guidelines for earthquakes, extreme winds, and floods are presented in Chapters 4, 5, and 6, respectively. In particular, these three chapters discuss recommended hazard probabilities, as well as design and evaluation procedures for response evaluation and permissible behavior criteria. Summaries of the guidelines for earthquake, wind, and flood may be found in Tables 4-2, 5-2, and 6-1, respectively.

1.3 Terminology and Definitions

Hazard - The term "hazard" is defined as a source of danger. In this report, natural phenomena such as earthquakes, extreme winds, and floods are hazards to the buildings, equipment, piping, and other structures making up DOE facilities. Toxic or radioactive materials contained within facilities are also hazards to the population or environment in the vicinity of DOE facilities. Throughout this report, the term "hazard" is used to mean both the external sources of danger (such as potential earthquakes, extreme winds, or floods) and internal sources of danger (such as toxic or radioactive materials).

Annual Probability of Exceedance - The likelihood of natural phenomena hazards has been evaluated on a probabilistic basis in References 1, 2, and 3. The frequency of occurrence of parameters describing the external hazard severity (such as maximum earthquake ground acceleration, maximum wind speed, or maximum depth of inundation) is estimated by probabilistic methods. Common frequency statistics employed for rare events such as natural phenomena hazards include return period and annual probability of exceedance. Return period is the average time between consecutive events of the same or greater severity (for example, earthquakes with maximum ground acceleration of 0.2g or greater). It must be emphasized that the return period is only an average duration between events and should not be construed as the actual time between occurrences, which would be highly variable. If a given event of return period, T, is equally likely to occur any year, the probability of that event being exceeded in any one year is approximately 1/T. The annual probability of exceedance, p, of an event is the reciprocal of the return period of that event. As an example, consider a site at which the return period for an earthquake of 0.2g or greater is 1000 years. In this case, the annual probability of exceedance of 0.2g is 10^{-3} or 0.1 percent.

Exceedance Probability for a Given Number of Years - It is of interest in the design of facilities to define the probability that an event will be exceeded during the design life of the facilities. For an event with return period, T, and annual probability of exceedance, p, the exceedance probability, EP, over design life, n, is given by:

$$EP = 1-(1-p)^n = 1-(1-1/T)^n = 1-e^{-n/T} \qquad (1\text{-}1)$$

where EP and p are expressed as fractions of unity and n and T are expressed in years. As an example, the exceedance probabilities over a design life of 50 years of a given event with various annual probabilities of exceedance are as follows:

p	EP over 50 years
10^{-2}	0.39
10^{-3}	0.05
10^{-4}	0.005
10^{-5}	0.0005

Hence, an event with a 10^{-2} annual probability of exceedance (100 year return period) has a 39 percent chance of being exceeded in a 50-year period, while an event with a 10^{-4} annual probability of exceedance has only a 0.5 percent chance of being exceeded during a 50-year period.

Hazard Curves - In References 1, 2 and 3, the likelihood of earthquake, wind, and flood hazards at DOE sites has been defined by graphical relationships between maximum ground acceleration, maximum wind speed, or maximum water elevation and return period (reciprocal of annual probability of exceedance). These relationships are termed seismic, wind or flood hazard curves. The earthquake or wind loads or the flood levels used for the design or evaluation of DOE facilities are based on hazard parameters from these curves at selected annual probabilities of exceedance.

Performance Goals - The likelihood of adverse facility behavior during natural phenomena hazards can also be expressed on a probabilistic basis. Goals for facility performance during natural phenomena hazards have been selected and expressed in terms of annual probability of exceedance. As an example, if the performance goal is 10^{-3} annual probability of exceedance for structural damage, there would be less than about a 5 percent chance that such damage could occur over a 50-year design life. If the performance goal is 10^{-4} annual probability of exceedance for structural or equipment damage, there would be about a 0.5 percent chance of such damage over a 50-year design life. The level of damage considered in the performance goal depends on the facility characteristics. For example, the performance goal for general use facilities is major damage to the extent that occupants are endangered. However, the performance goal for hazardous use facilities is lesser damage to the extent that the facility cannot perform its function.

Confidence Level - Because of uncertainty in the underlying hazard process (e.g., earthquake mechanism for seismic hazard), performance goals or hazard probabilies can be specified at higher confidence levels to provide greater conservatism for more critical conditions.

1.4 Earthquake, Wind, and Flood Hazards for DOE Facilities

For the guidelines presented in this document, loads induced by natural phenomena hazards are based on external hazard parameters (e.g., maximum earthquake ground acceleration, maximum wind speed, and maximum depth of inundation) at specified annual probabilities of exceedance. As a result, probabilistic hazard curves are needed for each DOE facility. This information can be obtained from independent site-specific studies or from References 1, 2, and 3 for earthquake, wind, and flood hazards, respectively. The hazard information from these references is discussed throughout this report. In conjunction with the guidelines, the use of independent site-specific evaluations of natural phenomena hazards may also be used as the basis for loads on facilities.

Seismic and wind hazard curves have been evaluated by site-specific studies of the DOE sites considered (References 1 and 2). In addition, flood hazard curves have been evaluated for some of the DOE sites considered (Ref. 3). Flood hazard curves developed from screening studies are currently available for the eight Albuquerque Operations Office sites and for the Richland Operations Office site.

For earthquakes, Reference 1 presents best-estimate peak ground accelerations as a function of return period, as illustrated by Figure 1-3. Acceleration values correspond to the maximum acceleration that would be recorded by a three-axis strong-motion instrument on a small foundation pad at the free ground-surface. In addition, ground response spectra for each site are provided in Reference 1. Ground response spectra indicate the dynamic amplification of the earthquake ground motion during linear, elastic, seismic response of facilities. These spectra provide information about the frequency content of potential earthquake ground motion at the site.

In Reference 2, mean predicted maximum wind speeds as a function of return period and annual probability of exceedance are given, as illustrated by Figure 1-4 for the 25 DOE sites considered. At annual probabilities of exceedance where tornadoes govern the wind loading on facilities, Reference 2 also specifies tornado-related effects. These effects include atmospheric pressure change and windborne missiles, which must be considered. At annual probabilities of exceedance where straight winds govern the wind loadings, these tornado-related effects do not significantly affect facility behavior and need not be considered.

Reference 3 provides the results of flood hazard evaluation work performed to date for DOE sites. The results of this work are flood hazard curves in which peak water elevation is expressed as a function of return period and annual probability of exceedance, as shown in Figure 1-5. Note that the work performed thus far is the result of flood screening analyses and not detailed flood hazard studies, such as those conducted for seismic and wind hazards. The scope of the flood screening analysis is restricted to evaluating the flood hazards that may exist in proximity to a site. The analysis does not involve an assessment of the potential encroachment of flooding at individual facility locations. Furthermore, the screening analyses do not consider localized flooding at a site due to precipitation (e.g., local run-off, storm sewer capacity, roof drainage). The results of the flood screening analyses serve as the primary input to DOE site managers to review the impact of flood hazards on individual facilities and to evaluate the need for more detailed flood hazard assessment.

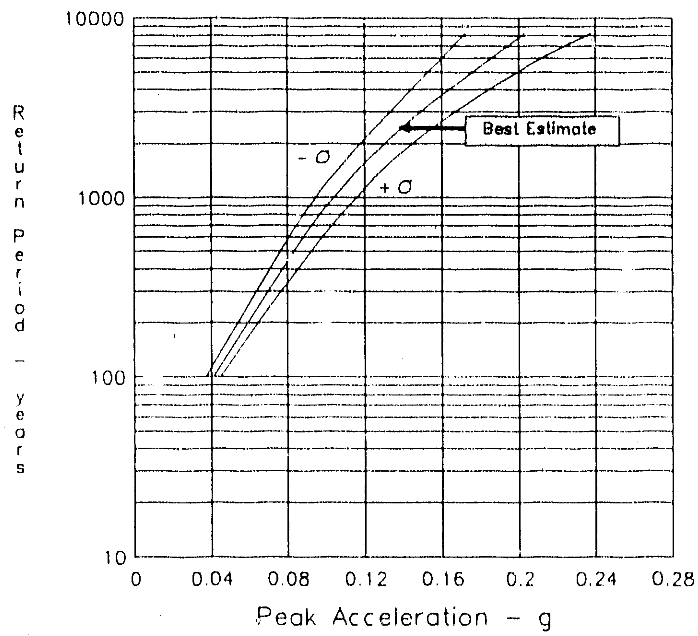


Figure 1-3. Example Seismic Hazard Curve

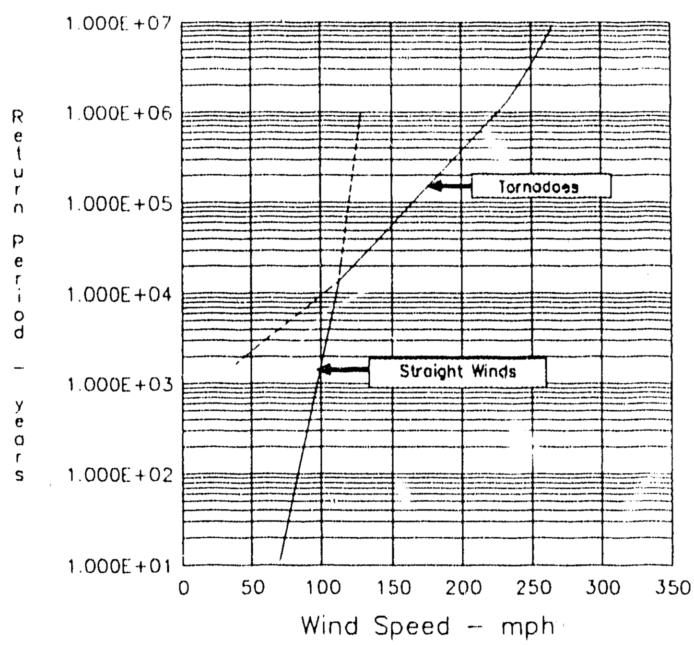


Figure 1-4. Example Wind/Tornado Hazard Curve

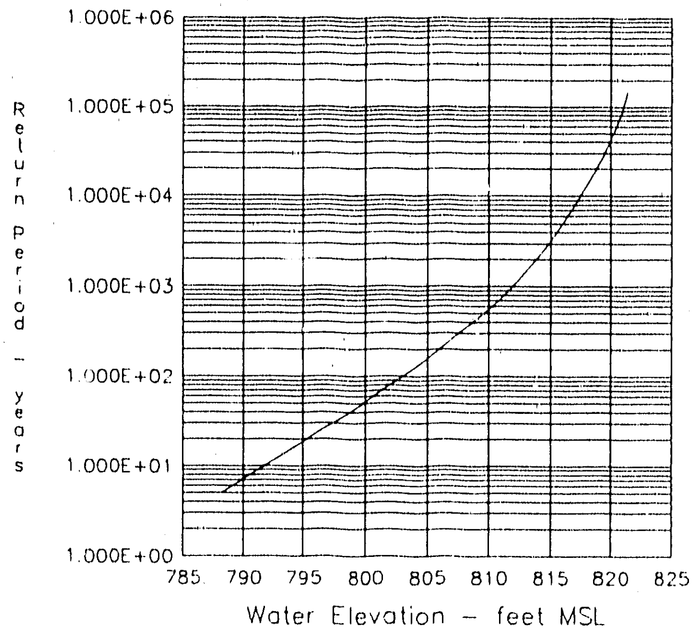


Figure 1-5. Example Flood Hazard Curve

2 General Design and Evaluation Guidelines

2.1 Design and Evaluation Philosophy

The guidelines presented in this document are intended to assure acceptable performance of DOE facilities in the event of earthquake, wind/tornado, and flood hazards. As discussed in Chapter 1, performance is measured by performance goals expressed as an annual probability of natural phenomena recurrence and resultant unacceptable damage. These annual probabilities of unacceptable damage are intended to be consistent with standard engineering practice for both normal use and hazardous use facilities. It must be emphasized that these performance goals correspond to probabilities of structure or equipment damage due to natural phenomena hazards; they do not extend to consequences beyond structure or equipment damage.

The responsibility for selecting performance goals rests with DOE management. Selection of performance goals for facilities subjected to natural phenomena hazards should be based on characteristics of the facility under consideration, including:

1. Vulnerability of occupants.
2. Cost of replacement of facility and contents.
3. Mission dependence or programmatic impact of the facility on operations at the DOE site.
4. Characteristics of hazardous materials contained within the facility, including quantity, physical state, and toxicity.
5. Factors affecting off-site release of hazardous materials, such as a high energy source or transport mechanism, as well as off-site land use and population distribution.

For example, a much higher likelihood of damage would be acceptable for an unoccupied storage building of low value than for a high-occupancy facility or a facility containing hazardous materials. Facilities containing hazardous materials which, in the event of damage, threaten public safety or the environment, and/or which are under close public scrutiny, should have a very low probability of damage due to natural phenomena hazards (i.e., much lower probability of damage than would exist from the use of conventional building code design and evaluation procedures). For ordinary facilities of relatively low cost, there is no reason to provide additional safety over that consistent with conventional building codes. Furthermore, it is probably not cost-effective to pay for additional resistance over that resulting from the use of conventional building codes that consider extreme loads due to natural phenomena hazards.

Because acceptable performance depends on facility characteristics, design and evaluation guidelines are provided for several different performance goals. To aid DOE management in the selection of appropriate performance goals, usage categories are described, each with different facility characteristics. These categories are sufficiently complete to allow assignment to a category of most DOE facilities. Category descriptions represent the understanding of the authors as to what types of facilities should be associated with different performance goals. These descriptions are offered as guidance to DOE management in performance goal selection for specific facilities. It is the responsibility of DOE management to decide what performance goals are appropriate for each portion of facilities under consideration.

The annual probability of exceedance of facility damage as a result of natural phenomena hazards (i.e., performance goal) is a combined function of the annual probability of exceedance of the event, factors of safety introduced by the design/evaluation procedures, and other sources of conservatism. These guidelines specify hazard annual probabilities of exceedance, response evaluation methods, and permissible behavior criteria for each natural phenomena hazard and for each usage category such that desired performance goals are achieved for either design or evaluation. The difference in the hazard annual probability of exceedance and the performance goal annual probability of exceedance establishes the level of conservatism to be employed in the design or evaluation process. For example, if the performance goal and hazard annual probabilities are the same, the design or evaluation approach should introduce no conservatism. However, if conservative design or evaluation approaches are employed, the hazard annual probability of exceedance can be larger (i.e., more frequent) than the performance goal annual probability. In the guidelines presented herein, the hazard probability and the conservatism in the design/evaluation method are not the same for earthquake, wind, and flood hazards. However, the accumulated effect of each step in the design/evaluation process should lead to reasonably consistent performance goals for each hazard.

Design and evaluation guidelines are presented in Chapters 4, 5, and 6 for earthquake, wind, and flood hazards, respectively. These guidelines are deterministic procedures that establish facility loadings from probabilistic hazard curves; recommend methods for evaluating facility response to these loadings; and provide criteria to judge whether computed facility response is acceptable. These guidelines are intended to apply equally to the design of new facilities and to the evaluation of existing facilities. In addition, the guidelines are intended to cover buildings, equipment, piping, and other structures.

The guidelines presented in this report cover (1) methods of establishing load levels on facilities from natural phenomena hazards and (2) methods of evaluating the behavior of structures and equipment to these load levels. These items are very important, and they are, typically, emphasized in design and evaluation criteria. However, there are other aspects of facility design that are equally important and that should be considered. These aspects include quality assurance considerations and attention to design details. Quality assurance requires peer review of design drawings and calculations; inspection of construction; and testing of material strengths, weld quality, etc. The peer reviewers should be qualified personnel who were not involved in the original design. Important design details include measures to assure ductile behavior and to provide redundant load paths, as well as proper anchorage of equipment and nonstructural building features. Although quality assurance and design details are not discussed in this report to the same extent as hazard load levels and response evaluation methods, the importance of these parts of the design/evaluation process should not be underestimated. Quality assurance and peer review are briefly addressed in Section 2.5, in addition to discussions in the individual chapters on each natural phenomena hazard. Design detailing for earthquake and wind hazards is covered by separate manuals currently being prepared or planned. Reference 4 gives structural details for wind design.

2.2 Performance Goals and Usage Categories

As stated previously, it is the responsibility of DOE management to select the appropriate performance goal for specific facilities. This may be accomplished by either of the following two approaches:

1. Place facilities or portions of facilities into usage categories based on characteristics such as mission dependence, occupancy, amount and type of hazardous materials involved, and distance to population centers.
2. Place facilities or portions of facilities into usage categories based on the associated performance goals as presented in this section and on an independent assessment of the appropriate performance goal for the facility.

Note that the categories are intended to provide general guidance for reasonable facility categorization and performance goals. DOE management may either accept the performance goals assigned to each category or else establish performance goals specifically for individual facilities or parts of facilities. In either case, the guidelines presented in this report may be utilized for design or evaluation.

2.2.1 Usage Categories

Four usage categories are suggested for design/evaluation of DOE facilities for natural phenomena hazards: (1) General Use, (2) Important or Low Hazard, (3) Moderate Hazard, and (4) High Hazard. These categories are defined in Table 2-1.

Table 2-1 Usage Category Guidelines

Usage Category	Description
General Use Facilities	Facilities that have a non-mission-dependent purpose, such as administration buildings, cafeterias, storage, maintenance and repair facilities which are plant- or grounds-oriented.
Important or Low Hazard Facilities	Facilities that have mission-dependent use (e.g., laboratories, production facilities, and computer centers) and emergency handling or hazard recovery facilities (e.g., hospitals, fire stations).
Moderate Hazard Facilities	Facilities where confinement of contents is necessary for public or employee protection. Examples would be uranium enrichment plants, or other facilities involving the handling or storage of significant quantities of radioactive or toxic materials.
High Hazard Facilities	Facilities where confinement of contents and public and environment protection are of paramount importance (e.g., facilities handling substantial quantities of in-process plutonium or fuel reprocessing facilities). Facilities in this category represent hazards with potential long-term and widespread effects.

The design and evaluation of General Use and Important or Low Hazard facilities would normally be governed by conventional building codes. The General Use category includes normal use facilities for which no extra conservatism against natural phenomena hazards is required beyond that in conventional building codes that include earthquake, wind, and flood considerations. Important or Low Hazard facilities are those where it is very important to maintain the capacity to function and to keep the facility operational in the event of natural phenomena hazards. Conventional building codes would treat hospitals, fire and police stations, and other emergency-handling facilities in a similar manner to the requirements of this document's guidelines for Important or Low Hazard facilities.

Moderate and High Hazard facilities handle significant amounts of hazardous materials. Damage to these facilities could potentially endanger worker and public safety and the environment. As a result, it is very important for these facilities to continue to function in the event of natural phenomena hazards, such that the hazardous materials may be controlled and confined. For both of these categories, there must be a very small likelihood of damage due to natural phenomena hazards. Guideline requirements for Moderate Hazard facilities are more conservative than requirements found in conventional building codes. Requirements for High Hazard facilities are even more conservative.

Factors distinguishing Moderate and High Hazard facilities are that the operations involving dangerous materials in High Hazard facilities pose a greater threat due to the potential for more widespread and/or long-term contamination in the event of off-site release. Examples of High Hazard operations are those involving large quantities of in-process radioactive or toxic materials that have a high energy source or transport mechanisms that facilitate off-site dispersion. High energy sources (such as high pressure and temperature steam or water associated with the operations of some facilities) can disperse hazardous materials widely. Radioactive material in liquid or powder form or toxic gases are easily transportable and may result in the facility being classified High Hazard. Hazardous materials in solid form or within storage canisters or casks may result in the same facility being classified Moderate Hazard. High Hazard facilities do not necessarily represent as great a potential hazard as commercial nuclear power plants, which are licensed by the Nuclear Regulatory Commission (NRC).

Table 2-2 illustrates that categories defined in these guidelines are comparable with facility categorization from other sources.

Table 2-2 Comparison of Usage Categories from Various Sources (Refs. 8-12)

Source	Facility Categorization			
UCRL-15910 - DOE Natural Phenomena Hazard Guidelines	General Use	Important or Low Hazard	Moderate Hazard	High Hazard
1988 Uniform Building Code	General Facilities	Essential Facilities	-	-
DOD Tri-Service Manual for Seismic Design of Essential Buildings	-	-	High Risk	Essential
IAEA-TECDOC-348 - Nuclear Facilities with Limited Radioactive Inventory	-	Class C	Class B	Class A
DOE 5481.1B SAR System	-	Low Hazard	Moderate Hazard	High Hazard
NFPA 13 (Classifications for Sprinkler Systems)	Light Hazard	Ordinary Hazard (Group 1)	Ordinary Hazard (Group 3)	Extra Hazard
Nuclear Regulatory Commission	-			*

* NRC licensed commercial nuclear power plants have slightly more conservative criteria than the criteria recommended for High Hazard facilities by these guidelines.

Designation of the usage category for a facility should be agreed upon among DOE and contractor line management, facility operations managers, designers, and safety analysts. DOE 5481.1B (Ref. 11) should be utilized for guidance in facility categorization.

2.2.2 Performance Goals

Table 2-3 presents performance goals for each usage category. The design and evaluation guidelines presented in this document for facilities subjected to natural phenomena hazards have been specified to meet these performance goals. The basis for selecting these performance goals and the associated annual probabilities of exceedance are described briefly in this section.

Table 2-3 Performance Goals for Each Usage Category

Usage Category	Performance Goal Description	Performance Goal Annual Probability of Exceedance
General Use	Maintain occupant safety	10^{-3} of the onset of major structural damage to the extent that occupants are endangered
Important or Low Hazard	Occupant safety, continued operation with minimal interruption	5×10^{-4} of facility damage to the extent that the facility cannot perform its function
Moderate Hazard	Occupant safety, continued function, hazard confinement	10^{-4} of facility damage to the extent that the facility cannot perform its function
High Hazard	Occupant safety, continued function, very high confidence of hazard confinement	10^{-5} of facility damage to the extent that the facility cannot perform its function

For *General Use facilities*, the primary concern is preventing major structural damage or facility collapse that would endanger personnel within the facility. A performance goal annual probability of exceedance of about 10^{-3} of the onset of significant facility damage is appropriate for this category. This performance is considered to be consistent with conventional building codes (Refs. 8, 13, and 14), at least for earthquake and wind considerations. The primary concern of conventional building codes is preventing major structural failure and maintaining life safety under major or severe earthquakes or winds. Repair or replacement of the facility or the ability of the facility to continue to function after the occurrence of the hazard is not considered.

Important or Low Hazard Use facilities are of greater importance due to mission-dependent considerations. In addition, these facilities may pose a greater danger to on-site personnel than general use facilities because of operations or materials within the facility. The performance goal is to maintain both capacity to function and occupant safety. Important or Low Hazard facilities should be allowed relatively minor structural damage in the event of natural phenomena hazards. This is damage that results in minimal interruption to facility operations and that can be easily and readily repaired following the event. A reasonable performance goal is judged to be an annual probability of exceedance of between 10^{-3} and 10^{-4} of structure/equipment damage, with the facility being able to function with minimal interruption. This performance goal is believed to be consistent with the design criteria for essential facilities (e.g., hospitals, fire and police stations, centers for emergency operations) in accordance with conventional building codes (Ref. 8).

Moderate or High Hazard Use facilities pose a potential hazard to public safety and the environment because radioactive or toxic materials are present. Design considerations for these categories are to limit facility damage so that hazardous materials can be controlled and confined, occupants are protected, and functioning of the facility is not interrupted. The performance goal for Moderate Hazard facilities is to limit damage such that confinement of hazardous materials is maintained. The performance goal for High Hazard facilities is to provide very high confidence that hazardous materials are confined both during and following natural phenomena occurrence. Maintaining confinement of hazardous materials requires that damage be limited in confinement barriers. Structural members and components should not be damaged to the extent that breach of the confinement or containment envelope is significant. Furthermore, ventilation filtering and containers of hazardous materials within the facility should not be damaged to the extent that they are not functional. In addition, confinement may depend on maintaining safety-related functions, so that monitoring and control equipment should remain operational following, and possibly during, the occurrence of severe earthquakes, winds, or floods.

For High Hazard facilities, a reasonable performance goal is an annual probability of exceedance of about 10^{-5} of damage beyond which hazardous material confinement is impaired. This performance goal approaches, at least for earthquake considerations, the performance goal for seismic-induced core damage associated with design of commercial nuclear power plants (Refs. 15, 16, 17, and 18). Annual frequencies of seismic core damage from published probabilistic risk assessments (PRA) of recent commercial nuclear plants have been summarized in Reference 19. This report indicates that mean seismic core damage frequencies ranged from 4×10^{-6}/year to 1×10^{-4}/year based on consideration of 12 plants. For 10 of the 12 plants, the annual seismic core damage frequency was greater than 1×10^{-5}. Hence, the High Hazard performance goal assigned in these guidelines is consistent with Reference 19 information. For Moderate Hazard facilities, an appropriate performance goal is an annual probability of exceedance of about 10^{-4} of damage beyond which hazardous material confinement is impaired.

2.3 Evaluation of Existing Facilities

Although these guidelines for natural phenomena hazards can be used for design of new facilities, they will probably be mostly applied to existing DOE facilities, because new design work may be infrequent. New facilities can be designed with these guidelines, but existing facilities may not meet the guidelines. For example, most facilities built a number of years ago in the eastern United States were designed without consideration of potential earthquake hazard. It is, therefore, likely that some older DOE facilities do not meet the earthquake guidelines presented in this document.

If an existing facility does not meet the natural phenomena hazard design/evaluation guidelines, several options (such as those illustrated by the flow diagram in Figure 2-1) need to be considered. Potential options for existing facilities include:

1. Conduct a more rigorous evaluation of facility behavior to reduce conservatism which may have been introduced by simple techniques used for initial facility evaluation. Alternatively, a probabilistic assessment of the facility might be undertaken in order to demonstrate that the performance goals for the facility can be met.
2. The facility may be strengthened to provide resistance to hazard effects that meets the guidelines.
3. The usage of the facility may be changed so that it falls within a less hazardous usage category and consequently, less stringent requirements.

If facility evaluation uncovers deficiencies or weaknesses that can be easily remedied, these should be upgraded without considering the other options. It is often more cost-effective to implement simple facility upgrades than to expend effort on further analytical studies.

If an existing facility is close to meeting the guidelines, a slight increase in the annual risk to natural phenomena hazards can be allowed because (1) existing facilities may have a shorter remaining life than a new facility and (2) it is far more difficult to upgrade an existing facility compared to incorporating increased resistance in a new design. As a result, some relief in the guidelines for earthquake and wind/tornado evaluations can be allowed by performing the evaluation using hazard exceedance probability of twice the recommended value. For example, if the hazard annual probability of exceedance for the facility under consideration was 10^{-4}, it would be acceptable to reconsider the facility at hazard annual probability of exceedance of 2×10^{-4}. This would have the effect of slightly reducing the seismic and wind loads in the facility evaluation. Relief in the guidelines is not permitted for flood evaluation because the performance of facilities during floods is very sensitive to the water elevation and a factor of two change in hazard exceedance probability may result in a significant change in water elevation. In addition, relief is not permitted for flood evaluation because the facility may not have margin to accomodate additional water similar to the way facilities have margin to accomodate increased earthquake acceleration or wind speed.

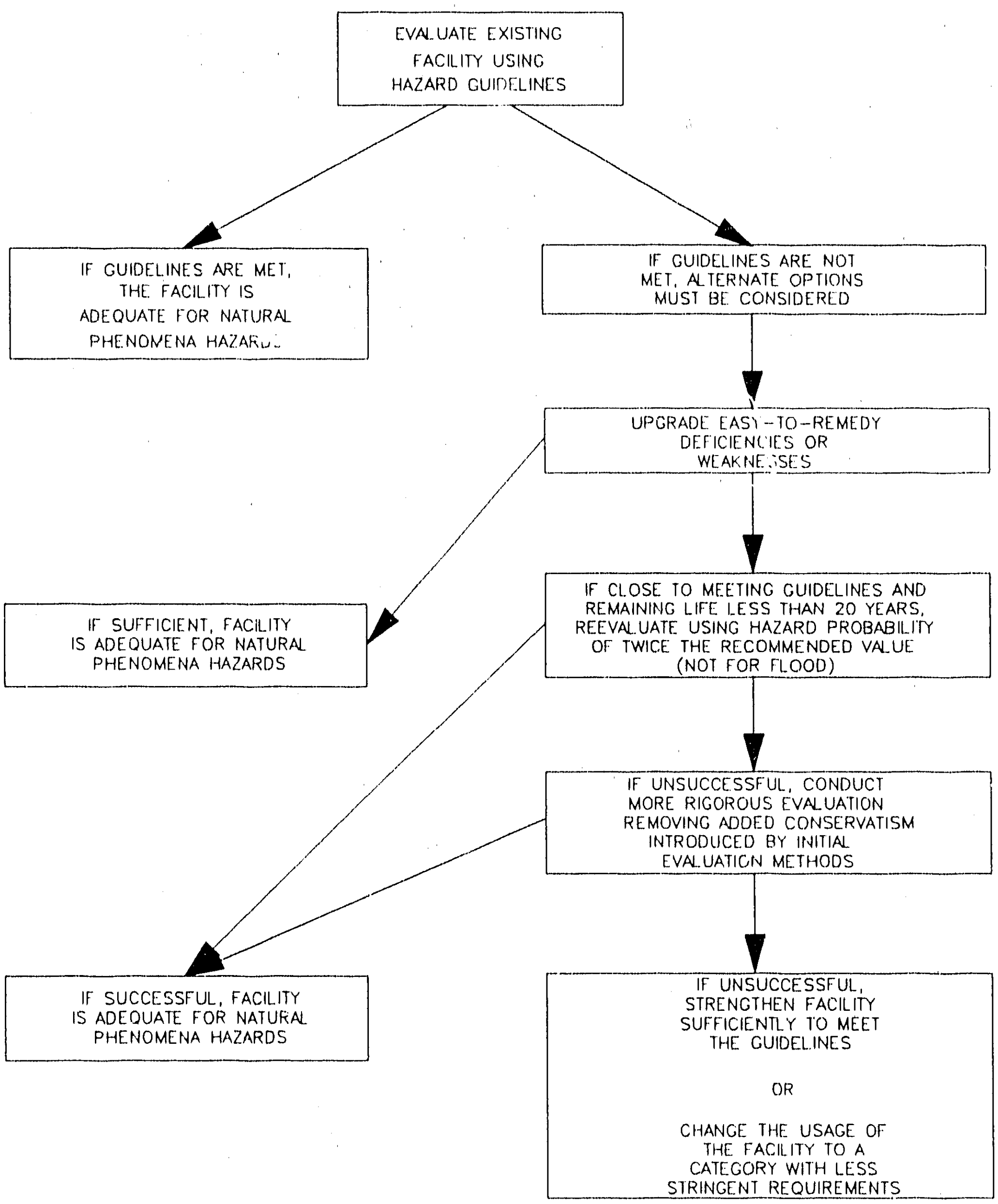


Figure 2-1 Evaluation Approach for an Existing Facility

For existing facilities, an assessment must be made for both the as-built and as-is condition. This assessment includes reviewing drawings and conducting site visits to determine deviations from the drawings and any in-service deterioration. In-place strength of the materials can be used when available. Corrosive action and other aging processes should be considered. Evaluation of existing facilities is similar to evaluations performed of new designs except that a single as-is configuration is evaluated instead of several configurations in an iterative manner, as required in the design process. Evaluations should be conducted in order of priority, with highest priority given to those areas identified as weak links by preliminary investigations and to areas that are most important to personnel safety and operations with hazardous materials.

2.4 Quality Assurance and Peer Review

It is recommended that important, hazardous (Important or Low Hazard, Moderate Hazard, and High Hazard categories) or unusual facilities be designed or evaluated utilizing an engineering quality assurance plan. Specific details about an engineering quality assurance plan for seismic design and evaluation are described in Chapter 4. Engineering quality assurance plans for wind or flood design and evaluation should be similar to the plan described in Chapter 4. General characteristic recommended for engineering quality assurance plans for design or evaluation for natural phenomena hazards are described below.

In general, an engineering quality assurance plan should include the following requirements. On the design drawings or evaluation calculations, the engineer must describe the hazard design basis including (1) description of the system resisting hazard effects and (2) definition of the hazard loading used for the design or evaluation. Design or evaluation calculations should be checked for numerical accuracy and for theory and assumptions. For new construction, the engineer should specify a program to test materials and inspect construction. In addition, the engineer should review all testing and inspection reports and visit the site periodically to observe compliance with plans and specifications. For important or hazardous facilities, all aspects of the design or evaluation must include independent peer review. For various reasons, a designer may not be able to devote as much attention to natural phenomena hazard design as he or she might like. Therefore, it is required that the design be reviewed by a qualified, independent consultant or group. For existing facilities, the engineer conducting an evaluation for the effects of natural phenomena hazards will probably be qualified and can devote his/her full attention to the adequacy of the facility to withstand these hazards. In this case, an independent review is not as important as it is for a new design. Even so, for major hazardous facilities, it may be prudent to have concurrent independent evaluations performed or to have the evaluation independently reviewed.

For more information concerning the implementation of a formal engineering quality assurance program and peer review, Chapters 10 and 13 of Reference 20 should be consulted. This reference should also be consulted for information on a construction quality assurance program consistent with the implementation of the engineering quality assurance program.

3 Effects of Natural Phenomena Hazards

3.1 Effects of Earthquakes

For most facilities, the primary seismic hazard is earthquake ground shaking. These guidelines specifically cover the design and evaluation of buildings, equipment, piping, and other structures for shaking. Other earthquake effects that can be devastating to facilities include differential ground motion induced by fault displacement, liquefaction, and seismic-induced slope instability and ground settlement. If these latter earthquake effects cannot be avoided in facility siting, the hazard must be eliminated by site modification or foundation design. Existing facilities located on active fault traces, adjacent to potentially unstable slopes, or on saturated, poorly consolidated cohesionless soil or fill material pose serious questions as to their usage for critical missions or handling hazardous materials.

While earthquake hazards of potential fault movement or other gross soil movement are typically avoided or mitigated, the earthquake ground shaking hazard is unavoidable. When a structure or component is subjected to earthquake shaking, its foundation or support moves with the ground or with the structural element on which it rests. If the structure or equipment is rigid, it follows the motion of its foundation, and the dynamic forces acting on it are nearly equal to those associated with the base accelerations. However, if the structure is flexible, large relative movements can be induced between the structure and its base. Earthquake ground shaking consists of a short duration of time-varying motion that has significant energy content in the range of frequencies of many structures. Thus, for flexible structures, dynamic amplification is possible such that the motions of the structure may be significantly greater than the ground shaking motion. In order to survive these motions, the structural elements must be sufficiently strong, as well as sufficiently ductile, to resist the seismic-induced forces and deformations. The effects of earthquake shaking on structures and equipment depend not only on the earthquake motion to which they are subjected, but also on the properties of the structure or equipment. Among the more important structural properties are the ability to absorb energy (due to damping or inelastic behavior), the natural periods of vibration, and the strength or resistance.

Earthquake ground shaking generally has lateral, vertical, and rotational components. Structures are typically more vulnerable to the lateral component of seismic motion; therefore, a lateral force-resisting system must be developed. Typical lateral force-resisting systems for buildings include moment-resisting frames, braced frames, shear walls, diaphragms, and foundations. Properly designed lateral force-resisting systems provide a continuous load path from the top of the structure down to the foundation. Furthermore, it is recommended that redundant load paths exist. Proper design of lateral force-resisting systems must consider the relative rigidities of the elements taking the lateral load and their capacities to resist load. An example of lack of consideration for relative rigidity are frames with brittle unreinforced infill walls that are not capable of resisting the loads attracted by such rigid construction. In addition, unsymmetrical arrangement of lateral force-resisting elements can produce torsional response which, if not accounted for in design, can lead to damage.

Earthquake ground shaking causes limited energy transient loading. Structures have energy absorption capacity through material damping and hysteretic behavior during inelastic response. The capability of structures to respond to earthquake shaking beyond the elastic limit without major damage is strongly dependent on structural design details. For example, to develop ductile behavior of inelastic elements, it is necessary to prevent premature abrupt failure of connections. For reinforced concrete members, design is based on ductile steel reinforcement in which steel ratios are limited such that reinforcing steel yields before concrete crushes, abrupt bond or shear failure is prevented, and compression reinforcement includes adequate ties to prevent buckling or spalling. With proper design details, structures can be designed to undergo different amounts of inelastic behavior during an earthquake. For example, if the goal is to prevent collapse, structures may be permitted to undergo large inelastic deformations resulting in structural damage that would have to be repaired or replaced. If the goal is to allow only minor damage such that there is minimal or no interruption to the ability of the structure to function, only relatively small inelastic deformations should be permitted. For new facilities, it is assumed that proper detailing will result in permissible levels of inelastic deformation at the specified force levels, without unacceptable damage. For existing facilities, the amount of inelastic behavior that can be allowed without unacceptable damage must be estimated from the as-built condition of the structure.

Earthquake ground shaking also affects building contents and nonstructural features such as windows, facades, and hanging lights. It is not uncommon for the structure to survive an earthquake without serious structural damage but to have significant and expensive internal damage. This damage could be caused by overturned equipment or shelves, fallen lights or ceilings, broken glass, and failed infill walls. Glass and architectural finishes may be brittle relative to the main structure, and they can fail well before structural damage occurs. Windows and cladding must be carefully attached in order to accommodate the seismic movement of the structure without damage. Building contents can usually be protected against earthquake damage by anchorage to the floor, walls, or ceiling.

Facilities in which radioactive materials are handled are typically designed with redundant confinement barriers between the hazardous material and the environment. Such barriers include:

1. The building shell.
2. Ventilation system filtering and negative pressurization that inhibits outward air flow.
3. Storage canisters or glove boxes for storage or handling within the building.

Release of radioactive material to the environment requires failure of two or more of these barriers. Thus, seismic design considerations for these facilities aim to prevent collapse and control cracks or openings (e.g., failed doors, failed infill walls) such that the building can function as a hazardous materials confinement barrier. Seismic design considerations also include adequate anchorage and bracing of storage cannisters and glove boxes and adequate anchorage of ventilation ducting, filters, and pumps to prevent their loss of function during an earthquake. Storage canisters are usually very rugged, and they are not particularly vulnerable to earthquake damage.

Earthquake damage to components of a facility such as tanks, equipment, instrumentation, and piping can also cause injuries, loss of function, or loss of confinement. Many of these items can survive strong earthquake ground shaking with adequate anchorage. Some items, such as large vertical tanks, must be examined in more detail to assure that there is an adequate lateral force-resisting system for seismic loads. For components mounted within a structure, there are three additional considerations for earthquake shaking. First, the input excitation for structure-supported components is the response motion of the structure (which can be amplified from the ground motion) - <u>not</u> the earthquake ground motion. Second, potential dynamic coupling between the component and the structure must be taken into account if the component is massive enough to affect the seismic response of the structure. Third, large differential seismic motions may be induced on components which are supported at multiple locations on a structure or on adjacent structures.

3.2 Effects of Wind

In this document, three types of winds are discussed: extreme (straight), hurricane, and tornado. Extreme (straight) winds are to non-rotating such as those found in thunderstorm gust fronts. Wind circulating around high or low pressure systems are rotational in a global sense, but they are considered "straight" winds in the context of this report. Tornadoes and hurricanes both have rotating winds. The diameter of rotating winds in a small hurricane is considerably larger than the diameter of a large tornado. However, most tornado diameters are relatively large compared to the dimensions of typical buildings. It is estimated that the diameter of 80 percent of all tornadoes is greater than 300 feet.

Wind pressures produced by extreme winds are studied in boundary layer wind tunnels. The results are generally considered reliable because they have been verified by selected full-scale measurements. Investigations of damage produced by extreme winds tend to support the wind tunnel findings. Although the rotating nature of hurricane and tornado winds cannot easily be duplicated in a wind tunnel, damage investigations suggest that pressures produced on enclosed buildings and other structures are similar to those produced by extreme winds, if the relative direction of the rotating wind is taken into account. The appearance of damage to buildings and other structures produced by extreme, hurricane, and tornado winds is so similar that it is almost impossible to look at damage to an individual structure and tell which type of wind produced it. Thus, the approach for determining wind pressures on buildings and other structures proposed in this document is considered to be independent of the type of windstorm. The recommended procedure is essentially the same for straight, hurricane, and tornado winds.

3.2.1 Wind Pressures

Wind pressures on buildings can be classified as external and internal. External pressures develop as air flows over and around enclosed buildings. The air particles change speed and direction, which produces a variation of pressure on the external surfaces of the building. At sharp edges, the air particles separate from contact with the building surface, with an attendant energy loss. These particles produce large outward-acting pressures near the location where the separation takes place. External pressures act outward on all surfaces of an enclosed building, except on windward walls and on steep windward roofs. Overall external pressures include pressures on windward walls, leeward walls, side walls, and roof.

Internal pressures develop when air flows into or out of an enclosed building through broken windows, open doors, or fresh air intakes. In some cases, the natural porosity of the building also allows air to flow into or out of the building. The internal pressure can be either inward or outward, depending on the location of the openings. If air flows into the building through an opening in the windward wall, a "ballooning" effect takes place: pressure inside the building increases relative to the outside pressure. The pressure change produces additional net outward-acting pressures on all interior surfaces. An opening in any other wall or leeward roof surface permits air to flow out of the building: pressure inside the building decreases relative to the outside pressure. The pressure change produces net inward-acting pressure on all interior surfaces. Internal pressures combine with external pressures acting on a building's surface.

On structures other than buildings - such as towers, tanks, or chimneys - interest focuses on the net force acting to overturn or slide the structure, rather than the wind pressure distribution. The magnitude of these forces is determined by wind tunnel or full-scale tests. Also, in special instances particularly associated with aerodynamically sensitive structures, it may be necessary to consider vortex shedding or flutter as a design requirement. Typical sensitive structures are: chimneys, stacks, poles, cooling towers, cable-stayed or supported bridges, and relatively light structures with large smooth surfaces.

Gusts of wind produce dynamic pressures on structures. Gust effects depend on the gust size relative to building size and gust frequency relative to the natural frequency of the building. Except for tall, slender structures (designated wind-sensitive structures), the gust frequencies and the structure frequencies of vibration are sufficiently different that resonance effects are small, but they are not negligible. The size (spatial extent) of a gust relative to the size of the structure, or the size of a component on which the gust impinges, contributes to the magnitude of the dynamic pressure. A large gust that engulfs an entire structure has a greater dynamic effect on the main wind-force resisting system than a small gust whose extent only partially covers the building. On the other hand, a small gust may engulf the entire tributary area of components such as a purlin, a girt, or cladding. In any event, wind loads may be treated as quasi-static loads by including an appropriate gust response factor in calculating the magnitude of wind pressures. Extreme wind, hurricane, and tornado gusts are not exactly the same. However, errors owing to the difference in gust characteristics are believed to be relatively small for those structures that are not wind sensitive.

The roughness of terrain surrounding a structure significantly affects the magnitude of wind speed. Terrain roughness is typically defined in four classes: urban, suburban, open, and smooth. Wind speed profiles as a function of height above ground are represented by a power law relationship for engineering purposes. The relationship gives zero wind speed at ground level. The wind speed increases with height to the top of the boundary layer, where the wind speed remains constant with height.

3.2.2 Additional Adverse Effects of Tornadoes

In addition to wind pressures, tornadoes can produce these hazards: low atmospheric pressure and debris transported by the tornado winds (tornado generated missiles).

Atmospheric pressure change (APC) affects only sealed buildings. Natural porosity, openings, or breach of the building envelope permits the inside and outside pressures of an unsealed building to equalize. Openings of one sq ft per 1000 cu ft volume are sufficiently large to permit equalization of inside and outside pressure as a tornado passes over a building. Buildings or other enclosures that are specifically sealed (e.g., a hot cell) will experience the net pressure difference caused by APC. When APC is present, it acts outward and combines with external wind pressures. The magnitude of APC is a function of the tangential wind speed of the tornado. However, the maximum tornado wind speed and the maximum APC pressure do not occur at the same place. The lowest APC occurs at the center of the tornado vortex, whereas the maximum wind pressure occurs at the radius of maximum winds, which ranges from 150 to 500 feet from the tornado center. The APC pressure is approximately one-half its maximum value at the radius of maximum wind speed.

The rate of APC is a function of the tornado's translational speed, which can vary from 5 to 60 mph. A rapid rate of pressure change can produce adverse effects on HVAC systems.

Violent tornado winds can pick up and transport various pieces of debris, including roof gravel, pieces of sheet metal, timber planks, pipes, and other objects that have high surface area to weight ratios. Automobiles, storage tanks, and railroad cars may be rolled or tumbled by tornado winds. In extremely rare instances, large-diameter pipes, steel wide-flange beams, and utility poles might be transported by very intense tornado winds. These latter missiles are so rare that practicality precludes concern except for high hazard facilities comparable to commercial nuclear power plants.

Three missiles are considered in the design and evaluation of DOE facilities. A timber missile is typical of debris found in the destruction of office trailers, storage sheds, residences, and other light timber structures. Hundreds of these missiles can be generated in the destruction of a residential neighborhood. A steel pipe missile represents a class of debris that includes electrical conduit, liquid and gas piping, fence posts, and light columns. This missile is less frequently available for transport and does not fly as easily as the timber missile. Tornado winds can roll or tumble automobiles, pickup trucks, small vans, forklifts, and storage tanks of comparable size and weight. These missiles do not become airborne, but can damage wall panels, frames, and columns.

The three types of missiles produce varying degrees of damage. A specific type of construction is required to stop each missile. The timber missile is capable of breaking glass and perforating curtain walls or unreinforced masonry walls. Reinforced concrete or masonry walls are required to stop the pipe missile. Timber and pipe missiles can perforate weak exterior walls and emerge with sufficient speed to perforate interior partitions or glove boxes. They also can damage HVAC ducts, HEPA filter enclosures, or pieces of control equipment. The impact of a rolling or tumbling automobile produces failure by excess structural response. Load bearing walls, rigid frames, and exterior columns are particularly susceptible to these objects. Failure of one of these elements could lead to progressive collapse of the structural system.

3.2.3 Effects on Structural Systems

A structural system consists of one-dimensional elements and two-dimensional subsystems that are combined to form the three-dimensional wind-load resisting system. The structural system is enclosed by walls and roof that make up the building envelope. Wind pressures develop on the surfaces of the building envelope and produce loads on the structural system, which in turn transmits the loads to the foundation. The structural system also must support dead and live loads.

Individual elements that make up the two-dimensional subsystems include girders, beams, columns, purlins, girts, piers, and footings. Failure of the elements themselves is quite rare. Element connections are the more common source of failure. A properly conceived wind-force resisting system should not fail as a result of the failure of a single element or element connection. A multiple degree of redundancy should be provided that allows redistribution of load in a ductile system when one element of the system is overloaded. Two-dimensional subsystems transmit wind loads from their points of application to the foundation. Typical subsystems include braced frames, rigid frames, shear walls, horizontal floor and roof diaphragms, and bearing walls. The subsystem must have sufficient strength and stiffness to resist the applied loads without excessive deflection or collapse. The three-dimensional wind-load resisting system is made up of two or more subsystems to form an overall system that is capable of transmitting all applied loads through various load paths to the foundation.

The main wind-force resisting system must be able to resist the wind loads without collapse or excessive deformation. The system must have sufficient ductility to permit relatively large deformations without sudden or catastrophic collapse. Ductility implies an ability of the system to redistribute loads to other components of the system when some part is overloaded.

Keys to successful performance of the wind-resisting system are well-designed connections and anchorages. Precast concrete structures and pre-engineered metal buildings generally have not demonstrated the same degree of satisfactory performance in high winds or tornadoes as conventional reinforced concrete and steel structures. The chief cause of the inadequate behavior is traced to weak connections and anchorages. These latter systems tend to have a lesser number of redundancies, which precludes redistribution of loads when yielding takes place. Failure under these circumstances can be sudden and catastrophic. Timber structures suffer from weak anchorages. Structures that rely on unreinforced load-bearing masonry walls suffer from a lack of ductility. These systems, likewise, can experience sudden collapse under high wind loads. Reinforced masonry walls have inherent strength and ductility of the same order as reinforced concrete walls. Weak anchorages of roof to walls sometimes lead to roof uplift and subsequent collapse of the walls.

3.2.4 Effects on Cladding

Cladding forms the surface of the building envelope. Cladding on walls includes window glass, siding, sandwich panels, curtain walls, brick veneer, masonry walls, precast panels, cast-in-place concrete walls, and in-fill walls. Roof cladding includes wood and metal deck, gypsum planks, poured gypsum, and concrete slabs. Roofing material, such as built-up roofs or single-ply membrane systems, are also a part of the roof cladding.

Cladding failure results in a breach of the building envelope. A breach can develop because the cladding itself fails (excessive yielding or fracture); the connections or anchorages are inadequate; or the cladding is perforated by missiles. Sometimes cladding provides lateral support to purlins, girts, and columns. If the cladding or its anchorage fails, this lateral support is lost, leaving the elements with a reduced load-carrying capacity.

Most cladding failures result from failure of fasteners or the material in the vicinity of the fastener. Cladding failures initiate at locations of high local wind pressures such as wall corners, eaves, ridges, and roof corners. Wind tunnel studies and damage investigations reveal that local pressures can be one to five times greater than overall external pressures.

Breach of the building envelope resulting from cladding failure allows air to flow into or out of the building, depending on where the breach occurs. The resulting internal pressures add to other external wind pressures, producing a worse loading case. Water damage is also a possibility, because most severe storms are accompanied by heavy rainfall.

If the building envelope is breached on two sides of the building, e.g., the windward and leeward walls, a channel of air can flow through the building from one opening to the other. The speed of the flowing air is related to the wind speed outside the building. A high-speed air flow (greater than 40 mph) could collapse interior partitions, pick up small pieces of equipment, or transport toxic or radioactive materials to the environment.

3.3 Effects of Flooding

3.3.1 Causes and Sources of Flooding and Flood Hazards

There are a number of phenomena that can cause flooding in the vicinity of a site. For each cause or source of flooding, a facility may be exposed to one or a number of flood hazards. In most cases, the principal hazard of interest is submergence or inundation. However, significant damage can also occur if there are impact or dynamic forces, hydrostatic forces, water-borne debris, etc. Depending on the cause of flooding (e.g., river flooding, coastal storm surge) and the hazard (e.g., submergence, wave forces), the consequences can be very different.

Table 3-1 lists the various types or causes of flooding that can occur and the particular hazards they pose.

Table 3-1 Causes of Flooding

Source/Cause	Hazard
River flooding/precipitation, snow melt, debris jams, ice jams	Inundation, dynamic forces, wave action, sedimentation, ice loads
Dam failure/earthquake, flood, landslide, static failure (e.g., internal erosion, failure of outlet works)	Inundation, erosion, dynamic loads, sedimentation
Levee or dike failure/earthquake, flood, static failure (e.g., internal erosion, subsidence)	Inundation, erosion, dynamic loads, sedimentation
Precipitation/storm runoff	Inundation (ponding), dynamic loads (flash flooding)
Tsunami/earthquake	Inundation, dynamic loads
Seiche/earthquake, wind	Inundation, dynamic loads
Storm surge, usually accompanied by wave action-/hurricane, tropical storm, squall line	Inundation, dynamic loads
Wave action	Inundation, dynamic loads
Debris	Dynamic loads

From the table, one notes that many of the causes or sources of flooding may be interrelated. For example, flooding on a river can occur due to dam or levee failure or to precipitation.

Depending on the type of flooding and local conditions, the particular hazard posed by a flood can vary. For example, extreme flooding on a river may simply inundate a site. However, in a different situation, channel conditions may be such that prior to the site being inundated, high flows could lead to embankment erosion and structural damage to levees or dikes. Similarly, at coastal sites, storm surge and/or wave action can pose different hazards to a site.

In most cases, flood hazards are characterized in terms of the depth of flooding that occurs on site. This is reasonable because the depth of inundation is probably the single most relevant measure of flood severity. However, the type of damage that is caused by flooding depends very much on the nature of the hazard. For example, it is not uncommon that coastal sites can suffer significant damage due to wave action alone, even if the site is not completely inundated by a storm surge. Similarly, high-velocity flood waters on a river can add substantially to the threat of possible loss of life and the extent of structural damage. In many cases, other hazards - such as wave action, sedimentation, and debris flow - can compound the damage caused by inundation.

3.3.2 Flooding Damage

In many ways, flood hazards differ significantly from other natural phenomena considered in this document. As an example, it is often relatively easy to eliminate flood hazards as a potential contributor to the chance of damage at a hazardous facility by strict siting requirements. Similarly, the opportunity to effectively utilize warning systems and emergency procedures to limit damage and personnel injury is significantly greater in the case of flooding than it is for seismic or extreme winds and tornadoes.

The damage to buildings and the threat to public health vary depending on the type of flood hazard. In general, structural and nonstructural damage will occur if a site is inundated. Depending on the dynamic intensity of on-site flooding, severe structural damage and complete destruction of buildings can result. In many cases, structural failure may be less of a concern than the damaging effects of inundation on building contents and the possible transport of hazardous or radioactive materials.

For hazardous facilities that are not hardened against possible on-site and in-building flooding, simply inundating the site can result in a loss of function of equipment required to maintain safety and in a breach of areas that contain valuable or hazardous materials.

Structural damage to buildings depends on a number of factors related to the intensity of the flood hazard and the local hydraulics of the site. Severe structural damage and collapse generally occur as a result of a combination of hazards such as flood stage level, flow velocity, debris or sediment transport, wave forces, and impact loads. Flood stage is quite obviously the single most important characteristic of the hazard (flood stages below grade generally do not result in severe damage).

In general, the consequences of on-site flooding dramatically increase because flooding varies from submergence to rapidly moving water loaded with debris. Submergence results in water damage to a building and its contents, loss of operation of electrical components, and possible structural damage resulting from extreme hydrostatic loads. Roof collapse can occur when drains become clogged or are inadequate, and when parapet walls allow water, snow, or ice to collect. Also, exterior walls of reinforced concrete or masonry buildings (above and below grade) can crack and possibly fail under hydrostatic conditions.

Dynamic flood hazards can cause excessive damage to buildings not properly designed to withstand dynamic forces. Where wave action is likely, erosion of shorelines or river banks can occur. Structures located near the shore are subject to continuous dynamic forces that can break up a reinforced concrete structure and at the same time undermine the foundation. Buildings with light steel frames and metal siding, wooden structures, and unreinforced masonry are susceptible to severe damage and even collapse if they are exposed to direct dynamic forces. Reinforced concrete buildings are less likely to suffer severe damage or collapse. Table 3-2 summarizes the damage to buildings and flood-protection devices that various flood hazards can cause.

Table 3-2 Flood Damage Summary

Hazard	Damage
Submergence	Water damage to building contents; loss of electric power and component function; settlements of dikes, levees; levee overtopping
Hydrostatic loads	Cracking in walls and foundation damage; ponding on roofs that can cause collapse; failure of levees and dikes due to hydrostatic pressure and leakage
Dynamic loads	Erosion of embankments and undermining of seawalls. High dynamic loads can cause severe structural damage and erosion of levees

The transport of hazardous or radioactive material represents a major consequence of on-site flooding if containment buildings or vaults are breached. Depending on the form and amount of material, the effects could be long-term and widespread once the contaminants enter the ground water or are deposited in populated areas.

4 Earthquake Design and Evaluation Guidelines

4.1 Introduction

This chapter and Appendix A describe the philosophy and procedures for the design or evaluation of facilities for earthquake ground shaking. Much of this material deals with how seismic hazard curves (such as those given in Ref. 1) may be utilized to establish Design Basis Earthquake (DBE per Ref. 7) loads on the facility; how to evaluate the response of the facility to these loads; and how to determine whether that response is acceptable with respect to the performance goals described in Chapter 2. In addition to facility evaluation for seismic loading, this chapter covers the importance of design details and quality assurance to earthquake safety of facilities. These earthquake design and evaluation guidelines are equally applicable to buildings and to items contained within the building, such as equipment and piping. In addition, the guidelines are intended to cover both new construction and existing facilities.

Design of facilities to withstand earthquake ground motion without significant damage or loss of function depends on the following considerations:

1. The facility must have sufficient strength and stiffness to resist the lateral loads induced by earthquake ground shaking. If a facility is designed for insufficient lateral forces or if deflections are unacceptably large, damage can result, even to well-detailed facilities.
2. Failures due to brittle behavior or instability that tend to be abrupt and potentially catastrophic must be avoided. The facility must be detailed in a manner to achieve ductile behavior such that it has greater energy absorption capacity than the energy content of earthquakes.
3. The behavior of the facility as it responds to earthquake ground motion must be fully understood by the designer such that a "weak link" that could produce an unexpected failure is not overlooked.
4. The facility must be constructed in the manner specified by the designer. Materials must be of high quality and as strong as specified by the designer. Construction must be of high quality and must conform to the design drawings.

Specification of lateral load levels and methods of evaluating facility response to these loads (i.e., Item 1 above) are the primary subjects of this chapter. They are discussed in Section 4.2, Section 4.4, and Appendix A. In addition, Reference 21 addresses these subjects. Items 2, 3, and 4 assure good seismic design of facilities; they are described in Section 4.3. References 22 and 23 may be consulted for additional guidance on these items.

Section 4.2 presents specific seismic design and analysis guidelines recommended for DOE facilities. Section 4.3 describes good earthquake design detailing practice and recommended quality assurance procedures. Section 4.4 discusses important seismic design and evaluation considerations such as effective peak ground motion, soil-structure interaction, evaluation of equipment and piping, and evaluation of existing facilities. Appendix A provides commentary that describes the basis for the guidelines presented in Section 4.2.

4.2 Seismic Guidelines for Each Usage Category

4.2.1 General

The guidelines include: (1) lateral force provisions; (2) story drift/damage control provisions; (3) detailing for ductility provisions; and (4) quality assurance provisions. This section presents specific lateral force and story drift provisions for seismic design and evaluation of facilities in each usage category. These seismic design and evaluation provisions include the following steps:

1. Selection of earthquake response spectra.
2. Evaluation of earthquake response.
3. Estimation of seismic capacity and drift limits.

For each usage category, a recommended exceedance probability for the earthquake hazard level is specified from which the maximum ground acceleration may be determined from the hazard curves in Reference 1 or from other site-specific studies of potential seismic hazard. DOE site organizations are encouraged to develop and document site-specific hazard models and other seismic characteristics using currently available information. Evaluating maximum ground acceleration from a specified annual probability of exceedance is illustrated in Figure 4-1. Earthquake input excitation to be used for design and evaluation by these guidelines is defined by a median amplification site-specific response spectrum shape such as that shown in Figure 4-1 anchored to this maximum ground acceleration. Note that the two sets of spectra presented in Figure 4-1 are identical; the top set has spectral amplification plotted against natural frequency on a log scale and the bottom set has spectral amplification plotted against natural period on a linear scale. Note also that design spectra are to be used rather than actual earthquake response spectra. Design spectra such as those shown in Figure 4-1 are smoothed and broadened compared to spectra from any single actual earthquake. With the earthquake input specified in the manner described above, a deterministic approach is outlined for evaluating both the demand placed on a facility and the capacity of that facility. From these data, new facilities may be designed or existing facilities may be evaluated for earthquake ground motion.

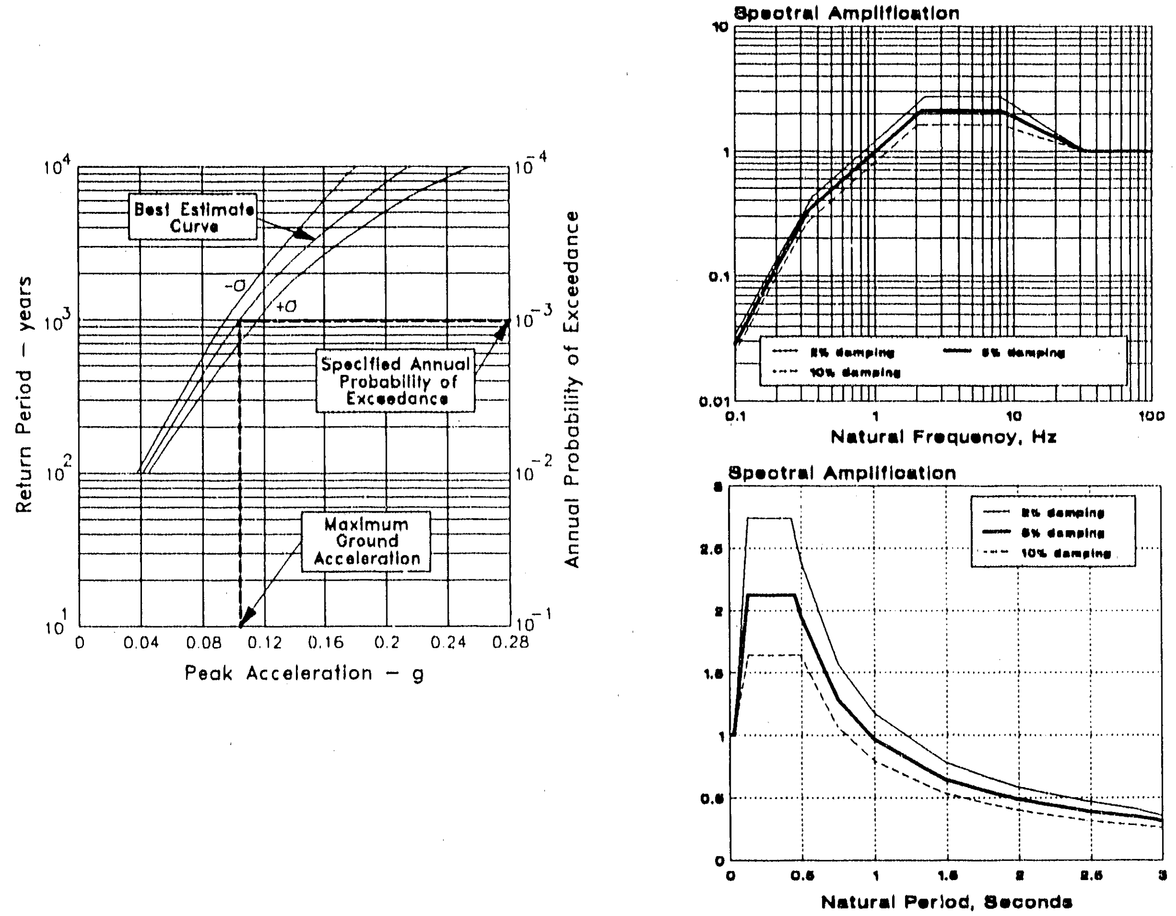


Figure 4-1 Earthquake Input Excitation is Defined by Maximum Ground Acceleration Anchoring Site-Specific Response Spectra

The procedures presented in this chapter are intended to meet the performance goals for structural behavior of facilities as defined in Chapter 2. Meeting performance goals is accomplished by specifying hazard probabilities of exceedance along with seismic behavior evaluation procedures and acceptance criteria in which the level of conservatism introduced is controlled such that desired performance can be achieved. Even though the earthquake ground motion and the performance goals are expressed in probabilistic terms, the design and evaluation procedures to be followed are deterministic response evaluation methods and rules for permissible limits of element stresses and story displacements. The guidelines generally follow the 1988 *Uniform Building Code* (UBC) provisions (Ref. 8) for General Use and Important or Low Hazard facilities and the DOD Tri-service manual for essential buildings (Ref. 9) for Moderate or High Hazard facilities as indicated in Figure 4-2.

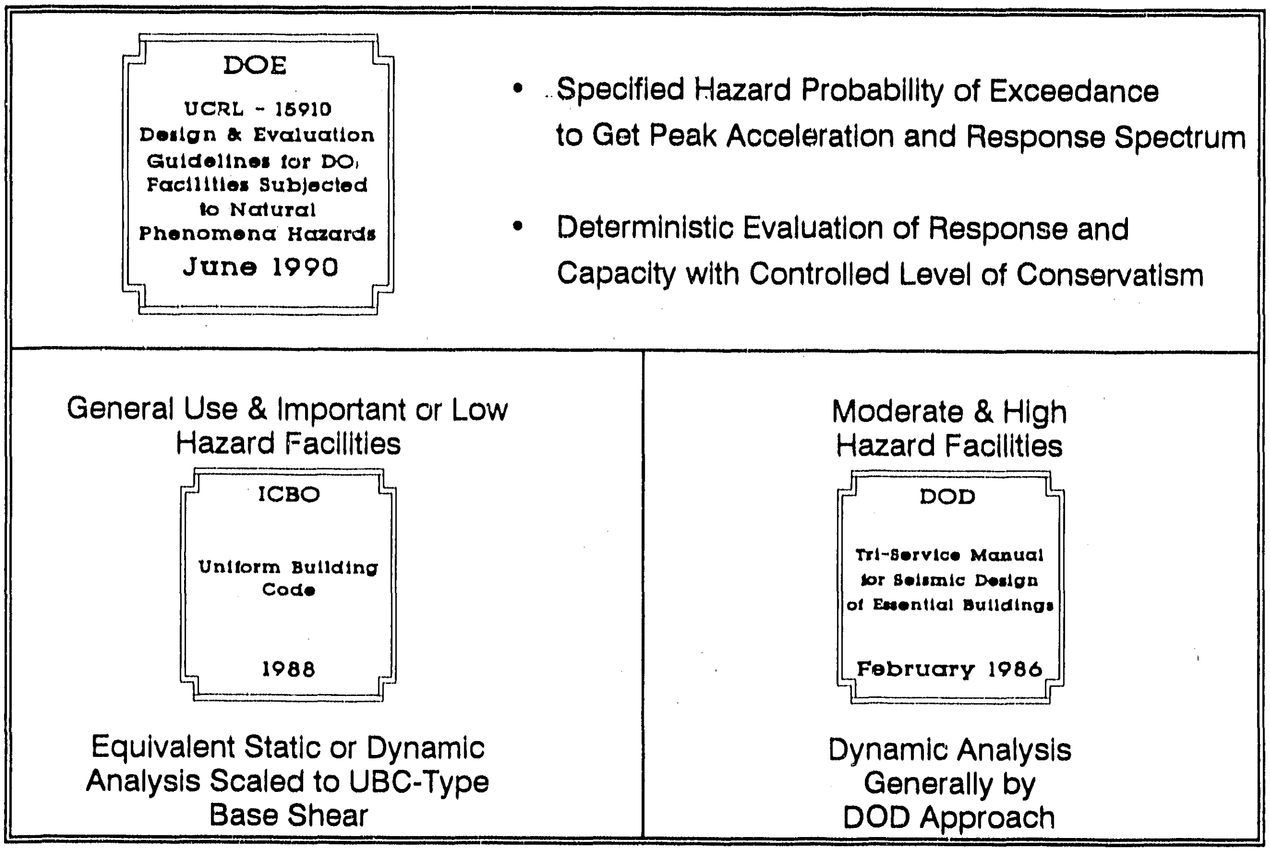


Figure 4-2 Earthquake Guidelines Basic Approch

For General Use and Important and Low Hazard Facilities, the guidelines employ the 1988 UBC provisions with the exception that site-specific information is used to define the earthquake input excitation for design or evaluation (see Table 4-1). The maximum ground acceleration and ground response spectra determined in the manner illustrated in Figure 4-1 are used in the appropriate terms of the UBC equation for base shear. Use of site-specific earthquake ground motion data is considered to be preferable to the general seismic zonation maps from the UBC. Note that if the resulting earthquake design loadings are less than those given by the UBC procedures directly, the differences must be explained and justified to DOE. UBC provisions require a static or dynamic analysis approach in which loadings are scaled to the base shear equation value. In the base shear equation, inelastic energy absorption capacity of structures is accounted for by the parameter, R_W.

Elastically computed seismic response is reduced by R_W values ranging from 4 to 12 as a means of accounting for inelastic energy absorption capability in the UBC provisions and by these guidelines for General Use and Important or Low Hazard facilities.

For Moderate and High Hazard facilities, the guidelines specify that seismic evaluation be accomplished by dynamic analysis (see Table 4-1). The recommended approach is to perform an elastic response spectrum dynamic analysis to evaluate elastic seismic demand on elements of facilities. However, inelastic energy absorption capability of structures is recognized by permitting limited inelastic behavior. By these guidelines for Moderate and High Hazard facilites, inelastic energy absorption capacity of structures is accounted for by the parameter, F_μ. Elastically computed seismic response is reduced by F_μ values ranging from 1 to 3 as a means of accounting for inelastic energy absorption capability for more hazardous facilities. F_μ values are much lower than R_W values introducing increased conservatism for higher hazard facilities and, thereby, reducing the probability of facility damage. By these guidelines, only the element forces due to earthquake loading are reduced by F_μ. This is a departure from the DOD manual (Ref. 9) in which combined element forces due to all concurrent loadings are reduced. The design detailing provisions from the UBC, which provide ductility, toughness, and redundancy for structures, are also recommended for Moderate and High Hazard facilities such that structures can fully realize potential inelastic energy absorption capability.

Table 4-1 General Description of Earthquake Guidelines

Guidelines for General Use & Important or Low Hazard DOE Facilities	Guidelines for Moderate & High Hazard DOE Facilities
Use 1988 UBC provisions $V = \frac{ZICW}{R_w}$ Except: Z from seismic hazard curves C is amplification factor from 5% damped median response spectra	• Generally follow the DOD Manual, "Seismic Design Guidelines for Essential Buildings." • Perform dynamic analysis explicitly modeling the mass & stiffness distribution of the structure. • The guidelines approach is to perform an elastic response spectrum analysis but to permit limited inelastic behavior. Elastic seismic response is reduced by the factor, F_μ to obtain inelastic seismic demand.

Table 4-2 summarizes recommended earthquake design and evaluation guidelines for each usage category. Specific procedures are described in detail in Sections 4.2.2 and 4.2.3. The basis for these procedures is described in Appendix A.

Table 4-2 Summary of Earthquake Evaluation Guidelines

	Usage Category			
	General Use	Important or Low Hazard	Moderate Hazard	High Hazard
Hazard Exceedance Probability	$2x10^{-3}$	$1x10^{-3}$	$1x10^{-3}$	$2x10^{-4}$
Response Spectra	Median amplification (no conservative bias)			
Damping	5%		Post yield (Table 4-6)	
Acceptable Analysis Approaches	Static or dynamic force method normalized to code level base shear		Dynamic analysis	
Importance Factor	I=1.0	I=1.25	Not used	
Load Factors	Code specified load factors appropriate for structural material		Load factors of unity	
Inelastic Demand-Capacity Ratios	Accounted for by R_W in code base shear equation (Ref. 8)		F_μ from Table 4-7 by which elastic seismic response is reduced to account for permissible inelastic behavior	
Material Strength	Minimum specified or known in-situ values			
Structural Capacity	Code ultimate or allowable level		Code ultimate level	
Peer Review, QA, Special Inspection	----	Required		

4.2.2 Evaluation of General Use and Important or Low Hazard Facility Seismic Behavior

Design or evaluation of General Use and Important or Low Hazard facilities for earthquake hazards is based on normal building code seismic provisions. In these guidelines, Reference 8, (1988 edition of the *Uniform Building Code*) is followed for these usage categories. Basic steps in the seismic design and analysis process are summarized in this section. **All 1988 UBC provisions are to be followed for General Use and Important or Low Hazard facilities (with modifications as described below), regardless of whether they are discussed in this section.**

In the 1988 UBC provisions, the lateral force representing the earthquake loading on buildings is expressed in terms of the total base shear, V, given by the following equation:

$$V = ZICW / R_W \tag{4-1}$$

where:
- Z = a seismic zone factor equivalent to peak ground acceleration,
- I = a factor accounting for the importance of the facility,
- C = a spectral amplification factor,
- W = the total weight of the facility,
- R_W = a reduction factor to account for energy absorption capability of the facility (Ref. 8 values are shown in Table 4-2).

For General Use and Important or Low Hazard DOE facilities, it is recommended that the 1988 UBC provisions be followed, with the exception that Z be evaluated from the hazard curves in Reference 1, and that C is the amplification factor from 5% damped median response spectra. It is recommended that both new and existing facilities (also refer to Section 4.4.5 for existing facilities) be evaluated for their adequacy to withstand earthquakes by the step-by-step procedure presented in Table 4-3.

The specified probabilities of exceedance at which maximum ground accelerations are determined from seismic hazard curves are $2x10^{-3}$/year for General Use facilities and $1x10^{-3}$/year for Important or Low Hazard facilities. Based upon these probabilities and the seismic hazard curves in Reference 1, the resulting maximum ground acceleration for each DOE site considered by this program are summarized in Table 4-4. Acceleration values from Reference 1 are given for the free ground surface for average soil/rock conditions at the site.

The approach presented in Table 4-3 is considered to be preferable to the UBC provided that maximum ground accelerations and ground response spectra are developed from site-specific geotechnical studies. Although seismic hazard curves from Reference 1 are site-specific information, the ground response spectra from this reference have been developed from generic soil conditions for the sites. As a result, if response spectra are utilized from Reference 1 or the other references cited in Table 4-3 and the resulting seismic loads are less than that determined by the UBC, the UBC loads should be used for design and evaluation. If the procedure presented in Table 4-3 is used along with earthquake ground motion based on site-specific geotechnical studies and the resulting seismic loads are less than that determined by the UBC, the differences should be justified and approval of seismic loads should be obtained from DOE.

Table 4-3 Earthquake Design/Evaluation Procedures for General Use and Important or Low Hazard DOE Facilities

1. Evaluate element forces, F(DL) and F(LL), throughout the facility for dead and live loads, respectively (realistic estimate of loads for existing facilities).
2. Evaluate element forces, F(EQ), throughout the facility for earthquake loads.
a. Static force method for regular facilities or dynamic force method for irregular facilities as described in the 1988 UBC provisions.
b. In either case, the total base shear is given by Equation 4-1 where the parameters are evaluated as follows:
(1) Z is the peak ground acceleration from the hazard curves (Table 4-4) at the following exceedance probabilities: General Use - 2×10^{-3} Important or Low Hazard - 1×10^{-3}
(2) C is the spectral amplification at the fundamental period of the facility from the 5 percent damped median site response spectra. For fundamental periods lower than the period at which the maximum spectral acceleration occurs, ZC should be taken as the maximum spectral acceleration as illustrated in Figure 4-3. Amplification factors from median spectra may be determined by: a) site-specific geotechnical studies b) References 1, 24, 25, or 26
(3) If ZC is less than the 1988 UBC provisions (Ref. 8):
(a) Earthquake loads should be based on the larger of ZC determined from items 1 and 2 above or from the 1988 UBC provisions unless ZC is based upon a site-specific geotechnical study.
(b) If ZC is based upon a site-specific geotechnical study, any significant differences with UBC will be justified and resolved. Final earthquake loads are subject to approval by DOE.
(4) Importance factor, I, should be taken as: General Use - I = 1.0 Important or Low Hazard - I = 1.25
(5) Reduction factors, R_W, are from Table No. 23-O of Reference 8.
3. Combine responses from various loadings to evaluate demand, D, by: D = LF [F(DL) + F(LL) + F(EQ)] or D = 0.9 F(DL) $\pm$ LF F(EQ) when strength design is used (LF is the load factor which would be 1.4 in the case of concrete) or D = 0.75 [F(DL) + F(LL) + F(EQ)] when allowable stress design is used (the 0.75 factor corresponds to the one-third increase in allowable stress permitted for seismic loads).
4. Evaluate capacities of the elements of the facility, CAP, from code ultimate values when strength design is used (e.g., UBC Sec. 2609 & 2625 for reinforced concrete or LRFD [Ref. 27] for steel) or from allowable stress levels when allowable stress design is used (e.g., UBC Sec. 2702 for steel). Minimum specified or known in-situ values for material strengths should be used for capacity estimation.
5. Compare demand, D, with capacity, CAP, for all structural elements. If D is less than or equal to CAP, the facility satisfies the seismic lateral force requirements. If D is greater than CAP, the facility has inadequate lateral force resistance.
6. Evaluate story drifts (i.e., the displacement of one level of the structure relative to the level above or below due to the design lateral forces), including both translation and torsion. Per Reference 8, calculated story drifts should not exceed $0.04/R_W$ times the story height nor 0.005 times the story height for buildings less than 65 feet in height. For taller buildings, the calculated story drift should not exceed $0.03/R_W$ nor 0.004 times the story height. Note that these story drifts are calculated from seismic loads reduced by R_W in accordance with Equation 4-1. These drift limits may be exceeded when it is demonstrated that greater drift can be tolerated by both structural systems and nonstructural elements.
7. Elements of the facility should be checked to assure that all detailing requirements of the 1988 UBC provisions are met. UBC Seismic Zone No. 2 provisions should be met when Z is between 0.12 and 0.24g. UBC Seismic Zone Nos. 3 & 4 provisions should be followed when Z is 0.25g or more. Special seismic provisions in the UBC need not be followed if Z is 0.11g or less.
8. Peer review of engineering drawings and calculations, special inspection and testing of new construction or existing facilities, and other quality assurance measures discussed in Section 4.3 should be implemented for Important or Low Hazard facilities.

Table 4-4 Maximum Horizontal Ground Surface Accelerations at DOE Sites (Reference 1)

DOE Site	Hazard Annual Probability of Exceedance		
	$2x10^{-3}$	$1x10^{-3}$	$2x10^{-4}$
Bendix Plant	.08	.10	.17
Los Alamos Scientific Laboratory	.18	.22	.38
Mound Laboratory	.12	.15	.23
Pantex Plant	.08	.10	.17
Rocky Flats Plants**	.13	.15	.21
Sandia National Laboratories, Albuquerque	.17	.22	.38
Sandia National Laboratories, Livermore, Ca	.41	.48	.68
Pinellas Plant, Florida	.04	.05	.09
Argonne National Laboratory-East	.09	.12	.21
Argonne National Laboratory-West	.12	.14	.21
Brookhaven National Laboratory	.12	.15	.25
Princeton National Laboratory	.13	.16	.27
Idaho National Engineering Laboratory	.12	.14	.21
Feed Materials Production Center	.10	.13	.20
Oak Ridge National Laboratory, X-10, K-25, and Y-12	.15	.19	.32
Paducah Gaseous Diffusion Plant	.33	.45	*
Portsmouth Gaseous Diffusion Plant	.08	.11	.17
Nevada Test Site	.21	.27	.46
Hanford Project Site	.09	.12	.17
Lawrence Berkeley Laboratory	.55	.64	*
Lawrence Livermore National Laboratory (LLNL)	.41	.48	.68
LLNL, Site 300-854	.32	.38	.56
LLNL, Site 300-834 & 836	.28	.34	.51
Energy Technology And Engineering Center	.53	.59	*
Stanford Linear Accelerator Center	.45	.59	*
Savannah River Plant	.08	.11	.19

* Value not available from Reference 1 and must be determined for High Hazard facilities at these sites.

** Bedrock slopes at Rocky Flats. This value is surface acceleration at an average soil depth at this site.

Note: Values given in this table are largest peak instrumental accelerations. Maximum vertical acceleration may be assumed to be 2/3 of the mean peak horizontal acceleration (see Section 4.4.1 for a discussion of earthquake components and mean peak horizontal acceleration).

Note that the maximum spectral amplification is used in the very low period (very high frequency) region instead of the actual spectral amplification (see Figure 4-3). This is done such that the Table 4-3 approach closely matches the UBC approach in the low period region (i.e., UBC C values remain at their maximum value and do not return to the maximum ground acceleration at very low periods as does an actual response spectrum. Refer to Appendix A for additional discussion on this subject.

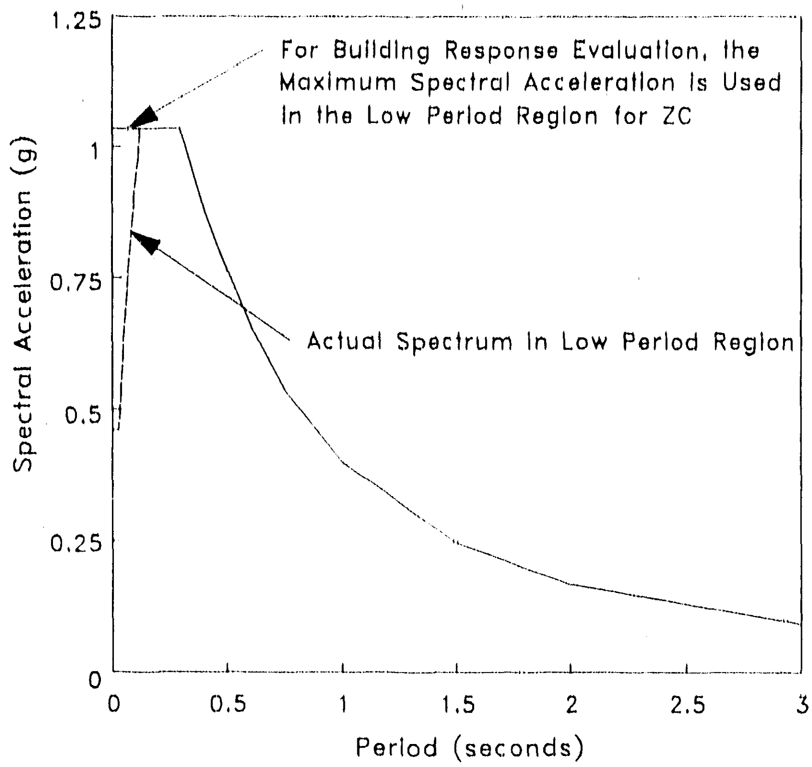


Figure 4-3 Example Design/Evaluation Earthquake Ground Motion Response Spectrum

In the evaluation of existing facilities, it is necessary to evaluate an appropriate value of R_W to be used in Equation 4-1. It is highly unlikely that existing facilities will satisfy the seismic detailing requirements of the 1988 Uniform Building Code if it was designed and constructed many years ago. If structures have less ductility than the UBC provisions require, those structures must be able to withstand larger lateral forces than specified by the guidelines to compensate for non-conforming structural details. As a result, R_W values must be reduced from the values given in the UBC. One acceptable option is that existing structural elements are adequate if they can resist seismic demand forces in an elastic manner (i.e., R_W of unity). To arrive at reduced R_W factors (i.e., between the full code value and unity) requires judgment and care by the engineer performing the evaluation. It is suggested that ATC-14, "Evaluating the Seismic Resistance of Existing Buildings" (Ref. 28) and ATC-22, "A Handbook for Seismic Evaluation of Existing Buildings" (Ref. 29) be reviewed for guidance on this subject.

4.2.3 Evaluation of Moderate and High Hazard Facility Seismic Behavior

Moderate and High Hazard facilities should be evaluated by elastic dynamic analysis. Earthquake input excitation to the elastic dynamic analysis is given by site-specific median response spectra anchored to a maximum ground acceleration determined from site-specific seismic hazard curves at specified annual probabilities of exceedance. Seismic hazard curves and response spectra were illustrated previously in Figure 4-1. Dynamic response analyses should be performed using post-yield damping values which are intended to be median centered, introducing no conservatism into the response evaluation. Limited inelastic behavior is permissible for those facilities with adequate design details such that ductile response is possible or for those facilities with redundant lateral load paths. Inelastic behavior is accounted for in the evaluation approach by specifying inelastic demand-capacity ratios, F_{μ}, for the elements of the facility. These ratios are the amount that the elastically computed element demand is reduced to determine the inelastic seismic demand for structural elements. The inelastic seismic demand is combined with other concurrent loadings to determine total demand on all elements of the facility. Load combination equations to be used for Moderate and High Hazard facilities should have unity load factors for combinations involving earthquake. Total demand is then compared to capacities given by ultimate strength code-type provisions including strength reduction factors.

The procedure described above for Moderate and High Hazard facilities applies to the evaluation of building-type structures. The procedure described above can also be utilized for the evaluation of equipment or systems within buildings. It is recognized that distribution systems such as piping, conduits, HVAC systems, and cable trays have considerable inelastic energy absorption capacity. DOE guidelines for the design and evaluation of such systems which take credit for inelastic energy absorption capacity have not been developed at this time. Special considerations for the seismic design and evaluation of equipment and piping are addressed in Section 4.4.4. It is recommended that both new and existing Moderate and High Hazard facilities (refer also to Section 4.4.5 for existing facilities) be evaluated for their adequacy to withstand earthquakes by the step-by-step procedure presented in Table 4-5.

Table 4-5 Earthquake Design/Evaluation Procedures for Moderate and High Hazard DOE Facilities

1.	Evaluate element forces, F(DL) and F(LL), throughout the facility for dead and live loads (realistic estimate of loads for existing facilities).
2.	Develop median input earthquake response spectra from the Reference 1 hazard curves based upon site-specific geotechnical studies. In lieu of a site-specific study, it is acceptable to determine the median response spectral shape from References 1, 24, 25, or 26. Input spectra should be anchored to peak ground accelerations (Table 4-4) determined from the hazard curves at the following exceedance probabilities: Moderate Hazard - 1×10^{-3} High Hazard - 2×10^{-4} Note that for fundamental periods lower than the period at which the maximum spectral amplification occurs, the maximum spectral acceleration should be used (see Figure 4-4). For higher modes, the actual spectral accelerations should be used in accordance with recommendations from Reference 9. (Note that this requirement necessitates that response spectrum dynamic analysis be performed for building response evaluation). The actual spectrum may be used for all modes if there is high confidence in the frequency evaluation and F_μ is taken to be unity. The actual spectrum at all frequencies should be used to evaluate subsystems mounted on the ground floor; and to develop floor response spectra used for the evaluation of structure-supported subsystems.
3.	Utilizing the input spectra developed above and a mathematical model of the facility, perform an elastic dynamic analysis of the facility to evaluate the elastic earthquake demand, F(EQ), of all elements of the facility. Damping should be determined from Table 4-6.
4.	Evaluate the inelastic earthquake demand, D(EQ), of all elements of the facility from: $D(EQ) = F(EQ) / F_\mu$ where F_μ is the allowable inelastic demand-capacity ratio as given in Table 4-7.
5.	Evaluate the total demand for all elements of the facility, D, from a load combination equation such as: $D = F(DL) + F(LL) + D(EQ)$ For Moderate and High Hazard facilities, the actual load combination equations to be considered for design are similar to those given in ACI 349-85 (Ref. 30) for concrete or ANSI/AISC N690-1984 (Ref. 31) for steel. The load combinations including the safe shutdown earthquake from these references should be used when considering earthquake ground motion by these guidelines. The load combinations including the operating basis earthquake from these references are not used for seismic design or evaluation by these guidelines.
6.	Evaluate capacities of the elements of the facility, CAP, from code ultimate or yield values (e.g., UBC Sec. 2609 & 2625 for reinforced concrete and LRFD [Ref. 27] provisions, AISC Part 2 provisions [Ref. 32], or 1.7 times UBC Sec. 2702 or UBC Sec. 2721 for steel). Note that strength reduction factors, ϕ, are retained for Moderate and High Hazard facilities. Minimum specified or known in-situ values for material strengths should be used for estimation of capacities.
7.	Compare total demand, D, with facility capacity, CAP. If D is less than or equal to CAP, the facility satisfies the seismic lateral force requirements. If D is greater than CAP, the facility has inadequate lateral force resistance.
8.	Evaluate story drifts due to lateral forces, including both translation and torsion. It may be assumed that inelastic drifts are adequately approximated by elastic analyses. Note that for Moderate and High Hazard facilities, loads used to compute drifts are not reduced as is the case for Section 4.2.2 guidelines where loads used to compute story drifts are reduced by R_W. Where confinement of hazardous materials is of importance, calculated story drifts should not exceed 0.010 times the story height. This drift limit may be exceeded when acceptable performance of both the structure and nonstructural elements can be demonstrated at greater drift.
9.	Check elements of the facility to assure that good detailing practice has been followed. Values of F_μ given in Table 4-7 are upper limit values assuming good design detailing practice as discussed in Section 4.3 and consistency with recent UBC provisions. UBC Seismic Zone No. 2 provisions should be met when Z is between 0.12 and 0.24g. UBC Seismic Zone Nos. 3 & 4 provisions should be followed when Z is 0.25g or more. Special seismic provisions in the UBC need not be followed if Z is 0.11g or less.
10.	Implement peer review of engineering drawings and calculations, special inspection and testing of new construction or existing facilities, and other quality assurance measures discussed in Section 4.3 for Moderate and High Hazard facilities.
11.	Inelastic analyses may, alternatively, be performed for Moderate and High Hazard facilities. Acceptable inelastic analysis procedures include: a. Capacity spectrum method as described in Reference 9. b. Direct integration time history analyses explicitly modeling inelastic behavior of individual elements of the facility. Several representative earthquake time histories are required for dependable results from these analyses.

The specified probabilities of exceedance at which maximum ground accelerations are determined from seismic hazard curves are 1×10^{-3}/year for Moderate Hazard facilities and 2×10^{-4}/year for High Hazard facilities. Based upon these probabilities and the seismic hazard curves in Reference 1, the resulting maximum ground acceleration for each DOE site considered by this program are summarized in Table 4-4. Acceleration values from Reference 1 are given for the free ground surface for average soil/rock conditions at the site.

Damping values recommended for dynamic analyses of Moderate and High Hazard facilities are presented in Table 4-6. These damping levels are intended to be appropriate for structural response at or beyond the yield level. Values for structural elements are based upon References 9 and 24. Damping values recommended for equipment and piping are based on Reference 33. These post-yield damping values are intended to be reasonable estimates of median centered damping such that no intentional conservative bias is introduced into seismic response analyses due to damping.

Table 4-6 Recommended Damping Values*
(Based on Ref. 9, 24 and 33)

Type of Structure	Damping (% of Critical)
Equipment and Piping	5
Welded Steel and Prestressed Concrete	7
Bolted Steel and Reinforced Concrete	10
Masonry Shear Walls	12
Wood	15

* Corresponding to post-yield stress levels to be used for evaluation of Moderate and High Hazard Facilities.

Note that the maximum spectral amplification is used for the fundamental frequency of the facility in the very high frequency (very low period) region instead of the actual spectral amplification (see Figure 4-4). For higher modes the actual spectrum is used. This is a simplified and conservative method to account for F_μ being a function of frequency, but using constant F_μ factors. F_μ values given in the guidelines apply to the amplified region of the response spectrum (generally up to 8 or 9 hz). At higher frequencies, F_μ should be unity where the spectrum returns to the maximum ground acceleration and intermediate values between the amplified region and the maximum ground acceleration region. There are other reasons for using maximum amplification in the high frequency range including: (1)

stiff structures may not be as stiff as idealized by dynamic models due to soil-structure interaction (SSI), concrete cracking, basemat flexibility, softening during inelastic response, etc.; and (2) earthquake ground motion in the eastern United States may have more high frequency content than is in standard median design spectra such as those from Reference 24. Refer to Appendix A for additional discussion on this subject.

There are acceptable alternatives to utilizing the maximum spectral amplification for high frequency fundamental modes. The actual spectra may be used for all modes if there is high confidence in the frequency evaluation and F_μ is taken to be unity. Alternatively, a frequency-dependent formulation of F_μ along with the actual spectra may be used. By such a formulation, reduced values of F_μ would be used at frequencies higher than the maximum amplified region of the spectrum. The reduced F_μ values would range from the full Table 4-7 value at the region of maximum amplification to unity where the spectrum returns to the maximum ground acceleration (straight line interpolation on a log-log scale is acceptable).

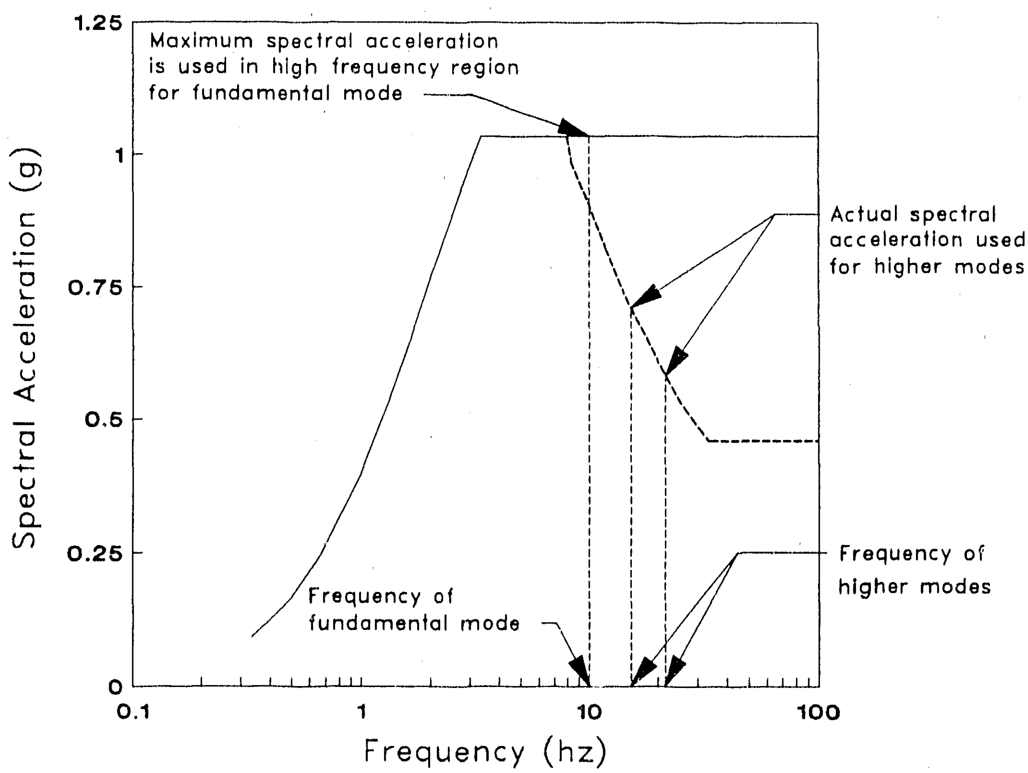


Figure 4-4 Spectral Acceleration in the High Frequency Region for an Example Design/Evaluation Ground Response Spectrum

For building response analyses, the maximum spectral acceleration is to be used for the fundamental mode for all usage categories (or alternative methods as described above). On the other hand, for structure-supported subsystem response analyses, the actual spectra at all frequencies should be used as the basis for in-structure response spectra for Moderate and High Hazard facilities. For evaluation of subsystems in General Use and Important or Low Hazard facilities, it is recommended that the static force coefficient approach as given in the UBC (Ref. 8) be used such that the ground response spectrum need not be considered.

Moderate and High Hazard facilities are evaluated by elastic dynamic analyses. However, limited inelastic behavior is permitted by utilizing inelastic demand-capacity ratios, F_{μ}, as a means to allow limited inelastic behavior in building structures. Recommended F_{μ} values for various structural elements, lateral force resisting systems, and materials of construction are presented in Table 4-7. The inelastic demand-capacity ratio, F_{μ}, is related to the amount of inelastic deformation that is permissible for each type of structural element and in each category. Less inelastic behavior is permitted in less ductile elements such as columns or masonry walls than in very ductile beams of specially detailed moment frames. In addition, by permitting less inelastic behavior for more hazardous categories, the margin of safety for that category is effectively increased and the probability of damage is reduced in accordance with the performance goals.

The approach employed for Moderate and High Hazard facilities is based on the Department of Defense (DOD) Tri-service manual entitled *Seismic Design Guidelines for Essential Buildings* (Ref. 9). However, a significant departure from the DOD manual has been taken by these guidelines in that, by UCRL-15910, only the element forces due to earthquake loading are reduced by F_{μ}. By the DOD manual, combined element forces due to all concurrent loadings are reduced (Note: F_{μ} is called μ by the DOD manual). The approach of reducing only the earthquake contribution to the element force is more conservative than the DOD approach and more consistent with conventional seismic design/evaluation practice. The inelastic demand-capacity ratios in Table 4-7 are similar to those from Reference 9 and they can be shown to be generally consistent with the performance goals for each category and with the R_W factors from the 1988 UBC provisions as discussed in Appendix A.

Table 4-7 Code Reduction Coefficients, R_W and Inelastic Demand Capacity Ratios, F_μ

Structural System (terminology is identical to Ref. 8)	Category		
	GU & I or LH	MH	HH
	R_W	F_μ	
MOMENT RESISTING FRAME SYSTEMS - Beams			
Steel Special Moment Resisting Space Frame (SMRSF)	12	3.0	2.5
Concrete SMRSF	12	2.7	2.2
Concrete Intermediate Moment Frame (IMRSF)	7	1.5	1.2
Steel Ordinary Moment Resisting Space Frame	6	1.5	1.2
Concrete Ordinary Moment Resisting Space Frame	5	1.2	1
SHEAR WALLS			
Concrete or Masonry Walls	8 (6)	1.7	1.4
Plywood Walls	9 (8)	1.7	1.4
Dual System, Concrete with SMRSF	12	2.5	2.0
Dual System, Concrete with Concrete IMRSF	9	2.0	1.7
Dual System, Masonry with SMRSF	8	1.5	1.2
Dual System, Masonry with Concrete IMRSF	7	1.4	1.1
STEEL ECCENTRIC BRACED FRAMES (EBF)			
Beams and Diagonal Braces	10	2.7	2.2
Beams and Diagonal Braces, Dual System with Steel SMRSF	12	3.0	2.5
CONCENTRIC BRACED FRAMES			
Steel Beams	8 (6)	2.0	1.7
Steel Diagonal Braces	8 (6)	1.7	1.4
Concrete Beams	8 (4)	1.7	1.4
Concrete Diagonal Braces	8 (4)	1.5	1.2
Wood Trusses	8 (4)	1.7	1.4
Beams and Diagonal Braces, Dual Systems			
Steel with Steel SMRSF	10	2.7	2.2
Concrete with Concrete SMRSF	9	2.0	1.7
Concrete with Concrete IMSRF	6	1.4	1.1

Note: Values herein assume good seismic detailing practice per Section 4.3 and Reference 8, along with reasonably uniform inelastic behavior. Otherwise, lower values should be used.

Values in parentheses apply to bearing wall systems or systems in which bracing carries gravity loads.

F_μ for columns of all structural systems is 1.5 for Moderate Hazard facilities and 1.2 for High Hazard facilities. For columns subjected to combined axial compression and bending, interaction formulas from Figures 4-2 and 4-3 of Reference 9 should be used for Moderate and High Hazard facilities.

Connections for steel concentric braced frames should be designed for the lesser of:
- the tensile strength of the bracing.
- the force in the brace corresponding to F_μ of unity.
- the maximum force that can be tranferred to the brace by the structural system

Connections for steel moment frames and eccentric braced frames and connections for concrete, masonry, and wood structural systems should follow Reference 8 provisions utilizing the prescribed seismic loads from these guidelines and the strength of the connecting members. In general, connections should develop the strength of the connecting members or be designed for member forces corresponding to F_μ of unity, whichever is less.

F_μ for chevron, vee, and K bracing is 1.5 for Moderate Hazard facilities and 1.2 for High Hazard facilities. K bracing is not permitted in buildings of more than two stories for Z of 0.25g or more. K bracing requires special consideration for any building if Z is 0.25g or more.

For Moderate and High Hazard facilities, it is permissible to use the F_μ value which applies to the overall structural system for structural elements not mentioned on the above table. For example, to evaluate diaphragm elements, footings, pile foundations, etc., F_μ of 3.0 may be used for a Moderate Hazard steel SMRSF. In the case of a Moderate Hazard steel concentric braced frame, F_μ of 1.7 may be used.

A significant difference between the R_W factors from the UBC and the F_μ factors to be used for Moderate and High Hazard facilities is that R_W applies to the entire lateral force resisting system and F_μ applies to individual elements of the lateral force resisting system. For elements for which F_μ is not specified in Table 4-7, it is permissible to use the F_μ value which applies to the overall structural system. For example, to evaluate diaphragm elements, footings, pile foundations, etc.: (1) F_μ of 3.0 may be used for a Moderate Hazard steel SMRSF or (2) in the case of a Moderate Hazard steel concentric braced frame, F_μ of 1.7 may be used.

The Uniform Building Code (Ref. 8) includes special provisions to increase the effective earthquake loading for elements of structures for which damage has been observed during past earthquakes. Such elements include columns, connections, and chevron and K bracing elements. The DOD manual (Ref. 9) accounts for these elements by using reduced inelastic demand-capacity ratios. These guidelines use the same approach as the DOD manual and reduced F_μ values are provided at the bottom of Table 4-7 for chevron, vee, and K bracing members, for connections, and for columns. In addition, further special treatment for columns is recommended in a manner similar to that recommended in the DOD manual. That is, the F_μ factor is only applied to moment or tension. For columns under a combination of axial compression and moment, the axial compression, P, and the moment, M, to be used in code interaction formulas are as follows:

$$P = P_{non-EQ} + P_{EQ} \qquad (4\text{-}2)$$

$$M = M_{non-EQ} + M_{EQ}/F_\mu$$

where P_{EQ} and M_{EQ} are column forces and moments due to earthquake, P_{non-EQ} and M_{non-EQ} are column forces and moments due to other concurrent loads, and P_{cr} and M_p are the compressive and bending capacities of the column. Alternatively, the interaction formulas from Figures 4-2 and 4-3 of the DOD Manual (Ref. 9) may be used for Moderate and High Hazard facilities.

In the evaluation of existing facilities, it is necessary to evaluate an appropriate value of F_μ to be used in the procedures outlined in Table 4-5. F_μ values assume good seismic detailing practice along with reasonably uniform inelastic behavior. Otherwise, lower values should be used. Good detailing practice corresponds to that specified in the 1988 UBC (Ref. 8). It is highly unlikely that existing facilities will satisfy the seismic detailing requirements of the 1988 Uniform Building Code if it was designed and constructed many years ago. If structures have less ductility than the UBC provisions require, those structures must be able to withstand larger lateral forces than specified by the guidelines to compensate for non-conforming structural details. As a result, F_μ values must be reduced from the values given in Table 4-7. One acceptable option is that existing structural elements are adequate if they can resist seismic demand forces in an elastic manner (i.e., F_μ of unity). To arrive at reduced F_μ factors (i.e., between the full Table 4-7 value and unity) requires judgment and care by the engineer performing the evaluation. It is suggested that ATC-14, "Evaluating the Seismic Resistance of Existing Buildings" (Ref. 28) and ATC-22, "A Handbook for Seismic Evaluation of Existing Buildings" (Ref. 29) be reviewed for guidance on this subject.

4.3 Earthquake Resistant Design Considerations

This section briefly describes general design considerations which enable structures or equipment to perform during an earthquake in the manner intended by the designer. These design considerations attempt to avoid premature, unexpected failures and to encourage ductile behavior during earthquakes. This material is intended for both design of new facilities and evaluation of existing facilities. For new facilities, this material addresses recommended seismic design practices. For existing facilities, this material may be used for identifying potential deficiencies in the capability of the facility to withstand earthquakes (i.e., ductile behavior, redundant load paths, high quality materials and construction, etc.). In addition, good seismic design practice, as discussed in this section, should be employed for upgrading or retrofitting existing facilities.

Characteristics of the lateral force-resisting systems are at least as important as the earthquake load level used for design or evaluation. These characteristics include redundancy; ductility; tying elements together to behave as a unit; adequate equipment anchorage; understanding behavior of non-uniform, non-symmetrical structures or equipment; detailing of connections and reinforced concrete elements; and the quality of design, materials, and construction. The level of earthquake ground shaking to be experienced by any facility in the future is highly uncertain. As a result, it is important for facilities to be tough enough to withstand ground motion in excess of their design ground motion level. There can be high confidence in the earthquake safety of facilities designed in this manner. Earthquakes produce transient, limited energy loading on facilities. Because of these earthquake characteristics, well-designed and -constructed facilities (i.e., those with good earthquake design details and high quality materials and construction which provide redundancy and energy absorption capacity) can withstand earthquake motion well in excess of design levels. However, if details that provide redundancy or energy absorbing capacity are not provided, there is little real margin of safety built into the facility. It would be possible for significant earthquake damage to occur at ground shaking levels only marginally above the design lateral force level. Poor materials or construction could potentially lead to damage at well below the design lateral force level. Furthermore, poor design details, materials, or construction increase the possibility that a dramatic failure of a facility may occur.

A separate document providing guidelines, examples, and recommendations for good seismic design of facilities is planned as part of this project. This section briefly describes general design considerations that are important for achieving well-designed and constructed earthquake-resistant facilities and for assessing existing facilities. Considerations for good earthquake resistance of structures, equipment, and piping include: (1) configuration; (2) continuous and redundant load paths; (3) detailing for ductile behavior; (4) tying systems together; (5) influence of nonstructural components; (6) survival of emergency systems; and (7) quality of materials and construction. Each is briefly discussed below. While the following discussion is concerned primarily with buildings, the principles are just as applicable to enhancing the earthquake resistance of equipment, piping, or other components.

Configuration - Structure configuration is very important to earthquake response. Irregular structures have experienced greater damage during past earthquakes than uniform, symmetrical structures. This has been the case even with good design and construction; therefore structures with regular configurations should be encouraged for new designs, and existing irregular structures should be scrutinized very closely. Irregularities such as large reentrant corners create stress concentrations which produce high local forces. Other plan irregularities can result in substantial torsional response during an earthquake. These include irregular distribution of mass or vertical seismic resisting elements or differences in stiffness between portions of a diaphragm. Vertical irregularities can produce large local forces during an earthquake. These include large differences in stiffness or mass in adjacent levels or significant horizontal offsets at one or more levels. An example is the soft story building which has a tall open frame on the bottom floor and shear wall or braced frame construction on upper floors (e.g., Olive View Hospital, San Fernando, CA earthquake, 1971 and Imperial County Services Building, Imperial Valley, CA earthquake, 1979). In addition, adjacent structures should be separated sufficiently so that they do not hammer one another during seismic response.

Continuous and Redundant Load Paths - Earthquake excitation induces forces at all points within structures or equipment of significant mass. These forces can be vertical or along any horizontal (lateral) direction. Structures are most vulnerable to damage from lateral seismic-induced forces, and prevention of damage requires a continuous load path (or paths) from regions of significant mass to the foundation or location of support. The designer/evaluator must follow seismic-induced forces through the structure (or equipment or piping) into the ground and make sure that every element and connection along the load path is adequate in strength and stiffness to maintain the integrity of the system. Redundancy of load paths is a highly desirable characteristic for earthquake-resistant design. When the primary element or system yields or fails, the lateral forces can be redistributed to a secondary system to prevent progressive failure. In a structural system without redundant components, every component must remain operative to preserve the integrity of the structure. It is good practice to incorporate redundancy into the seismic-resisting system rather than relying on any system in which distress in any member or element may cause progressive or catastrophic collapse.

Detailing For Ductile Behavior - In general, for earthquakes that have very low probability of occurrence, it is uneconomical or impractical to design structures to remain within the elastic range of stress. Furthermore, it is highly desirable to design structures or equipment in a manner that avoids brittle response and premature unexpected failure such that the structure or equipment is able to dissipate the energy of the earthquake excitation without unacceptable damage. As a result, good seismic design practice requires selection of an appropriate structural system with detailing to develop sufficient energy absorption capacity to limit damage to permissible levels.

Structural steel is an inherently ductile material. Energy absorption capacity may be achieved by designing connections to avoid tearing or fracture and to ensure an adequate path for a load to travel across the connection. Detailing for adequate stiffness and restraint of compression braces, outstanding legs of members, compression flanges, etc., must be provided to avoid instability by buckling for relatively slender steel members acting in compression. Furthermore, deflections must be limited to prevent overall frame instability due to P-delta effects.

Less ductile materials, such as concrete and unit-masonry, require steel reinforcement to provide the ductility characteristics necessary to resist seismic forces. Concrete structures should be designed to prevent concrete compressive failure, concrete shearing failure, or loss of reinforcing bond or anchorage. Compression failures in flexural members can be controlled by limiting the amount of tensile reinforcement or by providing compression reinforcement and requiring confinement by closely spaced transverse reinforcing of longitudinal reinforcing bars (e.g., spirals, stirrup ties, or hoops and supplementary cross ties). Confinement increases the strain capacity and compressive-, shear-, and bond-strengths of concrete. Maximum confinement should be provided near joints and in column members. Failures of concrete in shear or diagonal tension can be controlled by providing sufficient shear reinforcement, such as stirrups and inclined bars. Anchorage failures can be controlled by sufficient lapping of splices, mechanical connections, welded connections, etc. There should be added reinforcement around openings and at corners where stress concentrations might occur during earthquake motions. Masonry walls must be adequately reinforced and anchored to floors and roofs.

A general recommendation for good seismic detailing is to proportion steel members and to reinforce concrete members such that they can behave in a ductile manner and provide sufficient strength so that brittle or less ductile modes do not govern the overall seismic response. In this manner, sufficient energy absorption capacity can be achieved so that earthquake motion does not produce excessive or unacceptable damage.

Tying Systems Together - One of the most important attributes of an earthquake-resistant structural system is that it is tied together to act as a unit. This attribute not only aids earthquake resistance; it also helps to resist high winds, floods, explosions, progressive failure, and foundation settlement. Different parts of buildings should be interconnected. Beams and girders should be adequately tied to columns, and columns should be adequately tied to footings. Concrete and masonry walls should be anchored to all floors and roofs for lateral support. Diaphragms that distribute lateral loads to vertical resisting elements must be adequately tied to these elements. Collector or drag bars should be provided to collect shear forces and transmit them to the shear-resisting elements, such as shear walls or other bracing elements, that may not be uniformly spaced around the diaphragm. Shear walls must be adequately tied to floor and roof slabs and to footings.

Influence Of Nonstructural Components - For both evaluation of seismic response and for seismic detailing, the effects of nonstructural elements of buildings or equipment must be considered. Elements such as partitions, filler walls, stairs, piping systems, and architectural facings can have a substantial influence on the magnitude and distribution of earthquake-induced forces. Even though these elements are not part of the lateral force-resisting system, they can stiffen that system and carry some lateral force. In addition, nonstructural elements attached to the structure must be designed in a manner that allows for seismic deformations of the structure without excessive damage. Damage to such items as piping, equipment, glass, plaster, veneer, and partitions may constitute a hazard to personnel within or outside the facility and a major financial loss; such damage may also impair the function of the facility to the extent that hazardous operations cannot be shut down or confined. To minimize this type of damage, special care in detailing is required either to isolate these elements or to accommodate structural movements.

In some structures, the system carrying earthquake-induced loads may be separate from the system that carries gravity loads. Although the gravity load carrying systems are not needed for lateral resistance, they would deform with the rest of the structure as it deforms under lateral seismic loads. To ensure that it is adequately designed, the vertical load carrying system should be evaluated for compatibility with the deformations resulting from an earthquake. Similarly, gravity loads should be combined with earthquake loads in the evaluation of the lateral force resisting system.

Survival of Emergency Systems - In addition to preventing damage to structures, equipment, piping, nonstructural elements, etc., it is usually necessary for emergency systems and lifelines to survive the earthquake. Means of ingress and egress (such as stairways, elevator systems, and doorways) must remain functional for personnel safety and for control of hazardous operations. Fire protection systems must remain operational after an earthquake. Normal off-site power has been vulnerable during past earthquakes. Either normal off-site or emergency on-site water and power supplies must be available following an earthquake. Liquid fuels or other flammables may leak from broken lines. Electrical short circuits may occur. Hence, earthquake-resistant design considerations extend beyond the dynamic response of structures and equipment to include survival of systems that prevent facility damage or destruction due to fires or explosions.

Quality of Materials and Construction - Earthquake design or evaluation considerations discussed thus far address recommended engineering practice that maximizes earthquake resistance of facilities. For important or hazardous facilities, it is further recommended that designers or earthquake consultants employ quality assurance procedures and that their work be subjected to independent peer review. Additional earthquake design or evaluation considerations include:

a. Is the facility constructed of high quality materials that meet design specifications for strength and stiffness?

b. Have the design detailing measures, as described above, been implemented in the construction of the facility?

The remainder of this section discusses earthquake engineering quality assurance, peer review, and construction inspection requirements.

To achieve well-designed and -constructed earthquake-resistant facilities or to assess existing facilities, it is necessary to:

a. Understand the seismic response of the facility.
b. Select and provide an appropriate structural system.
c. Provide seismic design detailing that obtains ductile response and avoids premature failures due to instability or brittle response.
d. Provide material testing and construction inspection.

It is recommended that Important or Low Hazard, Moderate Hazard, and High Hazard facilities be designed or evaluated utilizing an earthquake engineering quality assurance plan similar to that recommended by *Recommended Lateral Force Requirements and Tentative Commentary*, Seismology Committee, Structural Engineers Association of California (Ref. 34). The earthquake engineering quality assurance plan should include:

1. A statement by the engineer of record on the earthquake design basis including: (1) description of the lateral force resisting system, and (2) definition of the earthquake loading used for the design or evaluation. For new designs, this statement should be on the design drawings; for evaluations of existing facilities, it should be at the beginning of the seismic evaluation calculations.
2. Seismic design or evaluation calculations should be checked for numerical accuracy and for theory and assumptions. The calculations should be signed by the responsible engineer who performed the calculations, the engineer who checked numerical accuracy, and the engineer who checked theory and assumptions. If the calculations include work performed on a computer, the responsible engineer should sign the first page of the output, describe the model used, and identify those values input or calculated by the computer.
3. For new construction, the engineer of record should specify a material testing and construction inspection program. In addition, the engineer should review all testing and inspection reports and make site visits periodically to observe compliance with plans and specifications. For certain circumstances, such as the placement of rebar and concrete for special ductile frame construction, the engineer of record should arrange to provide a specially qualified inspector to continuously inspect the construction and to certify compliance with the design.
4. For important or hazardous facilities, all aspects of the seismic design or evaluation must include independent peer review. For new construction, the designer will have been selected based on his abilities to design a very complex facility with many problems in addition to seismic safety. Furthermore, the designer will likely be under pressure to produce work on accelerated schedules and for low fees. As a result, the designer may not be able to devote as much attention to seismic design as he might like. Also, because of the low-fee criteria, the most qualified designer may not be selected. Therefore, it is required that the seismic

design be reviewed by a qualified, independent consultant or group. For existing facilities, the engineer conducting a seismic evaluation will likely be qualified and will be able to devote his full attention to evaluating the seismic adequacy of the facility. In this case, an independent review is not as important as it is for a new design. Even so, for major hazardous facilities, it may be prudent to have concurrent independent seismic evaluations performed or to have the seismic evaluation independently reviewed. The seismic design or evaluation review should include design philosophy, structural system, construction materials, design/evaluation criteria used, and other factors pertinent to the seismic capacity of the facility. The review need not provide a detailed check but rather an overview to help identify oversights, errors, conceptual deficiencies, and other potential problems that might affect facility performance during an earthquake.

4.4 Other Seismic Design and Evaluation Considerations

4.4.1 Effective Peak Ground Motion

In accordance with the guidelines presented in this report, loads induced by earthquake ground shaking to be used for the design or evaluation of facilities are based on median amplification response spectra anchored to maximum ground acceleration for specified annual probabilities of exceedance (see Section 4.2 and Appendix A). As a result, each DOE facility requires seismic hazard curves wherein peak ground accelerations are presented as a function of annual probability of exceedance and median amplification response spectra. This ground motion data can be obtained from site-specific studies. Alternatively, Reference 1 provides seismic hazard curves and earthquake response spectra for each DOE facility. In addition, Sections 4.2.2 and 4.2.3 allow the methods described in References 24, 25, and 26 to be used to estimate median spectral amplification. For convenience, this section discusses ground motion as defined by Reference 1. Maximum ground accelerations at the specified annual probabilities of exceedance recommended by these guidelines for each usage category are reproduced in Table 4-3. For some facility sites with high seismic hazard, note that the Reference 1 hazard curves do not provide acceleration values at hazard exceedance probability levels of $2x10^{-4}$. For the design or evaluation of High Hazard facilities at these sites, maximum ground accelerations will have to be developed at $2x10^{-4}$ annual probability of exceedance.

The peak ground accelerations reported in Reference 1 correspond to the maximum acceleration that would be recorded during an earthquake by a three-axis strong motion instrument on a small foundation pad at the free ground surface. This value is called the peak instrumental acceleration. For the following reasons, the largest peak instrumental acceleration and response spectra anchored to such an acceleration often provide an excessively conservative estimate of the ground motion actually input to a stiff, massive structure and/or the damage potential of the earthquake.

a. Peak value of other components is less than the largest peak acceleration as given in Reference 1.
b. Effective peak acceleration based on repeatable acceleration levels with frequency content corresponding to that of structures is a better measure of earthquake damage potential.
c. Soil-structure interaction reduces input motion from instrumental, free ground surface values.

These reasons are discussed extensively in Reference 35; they are briefly addressed below.

First, in most seismic evaluations, it is assumed that the defined ground motion represents both orthogonal horizontal components and that the vertical ground motion component is taken as two-thirds of the average horizontal component. This approach is consistent with the defined ground motion representing the mean peak (average of two horizontal components) instrumental acceleration, rather than the largest peak acceleration as defined by Reference 1. With the largest peak acceleration defined by Reference 1, it is permissible to assume that the second orthogonal horizontal component is 80 percent of the motion defined by Reference 1, while the vertical component is 60 percent of the Reference 1 motion. Note that this assumption is equivalent to the mean peak acceleration being 90 percent of the largest peak value and the vertical component being two-thirds of the mean peak value in accordance with common practice.

Second, the instrumental acceleration is a poor measure of the damage potential of ground motion associated with earthquakes at short epicentral ranges (less than about 20 km). Many structures located close to the epicentral region, which were subjected to high values of peak instrumental acceleration, have sustained much less damage than would be expected considering the acceleration level. In these cases, the differences in measured ground motion, design levels, and observed behavior were so great that it could not be reconciled by considering typical safety factors associated with seismic design. The problem with instrumental acceleration is that a limited number of high frequency spikes of high acceleration are not significant to structural response. Instead, it can be more appropriate to utilize a lower acceleration value that has more repeatable peaks and is within the frequency range of structures. Such a value, called effective peak acceleration, has been evaluated by many investigators who believe it to be a good measure of earthquake ground motion amplitude related to performance of structures. Reference 35 contains a suggested approach for defining the effective peak acceleration. However, this approach would require the development of representative ground motion time histories appropriate for the earthquake magnitudes and epicentral distances that are expected to dominate the seismic hazard at the site. Reference 1 does not contain this information, so special studies would be required for any site to take advantage of the resultant reduction. The reductions that are likely to be justifiable from such studies would most probably be significant for sites with peak instrumental accelerations defined by Reference 1 in excess of about 0.4g. The benefits would be expected to increase with increasing peak instrumental accelerations. These higher ground accelerations most probably are associated with short duration ground motion from earthquakes with short epicentral ranges. If such characteristics can be demonstrated for a particular site, then reductions would be warranted from an instrumental acceleration to an effective acceleration.

Third, various aspects of soil-structure interaction (SSI) result in reduced motion of the foundation basemat of a structure from that recorded by an instrument on a small pad. Such reductions are conclusively shown in Reference 35 and the references cited therein. These reductions are due to vertical spatial variation of the ground motion, horizontal spatial variation of the ground motion (basemat averaging effects), wave scattering effects, and radiation of energy back into the ground from the structure (radiation damping). These effects always result in a reduction of the foundation motion. This reduction tends to increase with increasing mass, increasing stiffness, increasing foundation plan dimensions, and increasing embedment depth. Soil-structure interaction also results in a frequency shift, primarily of the fundamental frequency of the structure. Such a frequency shift can either reduce or increase the response of the structure foundation. These SSI effects are more dramatic with the shorter duration, close epicentral range ground motions discussed in the previous paragraph. It should be emphasized that the ground motion defined by Reference 1 represents the ground motion recorded on a small instrument pad at the free ground surface. It is always permissible to do the necessary soil-structure interaction studies (briefly discussed in Section 4.4.2) in order to estimate more realistic and nearly always lesser foundation motions. It is also permissible, but discouraged, to ignore these beneficial SSI effects and assume the Reference 1 ground motion applies at the foundation level of the structure. However, any frequency shifting due to SSI, when significant, must always be considered.

In summary, it is acceptable, but often quite conservative, to use the ground motion and response spectra defined by Reference 1 as direct input to the dynamic model of the structure as if this motion was applicable at the structure base foundation level. It is also acceptable, and encouraged, for the seismic evaluation to include additional studies to remove sources of excessive conservatism on an individual facility basis, following the guidance described above.

4.4.2 Soil-Structure Interaction (SSI)

When massive stiff structures are founded on or embedded in a soil foundation media, both the frequency and amplitude of the response due to seismic excitation can be affected by soil-structure interaction (SSI), including spatial variation of the ground motion. For rock sites, the effects of the SSI are much less pronounced. It is recommended that the effects of SSI be considered for major structures for all sites with a median soil stiffness at the foundation base slab interface corresponding to a shear wave velocity, v_S, of 3500 fps or lower. Accounting for SSI requires sophisticated seismic analysis techniques that, if performed correctly, will most likely reduce the seismic forces in the structure. Frequency shift effects of SSI can lead to either increased or reduced seismic response and must be considered, if significant. Accounting for other SSI effects is recommended but not required. If SSI effects are considered, the seismic analysis should be reviewed by qualified experts.

The seismic hazard is defined by Reference 1 for the free ground surface. Input into the foundation is then most accurately determined by soil column site analysis. However, the free ground surface motion can be applied to the foundation provided the conservatism thus introduced is acceptable.

Horizontal spatial variations in ground motion result from nonvertically propagating shear waves and from incoherence of the input motion (i.e., refractions and reflections as earthquake waves pass through the underlying heterogeneous geologic media). The following reduction factors may be conservatively used to account for the statistical incoherence of the input wave for a 150-foot plan dimension of the structure foundation (Ref. 35):

Fundamental Frequency of the Soil-Structure System (Hz)	Reduction Factor
5	1.0
10	0.9
25	0.8

For structures with different plan dimensions, a linear reduction proportional to the plan dimension should be used: for example, 0.95 at 10 Hz for a 75-foot dimension and 0.8 at 10 Hz for a 300-foot dimension (based on 1.0 reduction factor at 0-foot plan dimension). These reductions are acceptable for rock sites as well as soil sites. The above reduction factors assume a rigid base slab. Unless a severely atypical condition is identified, a rigid base slab condition may be assumed to exist for all structures for purposes of computing this reduction.

The available information for soil properties at different sites tends to be quite variable concerning the level of detail. Further uncertainty is usually introduced in the development of soil parameters appropriate for SSI analysis. For instance, the degree of soil softening at the dynamic strain levels expected during the defined seismic event, the amount of soil hysteretic material damping, and the impedance mismatches that may exist due to layering are usually not known precisely. It is not the intent to require additional soil boring or laboratory investigations unless absolutely necessary. Rather, a relatively wide range of soil shear moduli (which are usually used to define the foundation stiffness) is recommended such that a conservative structure response calculation may be expected. Decreases in the shear modulus of soils with increasing shear strain should be considered when performing an SSI analysis. The variation in shear modulus as a function of shear strain for sands, gravelly soils, and saturated clays can be found in References 36 and 37.

If there is extensive site-specific soils data available for the DOE facility site being considered, uncertainties in the soil properties should be estimated and incorporated into the SSI analysis. Where extensive site data is not available, an acceptable alternate approach to account for uncertainty in soil properties is: the soil stiffness (horizontal, vertical, rocking, and torsional) employed in analysis should include a range of soil shear moduli bounded by (a) 50 percent of the modulus corresponding to the best estimate at the seismic strain level, and (b) 90 percent of the modulus corresponding to the best estimate at the low strain. Three soil modulus conditions are generally recommended corresponding to (a) and (b) above, and (c), a best estimate shear modulus.

Soil impedances (stiffness and damping) can be accounted for using either Finite Element Methods (FEM), elastic half-space solutions or more refined analytical techniques that address layering, various foundation shapes, and foundation elevations. Elastic half-space solutions using frequency-dependent impedance functions, such as those shown in Table 4-8, are acceptable for facilities on uniform soil sites or sites where the soil properties do not

Table 4-8 Frequency Dependent Elastic Half-Space Impedance

Direction of Motion	Equivalent Spring Constant for Rectangular Footing	Equivalent Spring Constant for Circular Footing	Equivalent Damping Coefficient
Horizontal	$k_x = k_1 2(1+\nu) G \beta_x \sqrt{BL}$	$k_x = k_1 \frac{32(1-\nu)GR}{7-8\nu}$	$c_x = c_1 k_x(static) R\sqrt{\rho/G}$
Rocking	$k_\psi = k_2 \frac{G}{1-\nu}\beta_\psi B^2 L$	$k_\psi = k_2 \frac{8GR^3}{3(1-\nu)}$	$c_\psi = c_2 k_\psi(static) R\sqrt{\rho/G}$
Vertical	$k_z = k_3 \frac{G}{1-\nu}\beta_z \sqrt{BL}$	$k_z = k_3 \frac{4GR}{1-\nu}$	$c_z = c_3 k_z(static) R\sqrt{\rho/G}$
Torsion	————	$k_\theta = k_4 \frac{16}{3} GR^3$	$c_t = c_4 k_t(static) R\sqrt{\rho/G}$

ν = Poisson's ratio of foundation medium,

G = shear modulus of foundation medium,

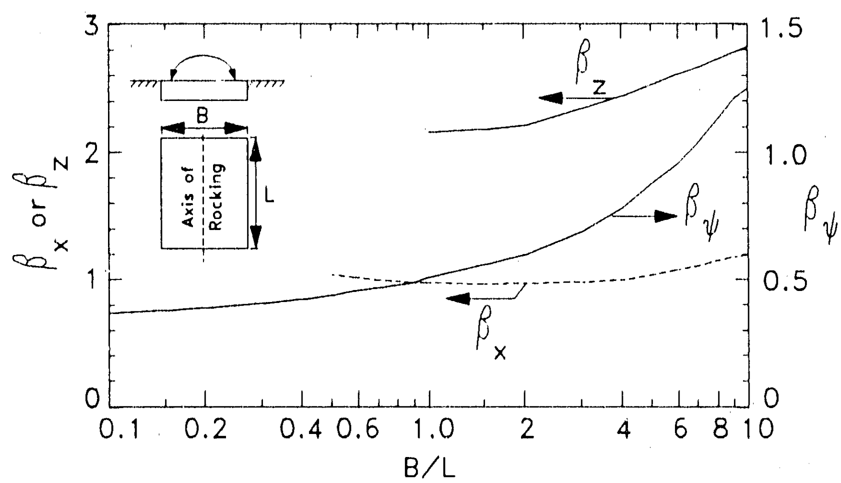
R = radius of the circular base mat,

ρ = density of foundation medium,

B = width of the base mat in the plane of horizontal excitation,

L = length of the base mat perpendicular to the plane of horizontal excitation, and

$k_1, k_2, k_3, k_4,$ c_1, c_2, c_3, c_4 = frequency dependent coefficients modifying the static stiffness or damping (Refs. 38, 39, 41).



Constants β_x, β_z, and β_ψ for a rectangular foundation create significant impedance mismatches between layers. In addition to geometric (radiation) damping developed using either elastic half-space or FEM methods, soil material damping should be included in an SSI analysis. Soil material damping as a function of shear strain can be found in References 36 and 37 for sands, gravelly soils, and saturated clays. Lacking site-specific data, it is appropriate to include soil material damping corresponding to the mean value at the earthquake shaking induced strain level from one of the above references.

For structures that are significantly embedded, the embedment effects should also be included in the SSI analysis. These effects can be incorporated by using available simplified methods for some geometries (Ref. 38 and 39). The potential for reduced lateral soil support of the structure should be considered when accounting for embedment effects. Section 3.3.1.9 of Reference 40 provides guidance on this subject. Similarly, some layer effects can also be incorporated using simplified methods (Reference 41). For more complex situations, more refined analysis, such as that discussed by various authors in Reference 42, is desirable.

4.4.3 Combination of Earthquake Directional Components

Records of actual earthquakes demonstrate that horizontal and vertical components of motion are essentially statistically independent. Consequently, there is only a small probability that the peak responses, due to each of the three individual earthquake components, will occur at the same time. Methods of combining responses from different earthquake components in a reasonable manner are described in this section.

For General Use and Important or Low Hazard facilities, the effects of concurrent earthquake ground motion in orthogonal horizontal directions should be considered for those cases required by the 1988 UBC provisions. This requirement is satisfied by designing elements for 100 percent of the prescribed seismic forces for the ground motion component in one horizontal direction plus 30 percent of the prescribed forces for the ground motion component in the perpendicular horizontal direction. The combination requiring the greater component strength should be used for design/evaluation. Alternatively, the effects of the two orthogonal directions may be combined on a square root of the sum of the squares (SRSS) basis. When the SRSS method of combining directional effects is used, each term computed is assigned the sign that produces the most conservative result. By

UBC provisions, the contribution due to the vertical component is not combined with response from other components. There is a UBC requirement to design horizontal cantilever components for a net upward force.

For Moderate and High Hazard facilities, earthquake responses in a given direction from the three earthquake components should be combined directly, using the assumption that, when the maximum response from one component occurs, the responses from the other two components are 40 percent of the maximum. In this method, all possible combinations of the three orthogonal components, including variations in sign, should be considered. Alternatively, the effects of the three orthogonal directions may be combined by SRSS, as discussed above.

In Section 4.4.1, it was established that the peak value of other components of earthquake ground motion is less than the largest peak acceleration as given in Reference 1. As a result, with the largest peak acceleration defined by Reference 1, it may be assumed that the second orthogonal horizontal component is 80 percent of the motion defined by Reference 1, while the vertical component is 60 percent of the Reference 1 motion. Therefore, when the largest peak acceleration as defined in Reference 1 is used to evaluate earthquake response in a given horizontal direction, response due to the other horizontal direction of motion should be taken as 40 percent of 80 percent of the response computed from the largest peak acceleration. Response due to the vertical component should be taken as 40 percent of 60 percent of the response computed from the largest peak acceleration. Note that this approach is approximately equivalent to the UBC provisions for designing elements for 100 percent of the prescribed seismic forces in one horizontal direction plus 30 percent of the prescribed forces in the perpendicular horizontal direction.

4.4.4 Special Considerations for Equipment and Piping

For DOE facilities that house hazardous operations and materials, the seismic adequacy of equipment and piping is as important as the adequacy of the building. As part of the DOE Natural Phenomena Hazards project, a document has been prepared that provides practical guidelines for the support and anchorage of many equipment items that are likely to be found in DOE facilities (Ref. 6). This document examines equipment strengthening and upgrading to increase the seismic capacity in existing facilities. However, the document is also recommended for considerations of equipment support and anchorage in new facilities.

Special considerations about the seismic resistant capacity of equipment and piping include:

1. Equipment or piping supported within a structure respond to the motion of the structure rather than the ground motion. Equipment supported on the ground or on the ground floor within a structure experiences the same earthquake ground motion as the structure.
2. Equipment or piping supported at two or more locations within a structure are stressed due to both inertial effects and relative support displacements.
3. Equipment or piping may have either negligible interaction or significant coupling with the response of the supporting structure. With negligible interaction, only the mass distribution of the equipment should be included in the model of the structure. The equipment may be analyzed independently. With strong coupling or if the equipment mass is 10 percent or more of the structure story mass, the equipment should be modeled along with the structure model.
4. Many equipment items are inherently rugged and can survive large ground motion if they are adequately anchored.
5. Many equipment items are common to many industrial facilities throughout the world. As a result, there is much experience data available on equipment from past earthquakes and from qualification testing. Equipment which has performed well, based on experience, would not require seismic analysis or testing, if it could be shown to be adequately anchored.
6. The presence of properly engineered anchorage is the most important single item affecting seismic performance of equipment. There are numerous examples of equipment sliding or overturning in earthquakes due to lack of anchorage or inadequate anchorage.

For General Use and Important or Low Hazard facilities, the design or evaluation of equipment or nonstructural elements supported within a structure should be based on the total lateral seismic force, F_p, as given by the 1988 UBC provisions (Ref. 8). For Moderate or High Hazard facilities, the design or evaluation of these items should be based on dynamic analysis, testing, or past earthquake experience data. In any case, equipment items and nonstructural elements must be adequately anchored to their supports. Anchorage must be verified for adequate strength and sufficient stiffness. In the remainder of this section, the UBC lateral force provisions are reproduced, important aspects of dynamic analyses are introduced, the use of past earthquake experience data is discussed, and guidance on equipment anchorage is provided.

UBC lateral force provisions - By the 1988 UBC provisions, parts of structures, permanent nonstructural components, and equipment supported by a structure and their anchorages and required bracing must be designed to resist seismic forces. Such elements should be designed to resist a total lateral seismic force, F_p, of:

$$F_p = ZIC_pW_p \tag{4-3}$$

where: W_p = the weight of element or component

C_p = a horizontal force factor as given by Table 23-P of the UBC for rigid elements, or determined from the dynamic properties of the element and supporting structure for nonrigid elements, as discussed in Section 4.4.4 (in the absence of detailed analysis, the value of C_p for a nonrigid element should be taken as twice the value listed in Table 23-P, but need not exceed 2.0).

The lateral force determined using Equation 4-3 should be distributed in proportion to the mass distribution of the element or component. Forces determined from Equation 4-3 should be used for the design or evaluation of elements or components and their connections and anchorage to the structure, and for members and connections that transfer the forces to the seismic-resisting systems. Forces should be applied in the horizontal directions that result in the most critical loadings for design/evaluation.

Dynamic analysis principles - Guidelines for the design and analysis of equipment or nonstructural elements supported within a structure by dynamic analysis are given in Chapter 6 of Reference 9 and in Reference 40. Elements attached to the floors, walls, or ceilings of a building (e.g., mechanical equipment, ornamentation, piping, and nonstructural partitions) respond to the motion of the building in much the same manner that the building responds to the earthquake ground motion. However, the building motion may vary substantially from the ground motion. The high frequency components of the ground motion are not amplified by the building, while the components of ground motion that correspond to the natural periods of vibrations of the building tend to be magnified. If the elements are rigid and rigidly attached to the structure, accelerations of the elements will be the same as the accelerations of the structure at the attachment points. But elements that are flexible and have periods of vibration close to any of the predominant modes of the building vibration will experience amplified accelerations over that which occurs in the structure.

The most common method of representing support excitation is by means of floor response spectra (also commonly called in-structure response spectra). A floor response spectrum is a response spectrum evaluated from the seismic response at support locations determined from a dynamic analysis of the structure. Floor response spectra can be computed most directly from a dynamic analysis of the structure conducted on a time-step by time-step basis. In addition, there are algorithms available that allow the generation of floor response spectra directly from the ground motion response spectrum and modal properties of the structure without time history analysis (e.g., Refs. 43, 44, and 45). Another method for evaluating floor spectra is provided in Chapter 6 of Reference 9. Note that floor response spectra should generally be developed assuming elastic behavior of the supporting structure even though inelastic behavior is permitted in the design of the structure.

Equipment or piping that is supported at multiple locations throughout the structure could have different floor spectra for each support point. In such a case, it is acceptable to use a single envelope spectrum of all locations as the input to all supports. Alternatively, there are analytical techniques available for using different spectra at each support location or for using different input time histories at each different support.

Testing - Testing may be used to demonstrate the seismic adequacy of all types of equipment. Testing is preferred for complex equipment which must perform an active function during a seismic event if there is not adequate past earthquake experience data to demonstrate seismic adequacy. Input excitation to the tested equipment is to be based on the seismic hazard curves at the specified annual probability for the usage category of the equipment. The resulting maximum ground acceleration is used to anchor median amplification ground response spectra for equipment mounted on the ground or on the ground floor. For equipment mounted in a building, corresponding floor response spectra are developed as input excitation for testing. It is recommended that the resulting input spectrum (ground or floor spectrum) be multiplied by a scale factor to introduce conservatism above that contained in the specification of input excitation such that achievement of Chapter 2 performance goals is more likely. The following input excitation scale factors are recommended for equipment in DOE facilities evaluated by proof testing:

- 1.0 for Important or Low Hazard facilities
- 1.4 for Moderate and High Hazard facilities

The scale factors given above combined with inherent conservatism in testing procedures and in the development of floor spectra is judged to be sufficient that Chapter 2 performance goals may be achieved for equipment items evaluated by testing. Note that it is not expected that testing of equipment for General Use facilities will ever be necessary.

Past earthquake experience data - Since many equipment items within DOE facilities will likely require seismic qualification, seismic experience data and data from past qualification program experience should be utilized, if possible. Seismic experience data is being developed in usable format by ongoing research programs sponsored by the nuclear power industry (Refs. 46, 47, 48, and 49). It is necessary to conduct either seismic analyses or shake table testing to demonstrate sufficient seismic capacity for those items that cannot be eliminated from consideration through the use of seismic experience data or for items that are not obviously invulnerable to earthquakes due to inherent ruggedness. It is also necessary to estimate the input excitation at locations of support for seismic qualification by experience data, analysis, or testing of structure-supported equipment or piping.

Anchorage - Engineered anchorage of equipment or components is required for all usage categories. It is intended that anchorage have both adequate strength and sufficient stiffness. Types of anchorage include: (1) cast-in-place bolts or headed studs; (2) expansion anchor bolts; and (3) welds to embedded steel plates or channels.

Adequate strength of equipment anchorage requires consideration of tension, shear, and tension-shear interaction load conditions. It is recommended that the strength of cast-in-place anchor bolts be based on UBC Sec. 2624 provisions (Ref. 8) for General Use and Important or Low Hazard facilities and on ACI 349-85 provisions (Ref. 30) for Moderate and High Hazard facilities. The strength of expansion anchor bolts should generally be based on design allowable strength values available from standard manufacturers' recommendations or sources such as Reference 49. Design-allowable strength values typically include a factor of safety of about 4 on the mean capacity of the anchorage. It is permissible to utilize strength values based on a lower factor of safety for evaluation of anchorage in existing facilities, provided the detailed inspection and evaluation of anchors is performed in accordance with Reference 49. Currently, a factor of safety on the order of 3 is judged to be appropriate for this situation. When anchorage is modified or new anchorage is designed, it is recommended that design-allowable strength values including the factor of safety of 4 be used. For strength considerations of welded anchorage, it is recommended that AISC, Part 1 (Ref. 32) allowable values multiplied by 1.7 be used.

Stiffness of equipment anchorage as discussed in Reference 47 should also be considered. Flexibility of base anchorage can be caused by the bending of anchorage components or equipment sheet metal. Excessive eccentricities in the load path between the equipment item and the anchor is a major cause of base anchorage flexibility. Equipment base flexibility can allow excessive equipment movement, reducing its natural frequency and possibly increasing its dynamic response. In addition, flexibility can lead to high stresses in anchorage components and failure of the anchorage or equipment sheet metal.

Summary - For General Use and Important or Low Hazard facilities, seismic evaluation of equipment or nonstructural elements supported by a structure can be based on the total lateral seismic force as given by Equation 4-3. For Moderate and High Hazard facilities, the seismic evaluation of equipment and piping necessitates the development of floor response spectra representing the input excitation. Once seismic loading is established, seismic capacity can be determined by analysis, testing, or the use of seismic experience data. It is recommended that, wherever possible, seismic qualification be accomplished through the use of experience data because such an approach is likely to be far less costly and time consuming.

4.4.5 Special Considerations for Evaluation of Existing Facilities

It is anticipated that these guidelines would also be applied to evaluations of existing facilities. General guidelines for the seismic evaluation of existing facilities are presented in a National Institute of Standards and Technology document (Ref. 50), a DOD manual (Ref. 51), and in ATC-14, "Evaluating the Seismic Resistance of Existing Buildings" (Ref. 28) and ATC-22, "A Handbook for Seismic Evaluation of Existing Buildings" (Ref. 29). In addition, guidelines for upgrading and strengthening equipment are presented in Reference 6. These documents should be referred to for the overall procedure of evaluating seismic adequacy of existing facilities, as well as for specific guidelines on upgrading and retrofitting. General requirements and considerations in the evaluation of existing facilities are presented briefly below.

Existing facilities should be evaluated for earthquake ground motion in accordance with the guidelines presented earlier in this chapter. The process of evaluation of existing facilities differs from the design of new facilities in that, the as-built condition of the existing facility must be assessed. This assessment includes reviewing drawings and making site visits to determine deviations from the drawings. In-place strength of the materials should also be determined. The actual strength of materials is likely to be greater than the minimum specified values used for design, and this may be determined from tests of core specimens or sample coupons. On the other hand, corrosive action and other aging processes may have had deteriorating effects on the strength of the structure or equipment, and these effects should also be evaluated in some manner. The inelastic action of facilities prior to occurrence of unacceptable damage should be taken into account because the inelastic range of response is where facilities can dissipate a major portion of the input earthquake energy. The ductility available in the existing facility without loss of desired performance should be estimated based on as-built design detailing rather than using the inelastic demand-capacity ratios presented in Table 4-7. An existing facility may not have seismic detailing to the desired level discussed in Section 4.3 and upon which the values presented in Table 4-7 are based.

Evaluation of existing facilities should begin with a preliminary inspection of site conditions, the building lateral force-resisting system and anchorage of building contents, mechanical and electrical systems, and nonstructural features. This inspection should include review of drawings and facility walkdowns. Site investigation should assess the potential for earthquake hazards in addition to ground shaking, such as active faults that might pass beneath facilities or potential for earthquake-induced landslides, liquefaction, and consolidation of foundation soils. Examination of the lateral force-resisting system, concentrating on seismic considerations as discussed in Section 4.3, may indicate obvious deficiencies or weakest links such that evaluation effort can be concentrated in the most useful areas and remedial work can be accomplished in the most timely manner. Inspection of connections for both structures and equipment indicates locations where earthquake resistance can be readily upgraded.

Once the as-built condition of a facility has been verified and deficiencies or weak links have been identified, detailed seismic evaluation and/or upgrading of the facility can be undertaken. Obvious deficiencies that can be readily improved should be remedied as soon as possible. Seismic evaluation for existing facilities would be similar to evaluations performed for new designs except that a single as-built configuration is evaluated instead of several configurations in an iterative manner (as is required in the design process). Evaluations should be conducted in order of priority. Highest priority should be given to those areas identified as weak links by the preliminary investigation and to areas that are most important to personnel safety and operations with hazardous materials.

As discussed in Chapter 2, the evaluation of existing facilities for natural phenomena hazards can result in a number of options based on the evaluation results. If the existing facility can be shown to meet the design and evaluation guidelines presented in Section 4.2 and good seismic design practice had been employed per Section 4.3, then the facility would be judged to be adequate for potential seismic hazards to which it might be subjected. If the facility does not meet the seismic evaluation guidelines of this chapter, several alternatives can be considered:

1. If an existing facility is close to meeting the guidelines and remaining life of the facility is less than 20 years, a slight increase in the annual risk to natural phenomena hazards can be allowed due to the difficulty in upgrading an existing facility compared to incorporating increased seismic resistance in a new design and due to the fact that existing facilities has a shorter remaining life than a new facility. Note that reduced criteria for existing facilities is supported in Reference 50. As a result, some relief in the guidelines can be allowed by either of the following approximately equivalent approaches:

a. permitting calculated inelastic seismic demand (elastic seismic demand divided by R_W or F_μ) to exceed the seismic capacity by no more than 20 percent, or

b. performing the evaluation using hazard exceedance probability of twice the value recommended in Section 4.2 for each usage category.

2. The facility may be strengthened such that its seismic resistance capacity is sufficiently increased to meet the guidelines. When upgrading is required, it should be accomplished in compliance with unreduced guidelines (i.e., Item 1 provisions should not be used for upgrading).

3. The usage of the facility may be changed such that it falls within a less hazardous usage category and consequently less stringent seismic requirements.

4. It may be possible to conduct the aspects of the seismic evaluation in a more rigorous manner that removes conservatism such that the facility may be shown to be adequate. Alternatively, a probabilistic assessment of the facility might be undertaken in order to demonstrate that the performance goals for the facility can be met.

5 Design and Evaluation Criteria for Wind Load

5.1 Introduction

This chapter presents a uniform approach to wind load determination that is applicable to the design of new facilities and the evaluation of existing ones. As discussed in Section 3.2, a uniform treatment of wind loads is recommended to accommodate extreme, hurricane, and tornado winds. Buildings or facilities are first assigned appropriate usage categories as defined in Chapter 2. Criteria are recommended such that the performance goals for each category can be achieved. Procedures according to ANSI A58.1-1982 (Ref. 14) are recommended for determining wind loads produced by straight, hurricane, and tornado winds. The extreme wind/tornado hazard models for DOE sites published in Reference 2 are used to establish site-specific criteria for each of the 25 DOE sites included in this study.

The performance goals established for General Use and Important or Low Hazard usage categories are met by conventional building codes or standards (see discussion in Chapter 2). These criteria do not account for the possibility of tornado winds, because wind speeds associated with extreme winds typically are greater than those for tornadoes at exceedance probabilities greater than approximately 1×10^{-4}. For this reason, tornado design criteria are specified only for buildings and facilities in Moderate and High Hazard categories, where hazard exceedance probabilities are less than 1×10^{-4}.

The traditional approach for establishing tornado design criteria is to select extremely low exceedance probabilities. For example, the exceedance probability for design of commercial nuclear power plants is 1×10^{-7}. There are reasons for departing from this traditional approach. The low exceedance value for commercial nuclear power plants was established circa 1960 when very little was known about tornadoes from an engineering perspective. Much has been learned about tornadoes since that time. Use of a low hazard probability is inconsistent with the practice relating to other natural hazards, such as earthquakes. There are many uncertainties in tornado hazard probability assessment, but they are not significantly greater than the uncertainties in earthquake probability assessment (see discussion in Appendix A). The strongest argument against using low probability criteria is that a relatively short period of record (37 years) must be extrapolated to extremely small exceedance probabilities. For these reasons, an alternative approach is proposed in these guidelines.

The rationale for establishing tornado criteria is described below. Figure 5-1 shows the tornado and straight wind hazard curves for two DOE sites (SLAC and ORNL). The wind speed at the intersection of the tornado and straight wind curves is defined for purposes of this discussion as the transition wind speed. An exceedance probability is associated with each transition wind speed. If the exceedance probability of the transition wind speed is less than 10^{-5} per year, tornadoes are not a viable threat to the site, because straight winds are more likely. Thus, from Figure 5-1, tornadoes should not be considered at SLAC, but should be at ORNL.

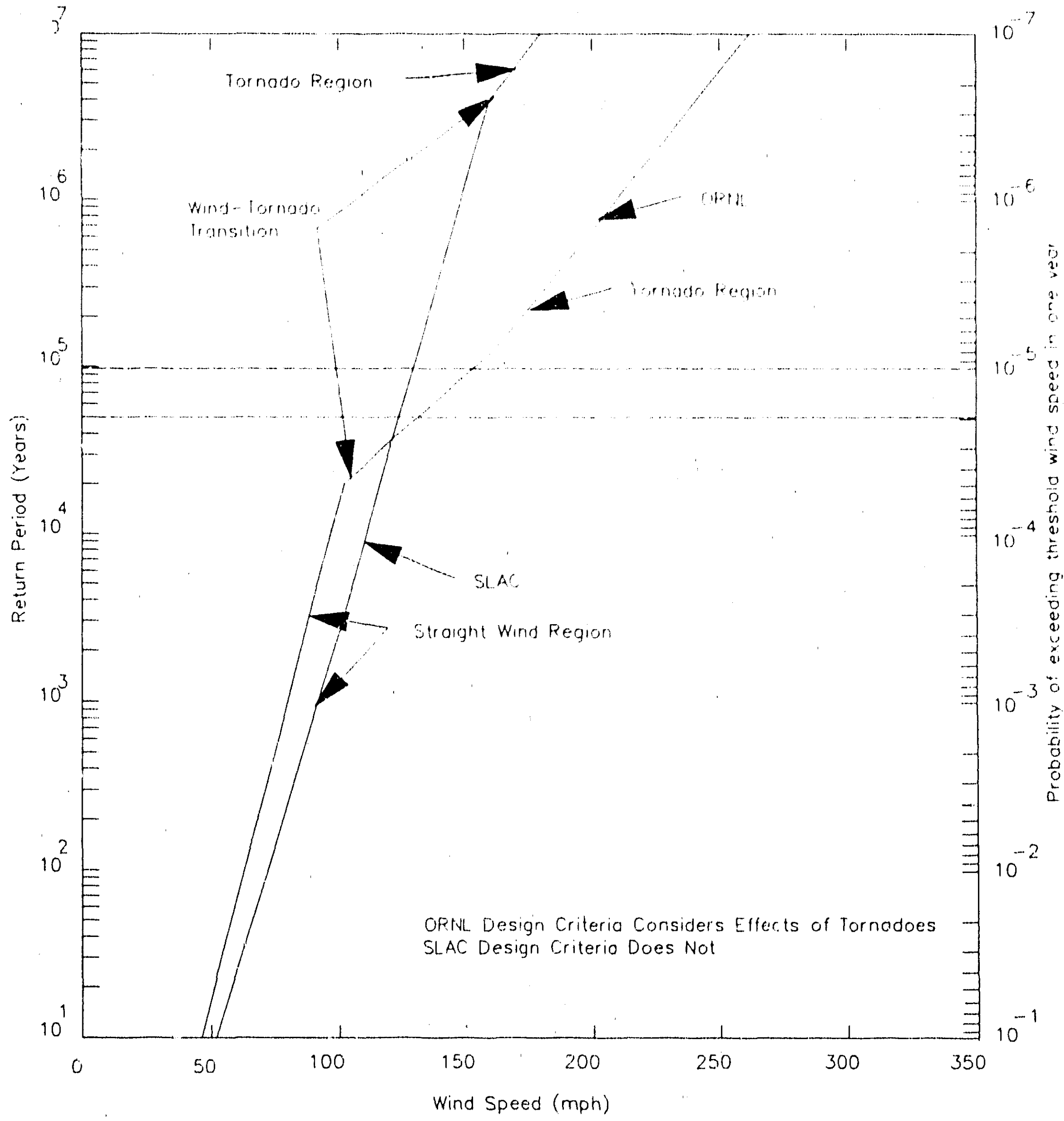


Figure 5-1 Straight Wind and Tornado Regions of Wind Hazard Curves

Table 5-1 tabulates best-estimate wind speeds from Reference 2 for each DOE site, along with the transition wind speed. Those sites with transition wind speed exceedance probabilities greater than 10^{-5} should be designed for tornadoes; others should be designed for extreme winds or hurricanes.

Table 5-1 Types of Wind for Design Loads

Best-Estimate Wind Speeds- mph[1]						
DOE Project sites	Annual Hazard Exceedance Probability				Transition Wind Speed[2]	Type of Wind for Design
	10^{-3}	10^{-4}	10^{-5}	10^{-6}		
Bendix Plant, MO	88	110	177	233	100	Tornado
Los Alamos National Scientific Laboratory, NM	93	107	122	136	140	Extreme
Mound Laboratory, OH	90	108	171	227	104	Tornado
Pantex Plant, TX	98	112	168	220	115	Tornado
Rocky Flats Plant, CO	138	161	183	206	--[3]	Both
Sandia National Laboratories, Albuquerque, NM	93	107	122	135	139	Extreme
Sandia National Laboratories, Livermore, CA	96	113	131	150	--	Extreme
Pinellas Plant, FL	130	150	174	204	181	Hurricane
Argonne National Laboratory--East, IL	72	118	176	226	77	Tornado
Argonne National Laboratory--West, ID	83	95	105	118	119	Extreme
Brookhaven National Laboratory, NY	86	100	127	179	106	Tornado
Princeton Plasma Physics Laboratory, NJ	80	83	135	182	90	Tornado
Idaho National Engineering Laboratory, ID	84	95	105	115	117	Extreme
Feed Materials Production Center, OH	87	108	173	231	96	Tornado
Oak Ridge National Laboratory, X-10, K-25, and Y-12, TN	80	90	152	210	101	Tornado
Paducah Gaseous Diffusion Plant, KY	75	115	180	235	80	Tornado
Portsmouth Gaseous Diffusion Plant, OH	83	95	145	205	98	Tornado
Nevada Test Site, NV	87	100	110	124	131	Extreme
Hanford Project Site, WA	68	77	85	112	89	Extreme
Lawrence Berkeley Laboratory, CA	95	111	130	148	--	Extreme
Lawrence Livermore National Laboratory, CA	96	113	131	150	--	Extreme
Lawrence Livermore National Laboratory Site 300, CA	104	125	145	164	--	Extreme
Energy Technology and Engineering Center, CA	59	68	98	141	74	Tornado
Stanford Linear Accelerator Center, CA	95	112	130	149	158	Extreme
Savannah River Plant, SC	109	138	172	228	155	Tornado

NOTES:

1. Best-estimate wind speeds come from Reference 2.
2. Transition wind speed is at the intersection of the extreme wind hazard and the tornado hazard curves.
3. When transition wind speed is not listed, it is associated with a probability less than 10^{-6}.

The tornado wind speed is obtained by selecting the wind speed associated with an exceedance probability of $2x10^{-5}$ per year. The value of $2x10^{-5}$ is the largest one that can be used and still represent a point on the tornado hazard curve. For example, the tornado wind speed for the ORNL site is 130 mph (peak gust at 10m).

A comparison of the slopes of the tornado hazard curves for the DOE sites in Reference 2 reveals that the slopes are essentially the same even though the transition wind speeds are different. The criteria required to meet the performance goals of Moderate and High Hazard facilities can be met by using multipliers that are equivalent to an importance factor in the ANSI A58.1-1982 design procedure. The multipliers are specified in lieu of two different exceedance probabilities for Moderate and High Hazard facilities. The value of the importance factor is selected to achieve lower probability of tornado damage for High Hazard facilities compared to Moderate Hazard facilities. While the exceedance probabilities specified for tornadoes presented herein still do not match values used for earthquakes, the differences have been reduced as much as possible. The importance factors are then chosen to meet the performance goals stated in Chapter 2.

In general, design criteria for each usage category include:

1. Annual hazard exceedance probability.
2. Importance factor.
3. Wind generated missile parameters for Moderate and High Hazard facilities.
4. Tornado parameters for Moderate and High Hazard facilities, if applicable.

The criteria are formulated in such a way that a uniform approach for determining design wind loads, as specified in ANSI A58.1-1982 (Ref. 14), can be used for extreme, hurricane, and tornado winds.

In order to apply the ANSI A58.1-1982 procedure, tornado gust wind speeds must be converted to fastest-mile (see Appendix B). Appropriate gust response factors and velocity pressure exposure coefficients are utilized in the process of determining wind loads. Appropriate exposure categories also are considered in the wind load calculations. Open terrain (Exposure C) should always be assumed for tornado winds, regardless of the actual terrain conditions.

For an overview of extreme wind and tornado hazards, Reference 52 should be consulted. Reference 53 provides guidance on the design of structures for wind and tornado loads. These references supplement the material presented in this chapter.

5.2 Criteria for Design of Facilities

The criteria presented herein are consistent with the performance goals described in Chapter 2 for each usage category. Buildings or facilities in each category have a different role and represent different levels of hazard to people and the environment. In addition, the degree of wind hazard varies geographically. Facilities in the same usage category, but at different geographical locations, will have different criteria specified to achieve the same performance goal.

The minimum wind design criteria for each of the four usage categories are summarized in Table 5-2. The recommended basic wind speeds for extreme wind, hurricanes, and tornadoes are contained in Table 5-3. All wind speeds are fastest-mile. Minimum recommended basic wind speeds are noted in the table. The use of importance factors in evaluating effective velocity pressure is summarized in Table 5-4. Performance goals and their implications are discussed for each of the categories.

Table 5-2 Summary of Minimum Wind Design Criteria

	Building Category	General Use	Important or Low Hazard	Moderate Hazard	High Hazard
Wind	Annual Probability of Exceedance	2×10^{-2}	2×10^{-2}	1×10^{-3}	1×10^{-4}
Wind	Importance Factor*	1.0	1.07	1.0	1.0
Wind	Missile Criteria			2x4 timber plank 15 lb @ 50 mph (horiz.); max. height 30 ft.	2x4 timber plank 15 lb @ 50 mph (horiz.); max. height 50 ft.
Tornado	Annual Hazard Probability of Exceedance			2×10^{-5}	2×10^{-5}
Tornado	Importance Factor*			I = 1.0	I = 1.35
Tornado	APC			40 psf @ 20 psf/sec	125 psf @ 50 psf/sec
Tornado	Missile Criteria			2x4 timber plank 15 lb @ 100 mph (horiz.); max. height 150 ft; 70 mph (vert.) 3 in. dia. std. steel pipe, 75 lb @ 50 mph (horiz.); max. height 75 ft, 35 mph (vert.)	2x4 timber plank 15 lb @ 150 mph (horiz.), max. height 200 ft; 100 mph (vert.) 3 in. dia. std. steel pipe, 75 lb @ 75 mph (horiz.); max. height 100 ft, 50 mph (vert.) 3,000 lb automobile @ 25 mph, rolls and tumbles

* See Table 5-4 for discussion of importance factors

Table 5-3 Recommended Basic Wind Speeds for DOE Sites

Building Category	Fastest-Mile Wind Speeds at 10m Height					
	General Use	Important or Low Hazard	Moderate Hazard		High Hazard	
	Wind	Wind	Wind	Tornado	Wind	Tornado
DOE PROJECT SITES	$2x10^{-2}$	$2x10^{-2}$	$1x10^{-3}$	$2x10^{-5}$	$1x10^{-4}$	$2x10^{-5}$
Bendix Plant, MO	72	72	--	144	--	144
Los Alamos National Scientific Laboratory, NM	77	77	93	--	107	--
Mound Laboratory, OH	73	73	--	136	--	136
Pantex Plant, TX	78	78	--	132	--	132
Rocky Flats Plant, CO	109	109	138	(3)	161	(3)
Sandia National Laboratories, Albuquerque, NM	78	78	93	--	107	--
Sandia National Laboratories, Livermore, CA	72	72	96	--	113	--
Pinellas Plant, FL	93	93	130	--	150	--
Argonne National Laboratory--East, IL	70(1)	70(1)	--	142	--	142
Argonne National Laboratory--West, ID	70(1)	70(1)	83	--	95	--
Brookhaven National Laboratory, NY	70(1)	70(1)	--	95(2)	--	95(2)
Princeton Plasma Physics Laboratory, NJ	70(1)	70(1)	--	103	--	103
Idaho National Engineering Laboratory	70(1)	70(1)	84	--	95	--
Feed Materials Production Center, OH	70(1)	70(1)	--	139	--	139
Oak Ridge National Laboratory, X-10, K-25, and Y-12, TN	70(1)	70(1)	--	113	--	113
Paducah Gaseous Diffusion Plant, KY	70(1)	70(1)	--	144	--	144
Portsmouth Gaseous Diffusion Plant, OH	70(1)	70(1)	--	110	--	110
Nevada Test Site, NV	72	72	87	--	100	--
Hanford Project Site, WA	70(1)	70(1)	80(1)	--	90(1)	--
Lawrence Berkeley Laboratory, CA	72	72	95	--	111	--
Lawrence Livermore National Laboratory, CA	72	72	96	--	113	--
Lawrence Livermore National Laboratory, Site 300, CA	80	80	104	--	125	--
Energy Technology and Engineering Center, CA	70(1)	70(1)	--	95(2)	--	95(2)
Stanford Linear Accelerator Center, CA	72	72	95	--	112	--
Savannah River Plant, SC	78	78	--	137	--	137

NOTES:

(1) Minimum extreme wind speed.

(2) Minimum tornado speed.

(3) Although extreme winds govern at Rocky Flats, it is recommended that facilities be designed for the tornado missile criteria. APC need not be considered.

Table 5-4 Importance Factors and Effective Velocity Pressures

Usage Category	Extreme Winds	At Hurricane Oceanlines*	Tornadoes
General Use	1.00	1.05	--
Important or Low Hazard	1.07	1.11	--
Moderate Hazard	1.00	1.05	1.00
High Hazard	1.00	1.05	1.35

* For regions between the hurricane oceanline and 100 miles inland, the importance factor, I, shall be determined by linear interpolation.

In ANSI A58.1-1982 (Ref. 14), effective velocity pressure, q_z, at any height z above ground is given by:

$$q_z = 0.00256K_z(IV)^2$$

where K_z is a velocity pressure coefficient evaluated at height z (as a function of terrain exposure category per Table 6 of Ref. 14).
I is importance factor given in Table 5-2 and above.
V is the basic wind speed given in Table 5-3.

5.2.1 General Use Facilities

The performance goals for General Use facilities are consistent with objectives of ANSI A58.1-1982 Building Class I, Ordinary Structures. The wind-force resisting structural system should not collapse under design load. Survival without collapse implies that occupants should be able to find an area of relative safety inside the building. Breach of the building envelope is acceptable, since confinement is not essential. Flow of air through the building and water damage are acceptable. Severe damage, including total loss, is acceptable, so long as the structure does not collapse.

The ANSI A58.1-1982 calls for the basic wind speed to be based on an exceedance probability of 0.02 per year. The importance factor for this class of building is 1.0. For those sites within 100 miles of the Gulf of Mexico or Atlantic coastlines, a slightly higher importance factor is recommended to account for hurricanes (see Table 5-4).

Distinctions are made in the ANSI Specification between buildings and other structures, between main wind-force resisting systems, components, and cladding. In the case of components and cladding, a further distinction is made between buildings less than or equal to 60 ft and those greater than 60 ft in height.

Terrain surrounding the facilities should be classified as Exposure B, C, or D, as appropriate. Gust response factors and velocity pressure exposure factors should be used according to rules of the ANSI A58.1-1982 procedures.

Wind pressures are calculated on the walls and roofs of enclosed buildings by appropriate pressure coefficients specified in the ANSI A58.1-1982 standard. Openings, either of necessity or created by wind forces or missiles, result in internal pressures that can increase wind forces on components and cladding. The worst cases of combined internal and external pressures should be considered, as required by the ANSI standard.

Structures in the General Use category may be designed by either allowable stress design (ASD) or strength design (SD) as appropriate for the material used in construction. Except when applicable codes provide otherwise, plausible load combinations shall be considered to determine the most unfavorable effect on the building, foundation, or structural member being considered. When using ASD methods, allowable stresses appropriate for the building material shall be used with the following combinations that involve wind (Ref. 14):

(a) DL + W
(b) 0.75(DL + W + LL) (5-1)

where

DL = dead load
LL = live load
W = wind load

When using SD methods, the following load combinations that involve wind are recommended:

(a) U = 0.9DL + 1.3W
(b) U = 1.2DL + 0.5LL + 1.3W (5-2)

The SD method requires that the strength provided be greater than or equal to the strength required to carry the factored loads. Appropriate strength reduction factors must be applied to the nominal strength of the material being used.

5.2.2 Important or Low Hazard Facilities

Important or Low Hazard facilities are equivalent to essential facilities (Class II), as defined in ANSI A58.1-1982. The structure's main wind-force resisting structural systems shall not collapse at design wind speeds. Complete integrity of the building envelope is not required because no significant quantities of toxic or radioactive materials are present. However, breach of the building envelope may not be acceptable if wind or water interfere with the facility function. If loss of facility function is caused by water damage to sensitive equipment, collapsed interior partitions, or excessive damage to HVAC ducts and equipment, then loss of cladding and missile perforation at the design wind speeds must be prevented.

An annual wind speed exceedance probability of 0.02 is specified, but the importance factor for Important or Low Hazard category structures is 1.07. For those sites located within 100 miles of the Gulf of Mexico or Atlantic coastlines, a slightly higher importance factor (as listed in Table 5-4) is used to account for hurricane winds.

Once the design wind speeds are established and the importance factors applied, the determination of wind loads on Important or Low Hazard category structures is identical to that described for General Use category structures. Facilities in this category may be designed by ASD or SD methods, as appropriate, for the construction material. The load combinations described for General Use structures are the same for Important or Low Hazard structures. However, greater attention should be paid to connections and anchorages for main members and components, such that the integrity of the structure is maintained.

5.2.3 Moderate Hazard Facilities

The performance goal for Moderate Hazard facilities requires more rigorous criteria than is provided by standards or model building codes. In some geographic regions, tornadoes must be considered.

Extreme Winds and Hurricanes

For those sites where tornadoes are not a viable threat (see Table 5-1), the recommended basic wind speed is based on an annual exceedance probability of 1×10^{-3}. The importance factor is 1.0. For those sites located within 100 miles of the Gulf of Mexico or Atlantic coastlines, a slightly higher importance factor is specified to account for hurricanes (see Table 5-4).

Once the basic wind speeds are established and the importance factors applied, determination of Moderate Hazard category wind loads is identical to that described for the General Use category. Facilities in this category may be designed by ASD or SD methods, as appropriate, for the material being used in construction. Plausible load combinations shall be considered to determine the most unfavorable effect on the building, foundation, or structural member being considered. When using ASD, allowable stresses appropriate for the building material shall be used with the following wind load combinations:

(a) $U = 0.9(DL + W)$
(b) $U = 0.67(DL + W + LL)$ (5-3)

The SD load combinations recommended for the Moderate Hazard category are:

(a) $U = DL + 1.3W$
(b) $U = 1.1DL + 0.5LL + 1.2W$ (5-4)

Greater attention should be paid to connections and anchorages for main members and components, such that the integrity of the structure is maintained.

A minimum missile criteria is specified to account for objects or debris that could be picked up by extreme winds, hurricane winds, or weak tornadoes. A 2x4-in. timber plank weighing 15 lbs. is the specified missile. Its impact speed is 50 mph at a maximum height of 30 ft above ground level. The missile will break glass; it will perforate sheet metal siding, wood siding up to 3/4 in. thick, or form board. The missile could pass through a window or a weak exterior wall and cause personal injury or damage to interior contents of a building. The specified missile will not perforate unreinforced concrete masonry or brick veneer walls or other more substantial walls.

Tornadoes

For those sites requiring design for tornadoes (see Table 5-1), the criteria are based on site-specific studies, as published in Reference 2. The basic wind speed is associated with an annual hazard probability of exceedance of 2×10^{-5}. The wind speed obtained from the tornado hazard model is converted to fastest-mile. The importance factor for the Moderate Hazard category is 1.0.

With the wind speed converted to fastest-mile wind and an importance factor of 1.0, the equations in Table 4 of the ANSI standard should be used to obtain design wind pressures on the structure. Exposure Category C should always be used with tornado winds regardless of the actual terrain roughness. The velocity pressure exposure coefficient and the gust response factor are obtained from appropriate tables in the ANSI standard. External pressure coefficients are used to obtain tornado wind pressures on various surfaces of the structure. A distinction is made between the main wind-force resisting system and components and cladding.

If the building is not specifically sealed to maintain an internal negative pressure for confinement of hazardous materials, or, if openings greater than one square foot per 1000 cubic foot of volume are present, or, if openings of this size can be created by missile perforation, then the effects of internal pressure should be considered according to ANSI procedures. On the other hand, if the building is sealed, then atmospheric pressure change (APC) pressures associated with the tornado should be considered.

APC pressure is half its maximum value at the radius of maximum wind speed in a tornado. Thus, critical tornado loading will be one-half maximum APC pressure plus maximum tornado wind pressure. A loading condition of APC alone can occur on the roof of a buried tank or sand filter, if the roof is exposed at the ground surface. APC pressure always acts outward. The effect of rate of pressure change on ventilation systems should be analyzed to assure that it does not interrupt function or processes carried out in the facility. Procedures and computer codes are available for such analyses.

Plausible load combinations shall be considered to determine the most unfavorable effect on the building, foundation, or structural member being considered. When using ASD methods, allowable stresses appropriate for the building materials shall be used with the following load combinations that involve wind (Ref. 14):

(a) $U = 0.75(DL + W_t)$
(b) $U = 0.625(DL + W_t + LL)$ (5-5)

The SD load combinations recommended for the Moderate Hazard category are:

(a) $U = DL + W_t$
(b) $U = DL + LL + W_t$ (5-6)

where

W_t = tornado loading, including APC, as appropriate.

The rationale for this criteria is explained in Appendix B.

Two missiles are specified as minimum criteria for this usage category. The 2x4-in. timber plank weighing 15 lbs. is assumed to travel in a horizontal direction at a speed up to 100 mph. The horizontal speed is effective up to a height of 150 ft above ground level. If carried to a great height by the tornado winds, the timber plank could achieve a terminal vertical speed of 70 mph in falling to the ground. The horizontal and vertical speeds are assumed to be uncoupled and they should not be combined. The missile will perforate most conventional wall and roof cladding except reinforced masonry or concrete. The cells of concrete masonry walls must be filled with grout to prevent perforation by the timber missile. The second missile is a 3-in.-diameter standard steel pipe, which weighs 75 lbs. It can achieve a horizontal impact speed of 50 mph and a vertical speed of 35 mph. Its horizontal speed could be effective to heights of 75 ft above ground level. The missile will perforate conventional metal siding, sandwich panels, wood and metal decking on roofs, and gypsum panels. In addition, it will perforate unreinforced concrete masonry and brick veneer walls, reinforced concrete masonry walls less than 8 in. thick, and reinforced concrete walls less than 6 in. thick. Although wind pressure, APC, and missile impact loads can act simultaneously in a tornado, the missile impact loads can be treated independently for design and evaluation purposes.

5.2.4 High Hazard Facilities

The performance goal can be achieved for this category if the main wind-force resisting members do not collapse, structural components do not fail, and the building envelope is not breached at the design wind loads. Loss of cladding, broken windows, collapsed doors, or significant missile perforations must be prevented. Strong air flow through the building or water damage cannot be tolerated.

Extreme Winds and Hurricanes

For those sites that do not require specific design for tornado resistance, the recommended basic wind speed is based on an annual hazard exceedance probability of 1×10^{-4}. The importance factor is 1.0, as shown in Table 5-4. The wind speed is fastest-mile at an anemometer height of 10 meters above ground level.

Once the basic wind speeds are established and the importance factors applied, determination of High Hazard facility wind loads is identical to that described for the General Use category. Facilities in this category may be designed by ASD or SD methods, as appropriate, for the material being used in construction. Recommended wind load combinations are the same as for Moderate Hazard facilities (Equations 5-3 and 5-4). Greater attention should be paid to connections and anchorages for main members and components, such that the integrity of the structure is maintained.

The missile criteria is the same as for the Moderate Hazard category, except that the maximum height achieved by the missile is 50 ft, instead of 30 ft.

Tornadoes

For those sites requiring design for tornado resistance (see Table 5-1), the criteria is based on site-specific studies as published in Reference 2. The recommended basic wind speed is associated with an annual hazard probability of exceedance of 2×10^{-5} (the same as the Moderate Hazard category). The wind speed obtained from the tornado hazard model is converted to fastest-mile. The importance factor for the High Hazard category is 1.35.

With the wind speed expressed as fastest-mile and an importance factor of 1.35, the equations in Table 4 of ANSI A58.1-1982 should be used to obtain design wind pressures on the structure. Exposure Category C should always be used with tornado winds regardless of actual terrain roughness. The velocity pressure exposure coefficient and the gust response factor are obtained from appropriate tables in the ANSI standard. External pressure coefficients are used to obtain tornado wind pressures on various surfaces of the structure. A distinction is made between the main wind-force resisting system and components and cladding in determining wind pressures.

If the building is sealed to confine hazardous materials, the wind and APC load combinations specified for the Moderate Hazard usage category also should be used for this category. The effects of rate of pressure change on ventilating systems should be analyzed. Recommended tornado wind load combinations for Moderate Hazard facilities (Equations 5-5 and 5-6) also apply to High Hazard facilities. See Appendix B for an explanation of the criteria.

Three missiles are specified as minimum criteria for this usage category. The 2x4-in. timber plank weighs 15 lbs. and is assumed to travel in a horizontal direction at speeds up to 150 mph. The horizontal missile is effective to a maximum height of 200 ft above ground level. If carried to a great height by the tornado winds, it could achieve a terminal speed in the vertical direction of 100 mph. The horizontal and vertical speeds are uncoupled and should not be combined. The missile will perforate most conventional wall and roof cladding except reinforced masonry and concrete. Each cell of the concrete masonry shall contain a 1/2-in.-diameter rebar and be grouted to prevent perforation by the missile. The second missile is a 3-in.-diameter standard steel pipe, which weighs 75 lbs. It can achieve a horizontal impact speed of 75 mph and a vertical speed of 50 mph. The horizontal speed could be effective at heights up to 100 ft above ground level. This missile will perforate unreinforced concrete masonry and brick veneer walls, reinforced concrete masonry walls less than 12 in. thick, and reinforced concrete walls less than 8 in. thick. The third missile is a 3000-lb automobile that is assumed to roll and tumble on the ground and achieve an impact speed of 25 mph. Impact of an automobile can cause excessive structural response to columns, walls, and frames. Impact analyses should be performed to determine specific effects. Collapse of columns, walls, or frames may lead to further progressive collapse.

5.2.5 Recommended Design Wind Speeds for Specific DOE Sites

The criteria specified in Table 5-2 for the four usage categories should be applied to the site-specific extreme wind/tornado hazard models for each of the 25 DOE sites included in this study. Table 5-3 summarizes the recommended design wind speeds. Appropriate importance factors to be used with the wind speeds are listed in Table 5-4. The wind speeds are fastest-mile. Minimum wind speed values for a particular usage category have been imposed. The wind speeds listed in Table 5-3 should be treated as basic design wind speeds in the ANSI A58.1-1982 procedures for determining wind pressures on buildings and other structures.

The following sites require design for extreme winds:

Argonne National Laboratory-West, ID
Hanford Project Site, WA
Idaho National Engineering Laboratory, ID
Lawrence Berkeley Laboratory, CA
Lawrence Livermore National Laboratory, CA
Lawrence Livermore National Laboratory Site 300, CA
Los Alamos National Scientific Laboratory, NM
Nevada Test Site, NV
Pinellas Plant, FL
Sandia National Laboratories, Albuquerque, NM
Sandia National Laboratories, Livermore, CA
Stanford Linear Accelerator Center, CA

The Rocky Flats Plant site presents a unique situation. The presence of downslope winds dominates the extreme wind distribution, suggesting that the design criteria should be based on extreme wind criteria. However, tornadoes are possible and have occurred near the site. Hence, both extreme winds and tornadoes should be considered in arriving at a final design criteria for this site. A specific hazard assessment was performed for the Pinellas Plant, FL, whose wind design is governed by hurricanes (see Table 5-1). The importance factor for this site should not be increased above the value for straight winds.

The sites for which tornadoes are the viable wind hazard include:

Argonne National Laboratory - East, IL
Bendix Plant, MO
Brookhaven National Laboratory, NY
Energy Technology and Engineering Center, CA
Feed Metals Production Center, OH
Mound Laboratory, OH
Oak Ridge National Laboratory, TN
Paducah Gaseous Diffusion Plant, KY
Pantex Plant, TX
Portsmouth Gaseous Diffusion Plant, OH
Princeton Plasma Physics Laboratory, NJ
Savannah River Plant, SC

Brookhaven National Laboratory, Long Island, NY, and Princeton Plasma Physics Laboratory, NJ, are located in hurricane-prone zones. See Table 5-4 for values of importance factor for hurricane winds. For Moderate and High Hazard categories, the minimum tornado wind speed criteria apply because they are a worse case than the hurricane criteria.

5.3 Criteria for Evaluation of Existing Facilities

The performance goals for design presented in the previous section may be used to evaluate existing facilities. The objective of the evaluation process is to determine if an existing facility meets the performance goals for a particular usage category.

The key to the evaluation of existing facilities is to identify the potential failure points in a structure. The critical failure mechanism could be failure of the wind-load resisting structural subsystem, or it could be a breach of the building envelope that allows release of toxic materials to the environment or results in wind or water damage to the building contents.

The structural subsystems of many old facilities (25 to 40 years old) have considerable reserve strength because of conservatism used in the design approach. However, the facility could still fail to meet performance goals if breach of the building envelope is not acceptable.

The weakest link in a structural system usually determines the adequacy or inadequacy of the performance of a structure under wind load. Thus, evaluation of existing facilities normally should focus on the strengths of connections and anchorages in both the wind-force resisting subsystems and in the components and cladding.

Experience from windstorm damage investigations provides the best guidelines for anticipating the potential performance of existing structural systems under wind load conditions. Reference 54 provides a methodology for estimating the performance of existing structural systems. A brief outline of the approach is now presented. The steps include:

1. Data collection.
2. Analysis of component failure.
3. Postulation of failure mechanisms and their consequences.
4. Comparison of postulated performance with performance goals.

5.3.1 Data Collection

Construction drawings and specifications of the building or facility are needed to make the evaluation for the wind hazard. A site visit is usually required to verify that the facility was built according to plans and specifications. Modifications subsequent to preparation of the drawings should be verified.

Material properties are required for the structural analyses. Accurate determination of material properties may be the most challenging part of evaluation of existing facilities. Median values of material properties should be obtained. This will allow an estimate of the degree of conservatism in the analysis if other than the median values were used.

5.3.2 Analysis of Components

After determining the as-built condition and the material properties, the wind-resistant subsystem(s) are modeled and analyzed. The type of model employed depends on the material, the loads, and the connections. Modeling of the structural system should include load path identification, stiffness calculations, and support restraint determination. Once the system is modeled, all appropriate loads and load combinations (including dead, live, and wind loads) should be considered in the analyses.

Most of the time, it is not feasible to model the three-dimensional load-resisting system. In that case, the system is decomposed into subsystems or individual elements. Wind loads, appropriate to the usage category, are imposed on these structural components and their ability to sustain the loads are evaluated.

Breach of the building envelope may not be tolerable for some usage categories. The building envelope is breached by cladding failure or by tornado missile impact.

Cladding failure can occur in the walls or the roof. Wall cladding, as used in the general sense, includes all types of attached material as well as in-fill walls, masonry walls, or precast walls. The strength of anchorages and fasteners should be checked, as well as the strength of the materials. Roof cladding includes material fastened to the roof support system (purlins or joists) such as metal deck, gypsum planks, or timber decking, as well as poured slabs of gypsum or concrete (normal or light weight). External wind pressures and appropriate internal pressures should be used to evaluate cladding performance.

The tornado missiles in the performance criteria are selected to require certain types of cladding to stop them, based on experimental tests. If existing facilities have exterior walls that are not capable of stopping the missile, then the consequences of the missile perforating exterior walls should be evaluated.

5.3.3 Postulation of Failure Mechanism

After analyzing the structural load-resisting systems under loads appropriate to the usage category, it is possible to identify potential failure mechanisms. The failure mechanism can range from subsystem collapse to the failure of an individual element such as a column, beam, or particular connection. The consequences of the postulated failure are evaluated in light of the stated performance goals for the designated usage category.

The failure of cladding or individual elements or subsystems can lead to a change in the loading condition or a change in the support restraints of various components of the load-resisting system. A breach in the envelope of a sealed building results in a change in the internal wind pressure of a building. The change in pressure, which can be an increase or a decrease, adds vectorially to external pressures, which may lead to additional component failures. The uplift of a building roof leaves the tops of walls unsupported, and therefore reduces their capacity to resist wind loads.

5.3.4 Comparison of Postulated Performance with Performance Goals

Once the postulated failure mechanisms are identified, the structural system performance is compared with the stated performance goals for the specified usage category. The general procedures described in Chapter 2 (Figure 2-1) are followed. If the wind load-resisting system is able to resist the design loads without violating performance goals, then the facility meets the criteria. If the guideline criteria are not met, then the assumption and methods of analysis can be modified to eliminate unnecessary conservatism introduced in the evaluation methods. The hazard probability levels can be raised slightly if the facility is close to meeting the criteria (it is acceptable to increase the hazard probability level by a factor of 2, as is done for the earthquake evaluation described in Chapter 4). Otherwise, various means of retrofit can be employed. Several options are listed below, but the list is not exhaustive.

1. Add x-bracing or shear walls to obtain additional lateral load-resisting capacity.
2. Modify connections in steel, timber, or precast concrete construction to permit them to transfer moment, thus increasing lateral load resistance in structural frames.
3. Brace a relatively weak structure against a more substantial one.
4. Install tension ties in walls that run from roof to foundation to improve roof anchorage.
5. Provide x-bracing in the plane of a roof to improve diaphragm stiffness and thus achieve a better distribution of lateral load to rigid frames, braced frames, or shear walls.

To prevent breach of building envelope or to reduce the consequences of missile perforation, the following general suggestions are presented:

1. Install additional fasteners to improve cladding anchorage.
2. Provide interior barriers around sensitive equipment or rooms containing hazardous materials.
3. Eliminate windows or cover them with missile-proof grills.
4. Place missile-proof barriers in front of doors or windows.
5. Replace ordinary overhead doors with heavy-duty ones that will resist design wind loads and provide missile impact resistance. The tracks must be capable of resisting the postulated loads.

Each building will likely have special situations that need attention. Consultants who evaluate existing facilities should have experience and knowledge of the behavior of buildings and other structures when subjected to wind loads.

6 Flood Design and Evaluation Guidelines

6.1 Flood Design Overview

The flood design and evaluation guidelines seek to ensure that DOE facilities satisfy the performance goals described in Chapter 2. The guidelines, which are applicable to new and existing construction, consider the design of DOE facilities for regional flood hazards (i.e., river flooding) and local precipitation that effects roof design and site drainage. They establish for floods the; design basis flood (DBFL) that must be considered, alternative design strategies, and criteria for the design of civil engineering systems (e.g., structures, site drainage, roof design, etc.). The provisions of DOE Order 6430.1A provide detailed guidance and design criteria for the design of civil engineering systems (i.e., building design, site drainage, roof drainage and structural design). Criteria for the design of facilities for the effects of the DBFL (i.e., hydrostatic loads, runoff due to local precipitation) as specified in DOE Order 6430.1A are adopted in these guidelines. These guidelines establish the DBFL and evaluation that must be considered to satisfy the performance goals for each usage category. Additional design criteria are specified in these guidelines as required. For existing facilities that may not meet the design criteria, evaluation guidelines are provided to assess whether the performance goals are satisfied.

Table 6-1 shows the guidelines recommended for each usage category in terms of the hazard input, hazard-annual probability, design requirements, and emergency operation plan requirements.

Evaluation of the flood design for a facility consists of:

1. the DBFL for each flood hazard as defined by the annual hazard exceedance probability and applicable combinations of flood hazards,
2. evaluate site and/or facility conditions (e.g., site drainage, facility location),
3. develop a flood design strategy to satisfy the guidelines and performance goals (e.g., build above the DBFL, harden the facility), and
4. design civil engineering systems (e.g., buildings, site drainage) as specified by the applicable design criteria.

Each of these areas is briefly described in the following subsections.

Table 6-1 Flood Guidelines Summary

Usage Category	General Use	Important or Low Hazard	Moderate Hazard	High Hazard
Flood Hazard Input (DBFL)	Flood insurance studies or equivalent input and Table 6-2 combinations	Flood insurance studies or equivalent input and Table 6-2 combinations	Site probabilistic hazard analysis and Table 6-2 combinations	Site probabilistic hazard analysis and Table 6-2 combinations
Hazard Annual Probability	$2x10^{-3}$	$5x10^{-4}$	$1x10^{-4}$	$1x10^{-5}$
Design Criteria	DOE 6430.1A or applicable criteria (e.g., governing local regulations, UBC) shall be used for building design for flood loads (i.e., load factors, design allowables), roof design and site drainage. The design of flood mitigation systems (i.e., levees, dams, etc.) shall comply with applicable standards as referred to in these guidelines.			
Emergency Operation Plans	Required to evacuate on-site personnel if facility is impacted by the DBFL	Required to evacuate on-site personnel and to secure vulnerable areas if site is impacted by the DBFL	Required to evacuate on-site personnel not involved in essential operations. Provide for an extended stay for personnel who remain. Procedures must be established to secure the facility during the flood such that operations may continue following the event.	

6.1.1 Design Basis Flood (DBFL)

As part of the flood hazard assessment that is performed for a site, the sources of flooding (i.e., rivers, lakes, local precipitation) and the individual flood hazards (e.g., hydrostatic forces, ice pressure, hydrodynamic loads) are identified. An individual site or facility may be impacted by multiple sources of flooding and flood hazards. For example, many DOE sites must consider the hazards associated with river flooding. In addition, all sites must design a site-drainage system to handle the runoff due to local precipitation. Events that contribute to potential river flooding, such as spring snowmelt, upstream-dam failure, etc., must be considered as part of a probabilistic flood hazard analysis for the site. Thus, the term, DBFL, should be understood to mean that multiple flood hazards may be included in the design. As a result, the site and individual facilities must be evaluated for each flood hazard that may occur.

The DBFL for a facility for each flood hazard (e.g., river flooding, local precipitation) is defined in terms of:

1. peak-hazard level (e.g., flow rate, depth of water) corresponding to the mean, hazard annual exceedance probability (see Table 6-1),
2. combinations of flood hazards (e.g., river flooding and wind-wave action) (see Table 6-2), and
3. corresponding loads associated with the DBFL peak hazard level and applicable load combinations (e.g., hydrostatic and/or hydrodynamic forces, debris loads).

The first two items are determined as part of the site probabilistic hazard assessment. Flood loads must be assessed for the DBFL on a facility-by- facility basis.

Table 6-2 defines the design basis events that must be considered. For example, if river flooding is the primary source of flooding, wind waves must be considered as part of the DBFL, as defined in Table 6-2. If the hazard annual probability of exceedance for a primary flood hazard is less than the design basis hazard annual probability for a facility category (see Table 6- 1), it need not be considered as a design basis event. For instance, if the hazard annual probability of exceedance for General-Use facilities is $2x10^{-3}$ per year, failure of an upstream dam need not be considered if it is demonstrated that the mean probability of flooding due to dam failure is less than $2x10^{-3}$. For purposes of design, the event combinations in Table 6-2 are assumed to be perfectly correlated.

Table 6-2 Design Basis Flood Events

Primary Hazard	Case No.	Event Combinations*
River Flooding	1	Peak flood elevation. Note: The hazard analysis for river flooding should include all contributors to flooding, including releases from upstream dams, ice jams, etc. Flooding associated with upstream-dam failure is included in the dam failure category.
	2	Wind-waves corresponding to the 2 year wind acting in the most favorable direction (Ref. 55), coincident with the peak flood.
	3	Ice forces (Refs. 55 and 56) and Case 1.
	4	Evaluate the potential for erosion, debris, etc.
Dam Failure	1	All modes of dam failure must be considered (i.e., overtopping seismically induced, random structural failures, upstream dam failure, etc.).
	2	Wind-waves corresponding to the 2 year wind acting in the most favorable direction (Ref. 55), coincident with the peak flood.
	3	Evaluate the potential for erosion, debris, etc.
Local Precipitation	1	Flooding based on the site runoff analysis shall be used to evaluate the site drainage system and flood loads on individual facilities.
	2	Ponding on roof to a maximum depth corresponding to the level of the secondary drainage system.
	3	Rain and snow, as specified in ANSI A53.1-1982 (Ref. 14).
Storm Surge, Seiche (due to hurricane, seiche, squall lines, etc.)	1	Tide effects corresponding to the mean high tide above the MLW** (if not included in the hazard analysis).
	2	Wave action amd Case 1. Wave action should include static and dynamic effects and potential for erosion (Ref. 55).
Levee or Dike Failure	1	Should be evaluated as part of the hazard analysis if overtopping and/or failure occurs.
Snow	1	Snow and drift roof loads as specified in ANSI A58.1-1982 (Ref. 14).
Tsunami	1	Tide effects corresponding to the mean high tide above the MLW (if not included in the hazard analysis).

* Events are added to the flood level produced by the primary hazard.

** MLW - Mean Low Water

6.1.2 Flood Evaluation Process

The following describes the basic steps involved in the evaluation of DOE facilities. The procedure is general and applies to new and existing construction. The process is oriented toward the evaluation of individual facilities. However, at some sites, it may be possible to construct all or a number of facilities above the DBFL for the highest category facility (i.e., the highest flood level), thus satisfying the guidelines.

The flood evaluation process is illustrated in Figure 6-1. It is divided into the consideration of regional flood hazards and local precipitation. For new construction, design practice (see Section 6.1.3) is to construct facilities above the DBFL, thus avoiding the flood hazard and eliminating the consideration of flood loads as part of the facility design. The design of the site-drainage system and structural systems (i.e., roofs) for local precipitation must be adequate to prevent flooding that may damage a facility or interrupt operations to the extent that the performance goals are not satisfied.

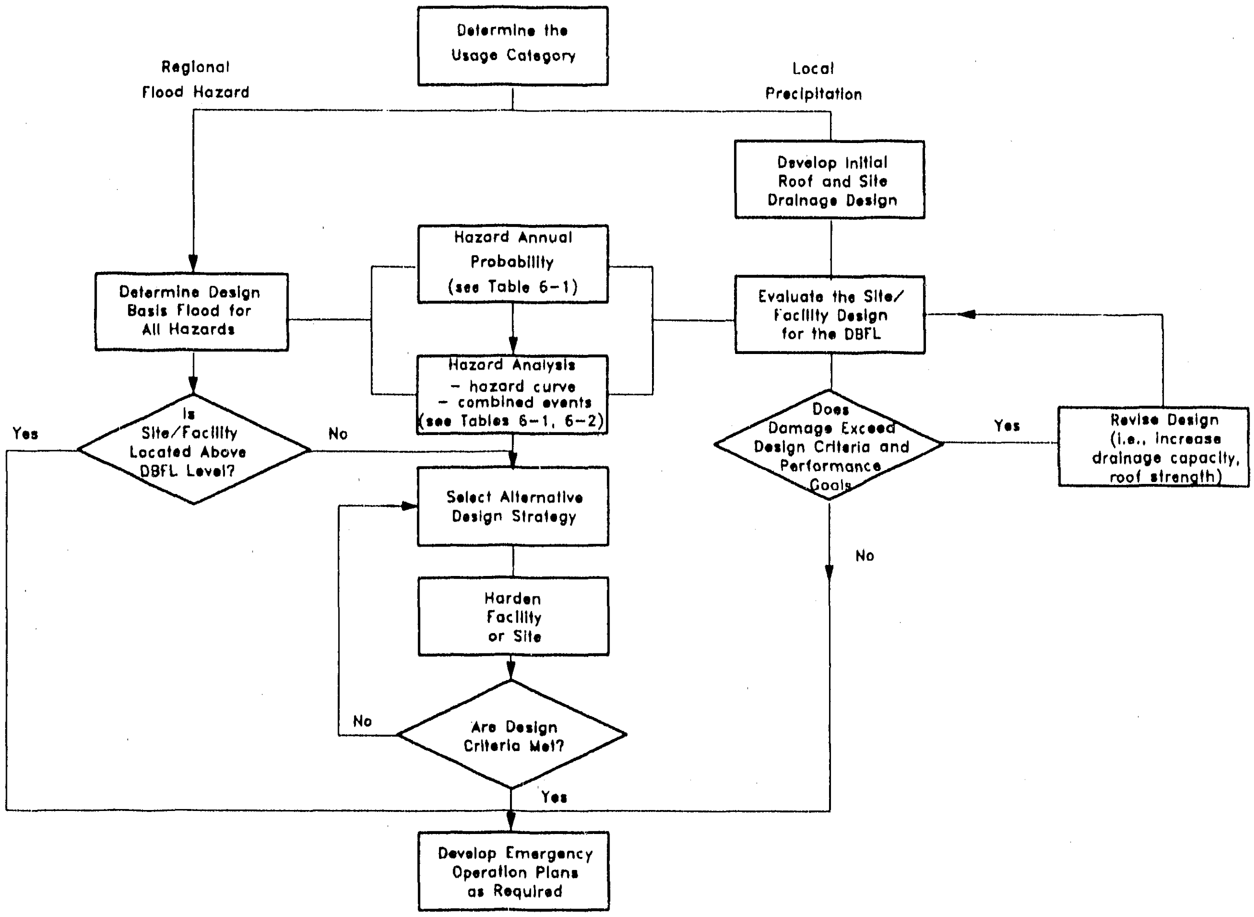


Figure 6-1. Flood Evaluation Process

To perform the flood evaluation for a facility, the results of a flood screening analysis (as a minimum) or a probabilistic flood hazard analysis should be available (see discussion in Section 1.4). The steps in the flood evaluation process include:

1. Determine the facility category (see Chapter 2).

<u>Evaluation for Regional Flood Hazards</u>

2. Determine the DBFL for each type or source of flooding (see Tables 6-1 and 6-2). The assessment of flood loads (e.g., hydrostatic and hydrodynamic loads) or other effects (e.g., scour, erosion) are determined as required for each facility.
3. For new construction, situate the facility above the DBFL, if possible. If this cannot be done, proceed to Step 4.
4. Develop a design approach to mitigate flood hazards that impact the facility. Options include hardening the facility and developing emergency operation plans to provide for occupant safety and to secure vulnerable areas. The flood hazard must be mitigated to a level such that the performance goals are met.
5. If the facility is located below the DBFL level (even if the facility has been hardened), emergency procedures should be provided to evacuate personnel and to secure the facility prior to the arrival of the flood (see Step 10).

<u>Evaluation for Local Precipitation</u>

6. Develop an initial site-drainage system and roof-system drainage plan and structural design based on the requirements and guidelines specified in DOE Order 6430.1A.
7. Evaluate the site-drainage system (considering man-made and natural watercourses) for the DBFL local precipitation for each type facility at the site. The analyst must determine whether local flooding occurs at a facility to an extent that the performance goals are not satisfied. If local flooding damages a facility or unsatisfactorily interrupts operations, a modification of the site-drainage system design is required (see Step 9).
8. Evaluate the facility roof drainage and structural design for the DBFL local precipitation. The structural design of the roof system must satisfy the design criteria for loads that occur due to ponding that results from clogged/blocked drains and snow and ice loads. If the design criteria for the roof is exceeded (i.e., deflection, stress allowables), the design must be revised (see Step 9).
9. If local precipitation produces levels of flooding (i.e., roof loads) that do not satisfy the performance goals (i.e., damage level due to inundation or exceedance of design criteria allowables), design modifications must be developed. The modifications must provide additional capacity (i.e., runoff capacity, additional strength) to satisfy the performance goals.
10. Develop emergency operation plans to provide for the safety of the facility personnel and to secure critical areas to satisfy performance goals.

In principle, buildings that fit into one category or another should be designed for different hazard levels because of the importance assigned to each. However, because floods have a common-cause impact on all buildings at or below a DBFL, the design basis for the most critical structure may govern the design for other buildings in proximity or, on the entire site. In this case, it may be feasible to harden a site (e.g., construct a levee system), thus protecting all facilities rather than an individual building. On the other hand, for many sites, it may be impractical to develop a design strategy that protects the entire site when facility locations vary substantially (i.e., they are at significantly different elevations or there are large spatial separations).

It is important to consider the possible interaction between buildings or building functions as part of the evaluation process. For example, if a High Hazard facility requires emergency electric power in order to maintain safety levels, buildings which house emergency generators and fuel should be designed for the DBFL for High Hazard facilities. In general, a systematic review of a site for possible common-cause dependencies is required. As an aid to the review, the analyst can develop a logic diagram that displays the functional dependencies and system interactions between operations housed in each building.

6.1.3 Flood Design Strategies

In practice, the basic design strategy for important, moderate or critical use facilities (excluding local precipitation), is to construct the facility above the applicable DBFL. In this way, it is not necessary to consider the effects of the DBFL in the design of structures or equipment. The flood guidelines have been established with this basic strategy in mind. Local precipitation is an exception since all sites must consider this hazard in the design of the site drainage, roof systems, etc.

Since it may not always be possible to construct a new facility above the DBFL level, alternate design strategies must be considered. The following lists the hierarchy of basic flood design strategies:

1. situate the facility above the DBFL level, or
2. harden the site or facility to mitigate the effects of the DBFL such that the performance goals are satisfied, and
3. establish emergency operation plans to safely evacuate employees and secure areas with hazardous, mission-dependent, or valuable materials.

If a facility is situated above the DBFL, the performance goals are readily satisfied. If a facility is located below the DBFL, alternatives can be considered to harden the facility or possibly the entire site against the effects of floods such that, the chance of damage and interruption of operations is acceptably low. In addition, emergency operation plans must establish the procedures that will be followed to identify the flood hazard that may occur at a site in a timely manner, provide for occupant safety, and secure areas that may be vulnerable to the effects of flooding. The implementation of emergency operation plans is not, in general, an alternative to satisfy the performance goals. While they are necessary to provide for occupant safety, generally, they will not adequately limit the level of damage and interruption to facility operations.

The strategy of hardening a facility or site and providing emergency operation plans is secondary to siting facilities above the DBFL level because some probability of damage does exist and facility operations may be interrupted. If it is determined that a facility will be impacted by the DBFL and thus must be hardened, the designer must determine the flood loads associated with the DBFL. The facility mitigative systems (i.e., exterior walls, flood-proof doors, etc.) must then be designed according to design requirements specified in DOE Order 6430.1A.

The evaluation of the site drainage system and roof design (i.e., drainage and structural capacity) differs somewhat from that for other flood hazards. First, all sites must design for the effects of local precipitation. Secondly, from the perspective of the facility performance goals, the adequacy of the site-drainage system is measured in terms of the impact of local flooding on facilities at the site. For example, the initial design of a site-drainage system may correspond to the 25-year rainfall, the minimum required by DOE Order 6430.1A. If the DBFL for a facility at the site corresponds to a $5x10^{-4}$ rainfall, the site-drainage system design clearly does not meet this criteria. However, at this point, the only conclusion that can be reached is that the system (i.e., storm sewers, etc.) will be filled to capacity. To determine the effect of the DBFL precipitation on the facility, a hydrologic evaluation must be performed to determine whether there is an adverse impact (i.e., damage, interruption of operations) on the facility to the extent that the performance goals are not satisfied. Based on an analysis that accounts for natural and man-made watercourses on site, roof drainage, etc., the analyst may conclude that flooding is limited to streets and parking lots. If this temporary inconvenience is not a problem, then it may be concluded that the design of the site-drainage system (i.e., for the 25-year rainfall) is adequate. On the other hand, if flooding does result in significant flood damage, appropriate measures would have to be taken to satisfy the performance goals. This may include increasing the capacity of the drainage system and/or hardening the facility against the effects of flooding caused by local precipitation.

6.2 DOE Flood Hazard Assessments

While the results of probabilistic hazard evaluations for seismic and wind phenomena are available for all of the DOE sites, comparable evaluations for flood hazards have not been performed. Flood-screening evaluations (i.e., preliminary flood analyses) have been performed for eight sites in the jurisdiction of the Albuquerque Operations Office. Also, a flood hazard assessment has been performed for the Hanford Project site. The results of these evaluations are summarized in Reference 3.

The objective of the probabilistic hazard evaluations for DOE sites is to assess the probability of events that have a low (less than 10^{-3} per year) probability of being exceeded. In the case of floods, facilities at DOE sites may not be exposed to extreme flood hazards. Because of topography, regional climate, or the location of sources of flooding in relation to a site, extreme flooding on-site may be precluded (with the exception of local precipitation). For existing facilities, design decisions may have resulted in all buildings being sited above possible flood levels. Consequently, in some cases, it may be apparent that floods do not pose a substantial hazard to facility operations. For these so-called "dry sites" (Ref. 55), it may be possible to demonstrate, without performing a detailed hazard assessment, that the design guidelines are satisfied. However, all sites must be designed for the runoff due to local precipitation.

The concept of a dry site, as used here, does not imply that a site is free of all sources of flooding (e.g., all sites are exposed at least to precipitation). Rather, a dry site is interpreted to mean that facilities (new or existing) are located high enough above potential flood sources such that a minimum level of analysis demonstrates that the design guidelines are satisfied.

To consider flood hazards at DOE sites, a two-phase evaluation process is being used. In the first phase, flood screening analyses are performed (Ref. 3). These studies provide an initial evaluation of the potential for flooding at a site. As part of the screening analysis, available hydrologic data and results of previous studies are gathered, and a preliminary assessment of the probability of extreme floods is performed. Results of the screening analysis can be used to assess whether flood hazards are extremely rare, thus providing a basis to conclude that performance goals are satisfied. For those sites with a potential for flooding and which have Moderate Hazard and High Hazard facilities, the second phase will be undertaken. This consists of a detailed probabilistic flood hazard assessment.

In estimating the probability of extreme floods, it is important that an uncertainty analysis be performed. The uncertainty analysis should consider the statistical uncertainty due to limited data and the uncertainty in the flood-evaluation models (e.g., choice of different statistical models, uncertainty in flood routing, etc.). Discussions of uncertainty assessments can be found in References 57 through 61.

6.3 Flood Design Guidelines for Each Usage Category

Unlike design strategies for seismic and wind hazards, it is not always possible to provide margin in the flood design of a facility. For example, the simple fact that a site is inundated (even if structural damage does not occur), may cause significant disruption (e.g., down time during the flood, clean-up). This may be an unacceptable risk in terms of the economic impact and disruption of the mission-dependent function of the site. In this case, there is no margin, as used in the structural sense, that can be provided in the facility design. Therefore, the facility must be kept dry and operations must not be interrupted. As a result, the annual probability of the DBFL corresponds to the performance goal probability of damage, since any exceedance of the DBFL results in consequences that are unacceptable.

The DBFL for General Use and Important or Low Hazard facilities can generally be estimated from available flood hazard assessment studies. These include: the results of flood-screening studies, flood-insurance analyses, or other comparable evaluations. For these facility types, it is not necessary that a detailed probabilistic hazard evaluation be performed, if the results of other recent studies are available and, if uncertainty in the hazard estimate is accounted for.

For Moderate and High hazard facilities, a comprehensive flood hazard assessment should be performed, unless the results of the screening analysis (see Ref. 61) demonstrate that the performance goals are satisfied.

6.3.1 General Use Facilities

The performance goal for General-Use facilities specifies that occupant safety be maintained and that the probability of severe structural damage be less than or about a 10^{-3} per year. For General-Use facilities, the DBFL corresponds to the hazard level whose mean-annual probability of exceedance is $2x10^{-3}$. In addition, event combinations that must be considered are listed in Table 6-2.

To meet the performance goal for this category, two requirements must be met: (1) the facility structural system must be capable of withstanding the forces associated with the DBFL, and (2) adequate flood warning time must be available to ensure that building occupants can be evacuated (i.e., 1 to 2 hours, Ref. 62). If the facility is located above the DBFL, then structural and occupant safety requirements are met.

Where the facility cannot be constructed above the DBFL level, an acceptable design can be achieved by:

1. Providing flood protection for the site or for the specific General Use facility, such that severe structural damage does not occur, and
2. Developing emergency procedures in order to secure facility contents above the design flood elevations in order to limit damage to the building to within acceptable levels and to provide adequate warning and evacuation capability to provide for the safety of building occupants.

For structural loads applied to roofs, exterior walls, etc., applicable requirements (e.g., DOE 6430.1A, Uniform Building Code (UBC); Refs. 7 and 8) provide standards for design that meet the performance goal for General-Use facilities.

6.3.2 Important or Low Hazard Facilities

The performance goal for Important or Low Hazard facilities is to limit damage and interruption of facility operations while also maintaining occupant safety. For these facilities, the DBFL is equal to the flood whose probability of exceedance is $5x10^{-4}$ per year plus the event combinations listed in Table 6-2. The results of flood-insurance studies (Ref. 63) routinely report the flood level corresponding to the $2x10^{-3}$ probability level. For purposes of establishing the DBFL for Important or Low Hazard Facilities, the results of these or equivalent studies can be extrapolated to obtain the flood with a mean-annual probability of $5.0x10^{-4}$ of being exceeded (if this result is not reported). This analysis must include the uncertainty in the hazard assessment in order to obtain an accurate estimate of the mean-annual probability level.

Facilities in this category should be located above the DBFL. For facilities that cannot be located above the DBFL, an acceptable design can be achieved by the same measures described for General-Use facilities. For Important or Low Hazard facilities whose critical elevation is below the DBFL, emergency procedures must be developed to mitigate the damage to mission- dependent components and systems. These procedures may include installation of temporary flood barriers, removal of equipment to protected areas, anchoring vulnerable items, or installing sumps or emergency pumps. As in the case of General Use facilities, design requirements in DOE Order 6430.1A should be used to incorporate flood loads in the building design.

6.3.3 Moderate Hazard Facilities

The performance goal for Moderate-Hazard facilities is continued function of the facility, including confinement of hazardous materials and occupant safety. Facilities in this category should be located above flood levels whose mean-annual probability of exceedance is 10^{-4}, including the event combinations shown in Table 6-2.

If Moderate-Hazard facilities cannot be constructed above the DBFL level, a design must be developed that provides continued facility operation. Depending on the strategy that is used to mitigate the flood (i.e., hardening the facility, building a levee to prevent flood encroachment), the design must mitigate the DBFL flood such that facility operations can continue. A higher level of protection is required for Moderate-Hazard facilities as compared to Low-Hazard or General-Use facilities. Whereas limited damage and interruption of operations may be acceptable for Low-Hazard facilities, for Moderate-Hazard facilities, the DBFL must be mitigated such that the flood does not impact operations.

The design of Moderate-Hazard facilities that may be impacted by the DBFL should be based on the loads (i.e., hydrostatic forces) and other hazards (i.e., debris) that occur at the facility. The design requirements in DOE Order 6430.1A should be used to incorporate flood loads in the building design or other structural systems. If mitigation systems such as watertight doors, sealants, etc., are used, manufacturer specifications should be applied. Section 6.4 describes design requirements for flood-mitigation systems such as levees, dikes, etc.

For facilities that may be impacted by the DBFL, emergency operation plans must be developed to evacuate personnel not involved in the emergency operation of the facility, secure hazardous materials, prepare the facility for possible extreme flooding and loss of power, and provide supplies for personnel for an extended stay on-site. Emergency procedures should be coordinated with the results of the flood hazard analysis, which provides input on the time variation of flooding, type of hazards to be expected, and their duration.

The use of emergency operation plans is not an alternative to hardening a facility to provide adequate confinement unless all hazardous materials can be completely removed from the site.

6.3.4 High Hazard Facilities

The performance goals for High-Hazard facilities are basically the same as for Moderate-Hazard facilities. However, a higher confidence is required that the performance goals are met. Facilities in this category should be located above flood levels whose mean-annual probability of exceedance is 10^{-5}, including combinations of events listed in Table 6-2.

6.4 Flood Design Practice for Facilities Below the DBFL Elevation

For structures located below the DBFL level, mitigation measures can be designed that provide an acceptable margin of safety. In practice, a combination of structural and non-structural measures (i.e., flood warning and emergency operation plans) are used. The design criteria for facilities that must consider flood loads are described in this section.

6.4.1 Flood Loads

To evaluate the effects of flood hazards, corresponding forces on structures must be evaluated. Force evaluations must consider hydrostatic and hydrodynamic effects, including the impact associated with wave action. In addition, the potential for erosion and scour and debris loads must be considered. The flood hazards that must be considered are determined in the flood hazard analysis. Good engineering practice should be used to evaluate flood loads (Refs. 64 through 69). The forces due to ice formation on bodies of water should be considered in accordance with DOE 6430.1A (Refs. 7, 56).

6.4.2 Design Requirements

Design criteria (i.e., for allowable stress or strength design, load factors, and load combinations) for loads on exterior walls or roofs due to rain, snow, and ice accumulation should follow requirements in DOE Order 6430.1A and ANSI A58.1-1982 as applicable or other criteria referred to herein (i.e., Ref. 8). The design criteria are to be used in conjunction with flood loads and effects derived from the facility DBFL (see Tables 6-1 and 6-2).

6.4.2.1 General Use and Important or Low Hazard Facilities

Facilities that are subject to flood loads should be designed according to provisions in DOE Order 6430.1A or governing local regulations. Design loads and load combinations are determined from the DBFL. Load factors specified in DOE Order 6430.1A, ANSI A58.1-1982, or governing local regulations shall be used.

6.4.2.2 Moderate and High Hazard Facilities

The exterior wall of buildings and related structures that are directly impacted by flood hazards should be constructed of reinforced concrete and designed according to ACI 349-85 (Ref. 30). Design loads and load combinations are determined from the DBFL. Load factors specified in DOE Order 6430.1A, ANSI A58.1-1982 or governing local regulations shall be used.

6.4.3 Site Drainage and Roof Design

For new construction, the stormwater-management system (i.e., street drainage, storm sewers, open channels, roof drainage) can be designed according to applicable procedures and design criteria specified in DOE Order 6430.1A (Ref. 7). As stated in DOE Order 6430.1A, governing local regulations must be considered in the design of the site-drainage system. The minimum design level for the stormwater management system is the 25-year, 6-hour storm.

Once the site and facility drainage design has been developed, it should be evaluated for the DBFL precipitation for each facility. The evaluation should consider the site-drainage area, natural and man-made watercourses, roof drainage, etc. The analysis shall determine the level of flooding that could occur at each facility. The analyst may choose to evaluate the site-drainage system for the highest category facility DBFL (as a limiting case). If the results of this analysis demonstrates that flooding does not compromise the site facilities, then it may be concluded that the site drainage is adequate. Note, that local flooding in streets, parking lots, etc., may occur due to the DBFL precipitation. This is acceptable if the effect of local flooding does not exceed the requirements of the performance goals. If, however, flooding does have an unacceptable impact, increased drainage capacity and/or flood protection will be required.

Building roof design should provide adequate drainage as specified by DOE Order 6430.1A (Ref. 7) and in accordance with local plumbing regulations. Secondary drainage (overflow) should be provided at a higher level and have a capacity at least that of the primary drain. Limitations of water depth on a roof specified by DOE 6430.1A or applicable local regulations apply. The rocf should be designed to consider the maximum depth of water that could accumulate if the primary-drainage system is blocked (Refs. 8 and 14).

Roof-drainage systems should be designed according to DOE Order 6430.1 or other governing local design regulations. The drainage system should be verified as part of the site analysis for the DBFL (discussed above). The design should then be verified for the DBFL precipitation. In the case of rainfall, a limiting check of the roof system structural design should be made. Ponding on the roof is assumed to occur to a maximum depth corresponding to the level of the seconda', drainage outlet system (i.e., assuming the primary system has clogged). As part of this evaluation, the deflection of the roof due to ponding must be considered. The design of the roof should be adequate to meet the DOE Order 6430.1A design allowables.

Design criteria for snow and rain-on-snow loads are defined in DOE Order 6430.1A and ANS A58.1-1982. In the design of roof systems for snow loads, the importance factor for General and Low Hazard facilities is 1.0 (Ref. 14). For Moderate and High Hazard facilities, an importance factor of 1.2 should be used.

6.4.4 Flood Protection and Emergency Operations Plans

For facilities that may be exposed to flood hazards (i.e., are located below the DBFL), a number of design alternatives are available. Depending on the flood hazards that a facility must withstand, various hardening systems may be considered. These include,

1. structural barriers (e.g., exterior building walls, floodwalls, watertight doors),
2. waterproofing (e.g., waterproofing exterior walls, watertight doors),
3. levees, dikes, seawalls, revetments, and
4. diversion dams and retention basins.

The design of structura' systems (i.e., exterior building walls) shall be developed in accordance with DOE Order 6430.1A. Waterproofing requirements are also given in DOE Order 6430.1A or by reference to applicable design standards. Guidelines for the design of earth structures such as levees, seawalls, etc., are provided in Reference 70. Guidance for the design of diversion dams and retention basins can be found in U.S. Army Corps of Engineers, U.S. Bureau of Reclamation, Soil Conservation Service reference documents (Refs. 62, 65, 69, 71).

Emergency operation plans are required in cases where the health and safety of on-site personnel must be provided for and where the facility must be secured to prevent damage or interruption of operations. The elements of the emergency operations are:

- flood recognition system - capability to identify impending floods and predicting their timing and magnitude.
- warning system - procedures and means to provide warning to those in the affected areas
- preparedness plan - establish the procedures, responsibility and capability (i.e., materials, transportation, etc.) to evacuate on-site personnel, secure vulnerable areas, etc.
- maintenance plan - establish a program to ensure that the emergency operation plan is up-to-date and operational.

Guidance for the development of the emergency operation plans can be found in emergency procedures developed for nuclear power plants, dams and local flood warning systems. Specific information on flood emergency procedures is provided in References 72 and 73.

6.5 Considerations for Existing Construction

For existing construction, a facility may not be situated above the DBFL, as defined by these guidelines. However, the fact that a facility is not constructed above the DBFL, does not necessarily imply that the guidelines are not satisfied. In this case, a facility must be reviewed to determine the level of flooding that can be sustained, while still satisfying the performance goals. This is referred to as the critical flood elevation (CFE). If the CFE is higher than the DBFL, then the guidelines are satisfied. This situation may not be unique for existing construction. For new construction, it may not be possible to situate all facilities above the DBFL, in which case, other design strategies must be considered.

For most facilities, there is a critical elevation which, if exceeded, would cause damage or disruption to the facility such that, the performance goal is not satisfied. The CFE may be located below grade (i.e., the base of the foundation) or because of the structural integrity of the building and the location of critical operations, it may be above the first-floor elevation.

Typically, the first-floor elevation or a below-grade elevation (i.e., foundation level) is assumed to be the critical elevation. However, based on a review of a facility, it may be determined that only greater flood depths would cause damage (e.g., critical equipment or materials may be located above the first floor). If the CFE for a facility exceeds the DBFL, then the performance goal is satisfied. If the CFE does not exceed the DBFL, options must be considered to harden the facility, change the usage category, etc.

For Moderate- and High-Hazard facilities, the performance goals require that little or no interruption of the facility operations should occur. This is an important consideration, since the assessment of the CFE must consider the impact of the flood on the facility operations (i.e, uninterrupted access) as well as on the building itself.

6.6 Probabilistic Flood Risk Assessment

In some cases, the need may arise for DOE or the site manager to perform a quantitative probabilistic flood risk assessment for a site. There may be a variety of reasons requiring a risk assessment. These considerations include:

1. Demonstration that the performance goals are satisfied.
2. Evaluation of alternative design strategies to meet the performance goals.
3. Detailed consideration of conditions at a site that may be complex, such as varying hydraulic loads (e.g., scour, high velocity flows), system interactions, secondary failures, or a potential for extraordinary health consequences.
4. A building is not reasonably incorporated in the four facility categories.

The objective of a risk assessment is to evaluate the risk of damage to systems important for maintaining safety and operating a critical facility. Risk calculations can be performed to evaluate the likelihood of damage to on-site facilities and public-health consequence. Procedures to perform probabilistic flood risk assessments are discussed in References 58-60, 74.

Appendix A

Commentary on Seismic Design and Analysis Guidelines

The overall approach employed for the seismic design and evaluation guidelines is discussed in Section A.1. The basis for selection of recommended hazard exceedance probabilities is described in Section A.2. Earthquake ground motion response spectra are discussed in Section A.3. The basic attributes of equivalent static force methods and dynamic analysis methods are described in Sections A.4 and A.5. Note that energy dissipation from damping or inelastic behavior is implicitly accounted for by the code formulas in equivalent static force methods. The means of accounting for energy absorption capacity of structures in dynamic analyses are discussed in Section A.6. In Section A.7, the basis for the specific seismic design and evaluation guidelines is described; this includes the inelastic demand-capacity ratios recommended for usage in the design and evaluation of Moderate and High Hazard facilities.

A.1 Basic Approach for Earthquake Design and Evaluation at Appropriate Lateral Force Levels

The performance of a DOE facility subjected to a natural phenomena hazard (earthquake, wind, or flood) depends not only on the level of hazard selected for design or evaluation, but also on the degree of conservatism used in the design or evaluation process. For instance, if one wishes to achieve less than about 10^{-4} annual probability of onset of loss of function, this goal can be achieved by using conservative design or evaluation approaches for a natural phenomena hazard that has a more frequent annual probability of exceedance (such as 10^{-3}), or it can be achieved by using median-centered design or evaluation approaches (i.e., approaches that have no intentional conservative or unconservative bias) coupled with a 10^{-4} hazard definition. At least for the earthquake hazard, the former alternate has been the most traditional. Conservative design or evaluation approaches are well-established, extensively documented, and commonly practiced. Median design or evaluation approaches are currently controversial, not well understood, and seldom practiced. Conservative design and evaluation approaches are utilized for both conventional facilities (similar to DOE category "General Use Facilities") and for nuclear power plants (equal to or more severe than DOE category "High Hazard Facilities"). For consistency with these other uses, the approach in this report recommends the use of conservative design and evaluation procedures coupled with a hazard definition consistent with these procedures.

The performance goals for General Use facilities are consistent with goals of conventional building codes for normal facilities; the performance goals for Important or Low Hazard facilities are consistent with goals of conventional building codes for important or essential facilities. For seismic design and evaluation of facilities, conventional building codes utilize equivalent static force methods except for very unusual or irregular facilities, for which a dynamic analysis method is employed. The performance goals for Moderate and High Hazard Facilities approach those used for nuclear power plants for which seismic design and evaluation is accomplished by means of dynamic analysis methods. For these reasons, the guidelines presented in this report recommend that lesser hazard facilities be evaluated by methods corresponding closely to conventional building codes and higher hazard facilities be evaluated by dynamic analyses.

The performance goals presented in Chapter 2 and the recommended hazard exceedance probabilities presented in Sections 4.2.2 and 4.2.3 are tabulated below for each usage category.

Usage Category	Performance Goal	Hazard Exceedance Probability	Ratio of Hazard to Performance Probability
General Use	$1x10^{-3}$	$2x10^{-3}$	2
Important or Low Hazard	$5x10^{-4}$	$1x10^{-3}$	2
Moderate Hazard	$1x10^{-4}$	$1x10^{-3}$	10
High Hazard	$1x10^{-5}$	$2x10^{-4}$	20

As shown above, the recommended hazard exceedance probabilities and performance goal exceedance probabilities are different. These differences indicate that conservatism must be introduced in the seismic behavior evaluation approach. In earthquake evaluation, there are many places where conservatism can be introduced, including:

1. Maximum design/evaluation ground acceleration.
2. Response spectra amplification.
3. Damping.
4. Analysis methods.
5. Specification of material strengths.
6. Estimation of structural capacity.
7. Load factors.
8. Importance factors.

9. Limits on inelastic behavior.
10. Soil-structure interaction.
11. Effective peak ground motion.
12. Effects of a large foundation or foundation embedment.

For the earthquake evaluation guidelines presented in this appendix, conservatism is intentionally introduced and controlled by specifying (1) hazard exceedance probabilities, (2) load factors, (3) importance factors, (4) limits on inelastic behavior, and (5) conservatively specified material strengths and structural capacities. Load and importance factors have been retained for the evaluation of General Use and Important or Low Hazard facilities because the 1988 UBC approach (which includes these factors) is followed for these categories. These factors are not used in general dynamic analyses of facilities or in Reference 9, and thus they were not used for the evaluation of Moderate and High Hazard facilities by dynamic analysis. Material strengths and structural capacities specified for Moderate and High Hazard facilities correspond to ultimate strength code-type provisions (i.e., ACI 318-83 for reinforced concrete, LRFD, AISC Part 2 or UBC Sec. 2721 for steel). Material strengths and structural capacities specified for General Use and Important or Low Hazard facilities correspond to either ultimate strength or allowable stress code-type provisions. It is recognized that such provisions introduce conservatism. In addition, it is acceptable by these guidelines to use peak ground accelerations from Reference 1 as the input earthquake excitation at the foundation level of facilities. As discussed in Sections 4.4.1 and 4.4.2, significant additional conservatism can be introduced if considerations of effective peak ground motion, soil-structure interaction, and effects of large foundation or foundation embedment are ignored.

The seismic design and evaluation guidelines presented in Section 4.2 are consistent from category to category. The 1988 UBC provisions (Reference 8) for General Use facilities are the baseline for the guidelines for all categories. The differences in seismic evaluation guidelines among categories in terms of load and importance factors, limits on inelastic behavior, and other factors as described in Section 4.2, and illustrated in Table 4-2, are summarized below:

1.	General Use and Important or Low Hazard	Only hazard exceedance probability and importance factor differ. All other factors are held the same.
2.	Important or Low Hazard and Moderate Hazard	Load factors, importance factors, damping, and limits on inelastic behavior differ. All other factors are essentially the same, although static force evaluation methods are used for Important or Low Hazard facilities and dynamic analysis is used for Moderate Hazard facilities.
3.	Moderate and High Hazard	Hazard exceedance probability and limits on inelastic behavior differ. All other factors are held the same.

The different load factors, importance factors, limits on inelastic behavior, and damping making up the seismic design and analysis guidelines for each usage category result in facilities in each category having a different inelastic demand (i.e., the value, D, computed as shown in Sections 4.2.2 and 4.2.3, which is compared to ultimate capacity to assess facility adequacy). Larger demand (i.e., required capacity) values result for more hazardous categories; this is indicative of the greater conservatism and reduced probability of damage or loss of capability to function associated with the higher hazard categories.

A.2 Earthquake Hazard Annual Exceedance Probabilities

Historically, non-Federal Government General Use and Essential or Low Hazard facilities located in California, Nevada, and Washington have been designed for the seismic hazard defined in the Uniform Building Code. Other regions of the U.S. have generally used either some version of the UBC seismic hazard definition or else have ignored seismic design. Past UBC seismic provisions (1985 and earlier) are based upon the largest earthquake intensity that has occurred in a given region during about the past 200 years. These provisions do not consider the probability of occurrence of such an earthquake and thus do not make any explicit use of a probabilistic seismic hazard analysis. However, within the last 15 years there has been considerable interest in developing a national seismic design code. Proponents have suggested that a seismic design code would be more widely accepted if the seismic hazard provisions of this code were based upon a consistent uniform annual probability of exceedance for all regions of the U.S. Several probabilistic-based seismic hazard provisions have been proposed (Refs. 9, 76, and 77). A probabilistic-based seismic zone map was recently incorporated into the 1988 *Uniform Building Code* (Ref. 8). Canada has adopted this approach (Ref. 13). The suggested annual frequency of exceedance for the design seismic hazard level differs somewhat between proposed codes, but all lie in the range of 10^{-2} to 10^{-3}. For instance, UBC 1988 (Ref. 8) and ATC-3 (Ref. 76) have suggested that the design seismic hazard level should have about a 10 percent frequency of exceedance level in 50 years which corresponds to an annual exceedance frequency of about $2x10^{-3}$. The Canadian building code used $1x10^{-2}$ as the annual exceedance level for their design seismic hazard definition. The Department of Defense (DOD) tri-services seismic design provisions for essential buildings (Ref. 9) suggests a dual level for the design seismic hazard. Facilities should remain essentially elastic for seismic hazard with about a 50 percent frequency of exceedance in 50 years or about a $1x10^{-2}$ annual exceedance frequency, and they should not fail for a seismic hazard which has about a 10 percent frequency of exceedance in 100 years or about $1x10^{-3}$ annual exceedance frequency.

On the other hand, nuclear power plants are designed so that safety systems do not fail if subjected to a safe shutdown earthquake (SSE). The SSE generally represents the expected ground motion at the site either from the largest historic earthquake within the tectonic province within which the site is located or from an assessment of the maximum earthquake potential of the appropriate tectonic structure or capable fault closest to the site. The key point is that this is a deterministic definition of the design SSE. Recent probabilistic hazard studies (e.g., Ref. 78) have indicated that for nuclear plants in the eastern U.S., the design SSE level generally corresponds to an estimated annual frequency of exceedance of between 10^{-3} and 10^{-4}. Also, during the last 10 years, considerable interest has developed in attempting to estimate the seismic risk of these nuclear power plants in terms of annual probability of seismic-induced core melt or risk to the public of early fatalities and latent cancer. Many studies have been conducted on the seismic risk of individual nuclear power plants. Because those plants are very conservatively designed to withstand the SSE, these studies have indicated that the seismic risk is acceptably low (on the order of about 10^{-5} annual probability of seismic-induced core damage) when such plants are designed for SSE levels with a mean annual frequency of exceedance between 10^{-3} and 10^{-4} (Refs. 15, 16, 17, 18, and 19).

With this comparative basis for other facilities, it is judged to be consistent and appropriate to define the seismic hazard for DOE facilities as follows:

Category	Earthquake Hazard Annual Exceedance Probability
General Use	$2x10^{-3}$
Important or Low Hazard	$1x10^{-3}$
Moderate Hazard	$1x10^{-3}$
High Hazard	$2x10^{-4}$

These hazard definitions are appropriate as long as the seismic design or evaluation of the facility for these hazards is conservatively performed. The level of conservatism of the evaluation for these hazards should increase as one goes from General Use to High Hazard facilities. The conservatism associated with General Use and Important or Low Hazard categories should be consistent with that contained in the UBC (Ref. 8) or ATC-3 (Ref. 76) for normal or essential facilities, respectively. The level of conservatism in the seismic evaluation for High Hazard facilities should approach that used for nuclear power plants when the seismic hazard is designated as shown above. The criteria contained in this report follow the philosophy of a gradual reduction in the annual exceedance probability of the hazard coupled with a gradual increase in the conservatism of the evaluation procedure as one goes from a General Use to a High Hazard facility.

A.3 Earthquake Ground Motion Response Spectra

Design/evaluation earthquake response spectra generally have the shape shown in Figures 4-1, 4-3, and 4-4 (Chapter 4). The design/evaluation spectrum shape is similar to that for an actual earthquake except that peaks and valleys that occur with actual earthquake spectra are smoothed out. Also, design/evaluation spectra typically include motions from several potential earthquakes such that they are broader in frequency content than spectra computed for actual earthquake ground motion. Such simplified spectral shapes are necessary in order to provide a practical input for seismic analyses.

Spectral amplication depends strongly on site conditions. For this reason, it would generally be expected that response spectra to be used for the design or evaluation of hazardous DOE facilities would be evaluated from site-specific geotechnical studies. There is a very good discussion on the development of response spectra from site-specific studies and other approaches in Reference 9. Alternatively, response spectra for DOE sites are available for use from Reference 1. Reference 1 spectra were developed from general site conditions and not from a site-specific geotechnical study. Additional approaches available for estimating response spectra from general site conditions are described in References 24, 25, and 26. Any of these methods is acceptable for estimating input design/evaluation response spectra. Note that to meet the performance goals in Chapter 2 using the guidelines presented in Sections 4.2.2 and 4.2.3 (Chapter 4), median amplification reponse spectra should be used. Mean amplification spectra are a conservative approximation of median spectra.

The C factor in the 1988 UBC base shear equation (e.g., Equation 4-1 in Chapter 4) is approximately equivalent to spectral amplification for 5 percent damping, and the Z factor corresponds to the maximum ground acceleration such that ZC corresponds to a 5 percent damping earthquake response spectrum. For General Use and Important or Low Hazard facilities, earthquake loading is evaluated from Equation 4-1 in accordance with UBC seismic provisions with the exception that the ZC is determined from input design/evaluation response spectra as described in Section 4.2.2. ZC as given by 1988 UBC provisions is plotted as a function of both natural period and natural frequency on Figure A-1. Also, Figure A-1 includes a typical design/evaluation spectra.

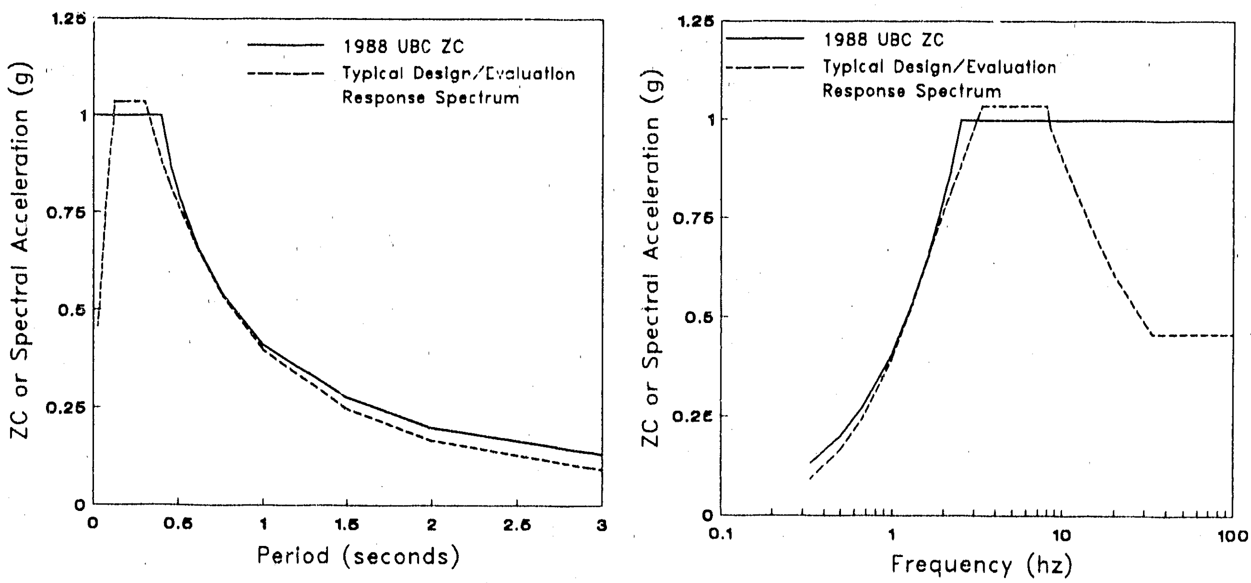


Figure A-1 Comparison of 1988 UBC ZC with Typical Design/Evaluation Response Spectrum

It is shown in Figure A-1 that an actual design evaluation spectrum differs significantly from the code coefficients, ZC, only in the low natural period region (i.e., less than about 0.125 seconds) or high natural frequency region (i.e., greater than about 8 hz). As a result, an adjustment must be made in the low period region in order to be conservative when the design/evaluation spectra are used along with other provisions of the code. The required adjustment to the design/evaluation spectra is to require that for fundamental periods lower than the period at which the maximum spectral acceleration occurs, ZC should be taken as the maximum spectral acceleration. This provision has the effect of making the design/evaluation spectra as shown in Figure 4-3 (Chapter 4), have a shape similar to that for ZC per the code provisions as shown in Figure A-1. In this manner, the recommended seismic evaluation approach for General Use and Important or Low Hazard facilities closely follows the 1988 UBC provisions while utilizing seismic hazard data from site-dependent studies.

In the design and evaluation guidelines presented in Section 4.2.3, for Moderate and High Hazard facilities, design/evaluation spectra as shown in Figure 4-4 are used for dynamic seismic analysis. However, in accordance with Reference 9, for fundamental periods lower than the period at which the maximum spectral acceleration occurs, spectral acceleration should be taken as the maximum spectral acceleration. For higher modes, the actual spectrum at all natural periods should be used in accordance with recommendations from Reference 9.

The basis for using the maximum spectral acceleration in the low period range by both the Reference 8 and 9 approaches is threefold: (1) to avoid being unconservative when using constant response reduction coefficients, R_W, or inelastic demand-capacity ratios, F_μ; (2) to account for the fact that stiff structures may not be as stiff as idealized in dynamic models; and (3) earthquakes in the eastern U.S. may have amplification extending to lower periods or higher frequencies than standard median design response spectra. Constant factors permit the elastically computed demand to exceed the capacity the same amount at all periods. Studies of inelastic response spectra such as those by Riddell and Newmark (Ref. 79), indicate that the elastically computed demand cannot safely exceed the capacity as much in the low period region as compared to larger periods. This means that lower inelastic demand-capacity ratios must be used for low period response if the actual spectra are used (i.e., the inelastic demand-capacity ratios are frequency dependent). Since constant demand-capacity factors are used herein, increased spectra as shown in Figure 4-4, must be used in the low period response region. Another reason for using increased spectral amplification at low periods is to assure conservatism for stiff structures. Due to factors such as soil-structure interaction, base mat flexibility, and concrete cracking, structures may not be as stiff as assumed. Thus, for stiff structures at natural periods below that corresponding to maximum spectral amplification, greater spectral amplification may be more realistic than that corresponding to the calculated natural period from the actual spectra. In addition, stiff structures that undergo inelastic behavior during earthquake ground motion soften (i.e., effectively respond at increased natural period) such that seismic response may be driven into regions of increased dynamic amplification compared to elastic response.

A.4 Static Force Method of Seismic Analysis

Seismic codes are based on a method that permits earthquake behavior of facilities to be translated into a relatively simple set of formulas. From these formulas, equivalent static seismic loads that may affect a facility can be approximated to provide a basis for design or evaluation. Equivalent static force methods apply only to relatively simple structures with nearly regular, symmetrical geometry and essentially uniform mass and stiffness distribution. More complex structures require a more rigorous approach to determine the distribution of seismic forces throughout the structure, as described in Section A.5.

Key elements of equivalent static force seismic evaluation methods are formulas that provide (1) total base shear; (2) fundamental period of vibration; and (3) distribution of seismic forces with height of the structure. These formulas are based on the response of structures with regular distribution of mass and stiffness over height in the fundamental mode of vibration. The 1988 UBC provisions (Reference 8) include, in their equation for total base shear, terms corresponding to maximum ground acceleration, spectral amplification as a function of natural period, a factor of conservatism based on the importance of the facility, and a reduction factor that accounts for energy absorption capacity. Very simple formulas estimate fundamental period by relating period to structure dimensions with coefficients for different materials or by a slightly more complex formula based on Rayleigh's method. This code defines the distribution of lateral forces of various floor levels. In addition, a top force is introduced to accommodate the higher modes by increasing the upper story shears where higher modes have the greatest effect. The overturning moment is calculated as the static effect of the forces acting at each floor level. Story shears are distributed to the various resisting elements in proportion to their rigidities, considering diaphragm rigidity. Increased shears due to actual and accidental torsion must be accounted for.

Seismic forces in members determined from the above approach and combined with forces due to other loadings are multiplied by a load factor and compared to code ultimate strength levels in order to evaluate whether or not the design is adequate for earthquake loads. In addition, deflections are computed from the lateral forces and compared to story drift limitations to provide for control of potential damage and overall structural frame stability from P-delta effects.

A.5 Dynamic Seismic Analysis

As mentioned previously, complex irregular structures cannot be evaluated by the equivalent static force method because the formulas for seismic forces throughout the structure would not be applicable. For such structures, more rigorous dynamic analysis approaches are required. In addition, for very important or highly hazardous facilities, such as the Moderate or High Hazard categories, it is recommended that the equivalent static force method not be used except for very simple structures. Dynamic analysis approaches lead to a greater understanding of seismic structural behavior; these approaches should generally be utilized for more hazardous facilities.

An analysis is considered dynamic if it recognizes that both loading and response are time-dependent and if it employs a suitable method capable of simulating and monitoring such time-dependent behavior. In this type of analysis, the dynamic characteristics of the structure are represented by a mathematical model. Input earthquake motion can be represented as a response spectrum or an acceleration time history.

The mathematical model describes the stiffness and mass characteristics of the structure as well as the support conditions. This model is described by designating nodal points that correspond to the structure geometry. Mass in the vicinity of each nodal point is typically lumped at the nodal point location in a manner that accounts for all of the mass of the structure and its contents. The nodal points are connected by elements that have properties corresponding to the stiffness of the structure between nodal point locations. Nodal points are free to move (called "degrees of freedom") or are constrained from movement at support locations. Equations of motion equal to the total number of degrees of freedom can be developed from the mathematical model. Response to any dynamic forcing function such as earthquake ground motion can be evaluated by direct integration of these equations. However, dynamic analyses are more commonly performed by considering the modal properties of the structure.

For each degree of freedom of the structure, there are natural modes of vibration, each of which responds at a particular natural period in a particular pattern of deformation (mode shape). There are many methods available for computing natural periods and associated mode shapes of vibration. Utilizing these modal properties, the equations of motion can be written as a number of single degree-of-freedom equations by which modal responses to dynamic forcing functions such as earthquake motion can be evaluated independently. Total response can then be determined by superposition of modal responses. The advantage of this approach is that much less computational effort is required for modal superposition analyses than direct integration analyses because fewer equations of motion require solution. Many of the vibration modes do not result in significant response and thus can be ignored. The significance of modes may be evaluated from modal properties before response analyses are performed.

The direct integration or modal superposition methods calculate response by considering the motions applied and the responses computed using a time-step by time-step numerical dynamic analysis. When the input earthquake excitation is given in terms of response spectra (as is the case for the motions provided for design and evaluation of DOE facilities in Reference 1) the maximum structural response may be most readily estimated by the response spectrum evaluation approach. The complete response history is seldom needed for design of structures; maximum response values usually suffice. Because the response in each vibration mode can be modeled by single degree-of-freedom equations, and response spectra provide the response of single degree-of-freedom systems to the input excitation, maximum modal response can be directly computed. Procedures are then available to estimate the total response from the modal maxima that do not necessarily occur simultaneously.

A.6 Analytical Treatment of Energy Dissipation and Absorption

Earthquake ground shaking is a limited energy transient loading, and structures have energy dissipation and absorption capacity through damping and through hysteretic behavior during inelastic response. This section discusses simplified methods of accounting for these modes of energy dissipation and absorption in seismic response analyses.

Damping - Damping accounts for energy dissipation in the linear range of response of structures and equipment to dynamic loading. Damping is a term utilized to account for various mechanisms of energy dissipation during seismic response such as cracking of concrete, slippage at bolted connections, and slippage between structural and nonstructural elements. Damping is primarily affected by:

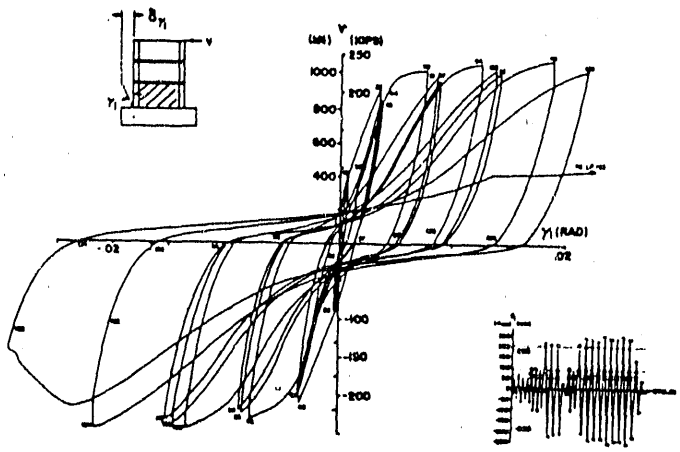
1. Type of construction and materials used.
2. The amount of nonstructural elements attached.
3. The earthquake response strain levels.

Damping increases with rising strain level as there are increased concrete cracking and internal work done within materials. Damping is also larger with greater amounts of nonstructural elements (interior partitions, etc.) in a structure that provide more opportunities for energy losses due to friction. For convenience in seismic response analyses, damping is generally assumed to be viscous in nature (velocity-dependent) and is so approximated. Damping is usually considered as a proportion or percentage of the critical damping value, which is defined as that damping in a system that would prevent oscillation for an initial disturbance not continuing through the motion.

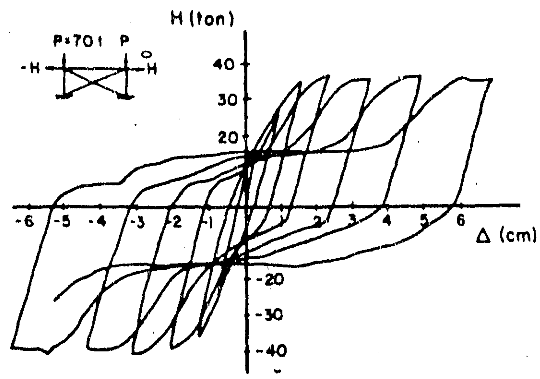
Table 4-6 (Chapter 4) reports typical structural damping values for various materials and construction (Refs. 9, 24, and 33). These values correspond to strains beyond yielding of the material, and, they are recommended for usage along with other provisions of this document for design or evaluation seismic response analyses of Moderate and High Hazard facilities. Post-yielding damping values are judged to be appropriate because facilities designed by these guidelines are intended to reach strains beyond yield level if subjected to the design/evaluation level earthquake ground motion, and such damping levels are consistent with other seismic analysis provisions based on Reference 9. For General Use and

Important or Low Hazard facilities, the guidelines recommend seismic evaluation by code-type equivalent static force methods but with the factors for maximum ground acceleration and spectral amplification in the total base shear formula taken from Reference 1. In this case, it is recommended that the 5 percent damped spectra be used for all General Use and Important or Low Hazard facilities to be consistent with building code evaluation methods. The spectral amplification factor in conventional building codes is based upon 5 percent damped spectral amplification.

Inelastic Behavior - Energy absorption in the inelastic range of response of structures and equipment to earthquake motions can be very significant. Figure A-2 shows that large hysteretic energy absorption can occur even for structural systems with relatively low ductility such as concrete shear walls or steel braced frames.



a. Shear force-distortion for concrete wall test (Ref. 80)



b. Lateral force-displacement for steel braced frame (Ref. 81)

Figure A-2 Cyclic Load-Deflection Behavior of Concrete Shear Walls and Steel Braced Frames

Generally, an accurate determination of inelastic behavior necessitates dynamic nonlinear analyses performed on a time-history basis. However, there are simplified methods to approximate nonlinear structural response based on elastic response spectrum analyses through the use of either spectral reduction factors or inelastic demand-capacity ratios. Spectral reduction factors and inelastic demand-capacity ratios permit structural response to exceed yield stress levels a limited amount as a means to account for energy absorption in the inelastic range. Based on observations during past earthquakes and considerable dynamic test data, it is known that structures can undergo limited inelastic deformations without unacceptable damage when subjected to transient earthquake ground motion. Simple linear analytical methods approximating inelastic behavior using spectral reduction factors and inelastic demand-capacity ratios are briefly described below.

1. ***Spectral reduction factors*** - Structural response is determined from a response spectrum dynamic analysis. The spectral reduction factors are used to deamplify the elastic response spectrum producing an inelastic response spectrum which is used in the analysis. The resulting member forces are compared to yield level stresses to determine structural adequacy.
2. ***Inelastic demand-capacity ratios*** - Structural response is determined from either response spectra or time history dynamic analyses with the input excitation consistent with the elastic response spectra. The resulting member forces are the demand on the structure which is compared to the capacity determined from member forces at yield stress level. If the permissible demand-capacity ratios are not exceeded, it would be concluded that the structure was adequate for earthquake loading.

The spectral reduction factors and inelastic demand-capacity ratios are evaluated based upon the permissible inelastic behavior level, which depends on the materials and type of construction. For ductile steel moment frames, relatively large reduction factors or demand-capacity ratios are used. For less ductile shear walls or braced frames, lower reduction values or demand-capacity ratios are employed. For more hazardous facilities, lower reduction factors or demand-capacity ratios may be used to add conservatism to the design or evaluation process, such that increased probability of surviving any given earthquake motion may be achieved.

The inelastic demand-capacity ratio approach is employed for design or evaluation of Moderate and High Hazard DOE facilities by these guidelines. This approach is recommended in the DOD manual for seismic design of essential buildings (Ref. 9). Inelastic demand-capacity ratios are called F_μ in this guidelines document. Inelastic demand-capacity ratios have the advantage over spectral reduction factors in that different values may be specified for individual elements of the facility instead of a single spectral reduction factor for the entire lateral force resisting system. As a result, critical elements such as columns or connections can be easily designed for larger forces by specifying a smaller inelastic demand-capacity ratio than for other elements.

Base shear reduction coefficients that account for energy absorption due to inelastic behavior and other factors are called R_W by the 1988 UBC provisions. R_W is more like a spectral reduction factor in that it is applied to the entire lateral force resisting system. There are special UBC provisions which require critical elements such as columns or connections to be designed for larger loads than those corresponding to the base shear equation using R_W. The UBC provisions are followed for General Use and Important or Low Hazard facilities by these guidelines.

Reduction coefficients, R_W, to be used for evaluation of General Use and Important or Low Hazard facilities and recommended inelastic demand-capacity ratios, F_μ, for Moderate and High Hazard facilities are presented in Table 4-7 (Chapter 4) for various structural systems. R_W factors given in the table are taken directly from Reference 8. The F_μ factors presented in Table 4-7 were established to approximately meet the performance goals for structural behavior of the facility as defined in Chapter 2 and as discussed in Section A.1. These factors are based both on values given in Reference 9 and on values calculated from code reduction coefficients in a manner based on the performance goals. The following section describes the detailed method for establishing the values of F_μ.

The code reduction coefficients, R_W, by the 1988 UBC approach and inelastic demand-capacity ratios, F_μ, by the DOD approach differ in the procedures that define permissible inelastic response under extreme earthquake loading. By the 1988 UBC approach, only the element forces due to earthquake loads are reduced by the reduction coefficient, R_W, in evaluating demand; while by the DOD approach, element forces due to both earthquake and dead and live loads are reduced by the inelastic demand-capacity ratio, F_μ, in evaluating demand. The effect of this difference is that the DOD approach may be less conservative for beam or brace members heavily loaded by dead and live loads. As a result, the guidlines presented in this document utilize F_μ in a manner more similar to the UBC in that only the elastically computed seismic response is reduced. This approach is more consistent with common seismic design/evaluation practice.

In addition, the approach for permitting inelastic behavior in columns subjected to both axial forces and bending moments differs between the 1988 UBC and DOD provisions. By the 1988 UBC approach, seismic axial forces and moments are both reduced by R_W, and then combined with forces and moments due to dead and live loads, along with an appropriate load factor. The resultant forces and moments are then checked in code-type interaction formulas to assess the adequacy of the column. By the DOD approach, column interaction formulas have been rewritten to incorporate the inelastic demand-capacity ratio (as shown in Figures 4-2 and 4-3 of Reference 9). By the DOD interaction formulas, the inelastic demand-capacity ratio is applied only to the bending moment, and axial forces are unaffected. In addition, the inelastic demand-capacity ratios are low compared to ratios for other types of members such as beams, as discussed in the next section, A.7. The DOD approach for columns is followed by these guidelines for Moderate and High Hazard facilities.

Several other factors may be noted about the inelastic demand-capacity ratios, F_μ:

1. Table 4-7 (Chapter 4) values assume that good seismic design detailing practice as discussed in Section 4.3 has been employed such that ductile behavior is maximized. If this is not the case (e.g., an existing facility constructed a number of years ago) lower inelastic demand-capacity ratios should be used instead of those presented in Table 4-7.
2. Table 4-7 values assume that inelastic behavior will occur in a reasonably uniformly manner throughout the lateral load-carrying system. If inelastic behavior during seismic response is concentrated in local regions of the lateral load carrying system, lower inelastic demand-capacity ratios should be used than those presented herein.
3. Inelastic demand-capacity ratios are provided in Table 4-7 for the structural systems described in References 8 and 9. For other structural systems not covered in the table, engineers must interpolate or extrapolate from the values given based on their own judgement in order to evaluate inelastic demand-capacity ratios that are consistent with the intent of these guidelines.

A.7 Basis for Seismic Guidelines for Important or Low Hazard, Moderate Hazard, and High Hazard Facilities

The performance goal for General Use facilities is probability of exceedance of 1×10^{-3} for significant structural damage to the facility. It is judged that this goal is approximately met by following the 1988 UBC provisions (Ref. 8) and with a probability of exceedance of 2×10^{-3} for the design/evaluation level peak ground acceleration. The facility demand for General Use facilities in accordance with the 1988 UBC provisions is based on maximum ground acceleration as described above, median spectral amplification at 5 percent damping, load factor of approximately 1.4, importance factor of unity, and reduction coefficients, as given in Table 4-7. This demand level is the baseline from which the design/evaluation demand level for other category facilities is determined as described below.

In the seismic design and analysis guidelines presented in Sections 4.2.2 and 4.2.3, the demand is compared to the ultimate capacity in order to assess the seismic adequacy of structures or equipment for all usage categories. While the ultimate capacities are the same for all categories, the demand is different for each usage category, with larger demand values being computed for more hazardous categories. The larger values are indicative of the greater conservatism and reduced probability of damage or loss of capability to function associated with the higher hazard categories. Demand provides a good means for comparing guidelines among the various categories. The demand for General Use and Important or Low Hazard facilities due to earthquake ground motion in accordance with the provisions in Section 4.2.2, can be approximated by:

$$D = LF\ I\ kZ\ DAF_{5\%}\ W / R_W \tag{A-1}$$

where:

- LF = load factor
- I = importance factor
  - I = 1.0 for General Use facilities
  - I = 1.25 for Important or Low Hazard facilities
- Z = peak ground acceleration appropriate for General Use facilities (i.e., 2x10-3 exceedance probability)
- k = a factor by which the peak ground acceleration differs from that corresponding to the General Use category
  - k = 1.0 for General Use facilities
  - k = 1.25 for Important or Low Hazard facilities

  In this section, peak ground acceleration for each category is expressed as kZ where Z is the General Use category peak ground acceleration and k is a factor accounting for the differences in peak ground accelerations among categories such that k = 1.0 for General Use facilities, k = 1.25 for Important or Low Hazard and Moderate Hazard facilities, and k = 2.0 for High Hazard facilities (k is the mean value of the ratio of peak ground acceleration at the exceedance probability for the category considered to peak ground acceleration at the General Use category exceedance probability determined from the Reference 1 seismic hazard curves).
- $DAF_{5\%}$ = dynamic amplification factor from the 5 percent ground response spectrum at the natural period of the facility
- W = total weight of the facility
- R_W = reduction coefficient accounting for available energy absorption (Ref. 8)

The demand for Moderate and High Hazard facilities due to earthquake ground motion in accordance with the provisions in Section 4.2.3 can be approximated by:

$$D = mDAF_{5\%}\, kZ\, W / F_{\mu} \tag{A-2}$$

where:
- m = a factor accounting for the difference in spectral amplification from 5 percent to the damping appropriate for the facility in accordance with Table 4-6
  e.g., $m = 0.9$ for 7 percent damping
  $m = 0.8$ for 10 percent damping
  $m = 0.7$ for 15 percent damping
  (m values are from References 9, 24, and 33)
- k = ground motion factor as defined above
  $k = 1.25$ for Moderate Hazard facilities
  $k = 2.0$ for High Hazard facilities
- F_{μ} = inelastic demand-capacity ratio (Table 4-7)

For any usage category, the demand, D, is compared to the code ultimate capacity, CAP, to determine if the facility is adequate for earthquake ground motion. Note that the demand as expressed by Equations A-1 and A-2 is only a general approximation. The demand for specific cases depends on the particular characteristics of the input ground motion and earthquake response spectra, as well as the effect of other loadings acting concurrently on the facility. However, these approximations for the demand are utilized to establish seismic design and analysis guidelines such that the performance goals described in Chapter 2 are approximately met.

The relationship between performance goal exceedance probability and facility demand is used to determine the specific values making up the seismic design and analysis guidelines such that the performance goals described in Chapter 2 can be approximately met for earthquake considerations. This relationship is the same as the relationship between hazard exceedance probability and peak ground acceleration, as determined from the seismic hazard curves. Differences in hazard exceedance probabilities correspond to differences in peak ground acceleration for which the facility is to be designed or evaluated for earthquake effects. These differences can be evaluated from the Reference 1 hazard curves by comparing ground acceleration levels at different hazard exceedance probabilities. From the Reference 1 data presented in Table 4-4, the mean ratio of peak ground acceleration for Low and Moderate Hazard categories to that for the General Use category is about 1.25 (standard deviation is 0.08), and the mean ratio of peak ground acceleration for the High Hazard category to that for the General Use category is about 2.0 (standard deviation is 0.21). As a result, a difference in probability of 2 should also correspond to a difference in demand (or required facility capacity) of about 1.25, and a difference in performance goal probability of 10 should correspond to a difference in demand of about 2.0.

The relationships described above between performance goal exceedance probability and earthquake demand have been used to develop the specific limits on inelastic behavior and other seismic provisions for Important or Low Hazard, Moderate Hazard, and High Hazard categories. The differences in performance goal probability and facility demand between Important or Low Hazard, Moderate Hazard, and High Hazard categories and that for the General Use category are tabulated below.

Category	Ratio of Performance Goal to that for General Use Facilities	Ratio of Earthquake Demand to that for General Use Facilities
Important or Low Hazard	2	1.25
Moderate Hazard	10	2.0
High Hazard	100	4.0

However, it should be noted that the performance goals for Important or Low Hazard, Moderate Hazard, and High Hazard categories are different from the General Use category in both probability level and in acceptable structural behavior. The goal for General Use facilities is to prevent structural damage to the extent that occupants might be endangered. The goal for the other categories is to maintain the capability of the facility to perform its function. As a result, the facility demand for Important or Low Hazard, Moderate Hazard, and High Hazard facilities should be even more different from General Use facilities than is indicated above. The 1988 UBC provisions suggest an importance factor of 1.25 for essential facilities (similar to the Important or Low Hazard category herein) to account for the difference in performance goals between normal use and essential facilities. It seems reasonable that if the demand levels for Important or Low Hazard, Moderate Hazard, and High Hazard categories were all increased by an additional factor of 1.25 greater relative to the General Use category, the differences in performance goal behavior would be fully accounted for.

In addition, because of the increased hazard associated with Moderate and High Hazard facilities, it is judged to be appropriate to provide some additional conservatism such that very high confidence of achieving the performance goal can be achieved. For this reason, the facility demand for Moderate and High Hazard categories is further increased by an additional factor of about 1.25 relative to other categories. More factors of conservatism have been incorporated into the guidelines for Moderate and High Hazard facilities than for General Use and Important or Low Hazard facilities in order to obtain higher levels of confidence of achieving the performance goal for these facilities, which contain hazardous materials and which may be sensitive to public opinion so that damage is especially undesirable. These additional factors have the effect of restricting inelastic behavior to be more closely elastic and of limiting drift of the facility such that damage is controlled in the event of a severe earthquake.

Hence, assuming the performance goal for General Use facilities is achieved for seismic design by following the 1988 UBC provisions, performance goals for other categories would be achieved if the earthquake demand levels for other categories were as follows:

$$D_{ILH} / D_{GU} = 1.25 \times 1.25 = 1.56$$
$$D_{MH} / D_{GU} = 1.25 \times 1.25 \times 2.0 = 3.13$$
$$D_{HH} / D_{GU} = 1.25 \times 1.25 \times 4.0 = 6.25$$

Note:
GU = General Use category
ILH = Important or Low Hazard category
MH = Moderate Hazard category
HH = High Hazard category

Based upon Equations (A-1) and (A-2), these differences in earthquake demand for Important or Low Hazard, Moderate Hazard, and High Hazard categories compared to that for the General Use category are given by the following equations (k and I for the General Use category are unity):

$$D_{ILH} / D_{GU} = I_{ILH} \, k_{ILH} = 1.56 \quad \text{(A-3)}$$
$$D_{MH} / D_{GU} = m \, k_{MH} \, R_W / (LF \, F_{\mu-MH}) = 3.13 \quad \text{(A-4)}$$
$$D_{HH} / D_{GU} = m \, k_{HH} \, R_W / (LF \, F_{\mu-HH}) = 6.25 \quad \text{(A-5)}$$

Note that using an importance factor of 1.25 for Important or Low Hazard facilities combined with a hazard exceedance probability which is one-half that for General Use facilities is approximately equivalent to an importance factor of slightly more than 1.5 for Important or Low Hazard facilities if the hazard exceedance probabilities were the same for both categories as shown above. Hence, the guidelines presented herein for Important or Low Hazard facilities are somewhat more conservative than the 1988 UBC provisions for essential or hazardous facilities.

By these seismic design and analysis guidelines, Moderate and High Hazard facilities are to be evaluated by elastic dynamic analysis. However, the elastically computed demand on the facility is permitted to exceed the capacity of the facility as a means of permitting limited inelastic behavior in good structural systems with detailing for ductile behavior. The amount that the elastic demand can exceed the capacity is the inelastic demand-capacity ratio, F_μ. Values for inelastic demand-capacity ratio, F_μ, when used with the seismic guidelines described in Section 4.2.3, assure that the performance goals presented in Chapter 2 are approximately met. A means of estimating Fu values that approximately meet the performance goals is described below.

Expressing the demand equations, (A-4) and (A-5) in general terms, the ratio of the demand for Moderate and High Hazard categories to that for the General Use category is:

$$\frac{D_{MH\,or\,HH}}{D_{GU}} = \frac{(k)(m)R_w}{(LF)F_\mu} = RATIO \qquad \text{(A-6)}$$

Where: k = 1.25 for Moderate Hazard
k = 2.0 for High Hazard
m = 0.9 for steel (7% damping)
m = 0.8 for concrete (10% damping)
m = 0.75 for masonry (12% damping)
m = 0.7 for wood (15% damping)
LF = 1.3 for steel
LF = 1.4 for concrete And Masonry
LF = 1.5 for wood
RATIO = 3.13 for Moderate Hazard
RATIO = 6.25 for High Hazard

Equation (A-6) may be solved for inelastic demand-capacity ratio, F_μ, as follows:

$$F_\mu = \frac{(k)(m)R_w}{(LF)(RATIO)} \qquad \text{(A-7)}$$

Example calculations of F_μ for Moderate and High Hazard steel moment frames using Equation (A-7) are shown below:

MODERATE HAZARD	HIGH HAZARD
k = 1.25 m = 0.9 R_W = 12 LF = 1.3 RATIO = 3.13	k = 2.0 m = 0.9 R_W = 12 LF = 1.3 RATIO = 6.25
$F_\mu = \frac{(1.25)(0.9)(12)}{(1.3)(3.13)} = 3.32$	$F_\mu = \frac{(2.0)(0.9)(12)}{(1.3)(6.25)} = 2.66$
F_μ = 2.5 IN DOD MANUAL F_μ = 3.0 IN GUIDELINES	F_μ = 2.0 IN DOD MANUAL F_μ = 2.5 IN GUIDELINES

Example calculations of F_μ for Moderate & High Hazard concrete shear walls in accordance with Equation (A-7) are shown below:

MODERATE HAZARD	HIGH HAZARD
k = 1.25 m = 0.8 R_W = 8 LF = 1.4 RATIO = 3.13	k = 2.0 m = 0.8 R_W = 8 LF = 1.4 RATIO = 6.25
$F_\mu = \frac{(1.25)(0.8)(8)}{(1.4)(3.13)} = 1.83$	$F_\mu = \frac{(2.0)(0.8)(8)}{(1.4)(6.25)} = 1.46$
F_μ = 1.5 IN DOD MANUAL F_μ = 1.7 IN GUIDELINES	F_μ = 1.25 IN DOD MANUAL F_μ = 1.4 IN GUIDELINES

Values of inelastic demand-capacity ratio, F_μ, from Equation (A-7) along with values from the DOD seismic provisions (Ref. 9), are presented in Table A-1 for many structural systems, materials, and construction. Note that these values are used differently in that the F_μ value in these guidelines is applied to response due to seismic loads only; while, by Reference 9, the inelastic demand-capacity ratio is applied to response due to total load.

The inelastic demand-capacity ratios from Equation (A-7) are based on the structural systems for which reduction coefficients, R_W, are given in the 1988 UBC provisions. These provisions give different reduction coefficients for bearing wall systems and for building frame systems in which gravity loads are carried by structural members that are different from the lateral force resisting system. In addition, the 1988 UBC provisions distinguish between different levels of detailing for moment resisting space frames, between eccentric and concentric braced frames, and between single and dual lateral load resisting systems. Consequently, Equation (A-7) results in more inelastic demand-capacity ratios than Reference 9, which does not make the above distinctions. On the other hand, DOD provisions give different inelastic demand-capacity ratios for individual members of the lateral load-resisting system, while 1988 UBC reduction coefficients refer to all members of the lateral load resisting system.

Table A-1 Inelastic Demand-Capacity Ratios from Equation A-7 and Reference 9

Structural System	R_W	F_μ* Moderate Hazard R9	F_μ* Moderate Hazard A-7	F_μ* High Hazard R9	F_μ* High Hazard A-7
MOMENT RESISTING FRAME SYSTEMS					
Columns	**	1.5	**	1.25	**
Beams					
Steel Special Moment Resisting Space Frame (SMRSF)	12	2.5	3.32	2.0	2.66
Concrete SMRSF	12	2.5	2.74	2.0	2.19
Concrete Intermediate Moment Frame (IMRSF)	7	-	1.60	-	1.28
Steel Ordinary Moment Resisting Space Frame	6	-	1.66	-	1.33
Concrete Ordinary Moment Resisting Space Frame	5	-	1.14	-	<1
SHEAR WALLS					
Concrete Bearing Walls	6	1.5	1.37	1.25	1.10
Concrete Non-Bearing Walls	8	1.5	1.83	1.25	1.46
Masonry Bearing Walls	6	1.25	1.29	1.1	1.03
Masonry Non-Bearing Walls	8	1.25	1.71	1.1	1.37
Plywood Bearing Walls	8	2.5	1.49	2.0	1.19
Plywood Non-Bearing Walls	9	2.5	1.68	2.0	1.34
Dual System, Concrete with SMRSF	12	1.5	2.74	1.25	2.19
Dual System, Concrete with Concrete IMRSF	9	1.5	2.06	1.25	1.65
Dual System, Masonry with SMRSF	8	1.25	1.71	1.1	1.37
Dual System, Masonry with Concrete IMRSF	7	1.25	1.50	1.1	1.20
CONCENTRIC BRACED FRAMES (BRACING CARRIES GRAVITY LOADS)					
Steel Beams	6	1.75	1.66	1.5	1.33
Steel Diagonal Braces	6	1.5	1.66	1.25	1.33
Steel Columns	6	1.5	1.66	1.25	1.33
Connections of Steel Members	6	1.25	1.66	1.0	1.33
Concrete Beams	4	1.75	<1	1.5	<1
Concrete Diagonal Braces	4	1.5	<1	1.25	<1
Concrete Columns	4	1.5	<1	1.25	<1
Connections of Concrete Members	4	1.25	<1	1.0	<1
Wood Trusses	4	1.75	<1	1.5	<1
Wood Columns	4	1.5	<1	1.25	<1
Connections in Wood (other than nails)	4	1.5	<1	1.25	<1
CONCENTRIC BRACED FRAMES (NO GRAVITY LOADS)					
Steel Beams	8	1.75	2.22	1.5	1.77
Steel Diagonal Braces	8	1.5	2.22	1.25	1.77
Steel Columns	8	1.5	2.22	1.25	1.77
Connections of Steel Members	8	1.25	2.22	1.0	1.77
Concrete Beams	8	1.75	1.83	1.5	1.46
Concrete Diagonal Braces	8	1.5	1.83	1.25	1.46
Concrete Columns	8	1.5	1.83	1.25	1.46
Connections of Concrete Members	8	1.25	1.83	1.0	1.46
Wood Trusses	8	1.75	1.49	1.5	1.19
Wood Columns	8	1.5	1.49	1.25	1.19
Connections in Wood (other than nails)	8	1.5	1.49	1.25	1.19
Beams and Diagonal Braces, Dual Systems					
Steel with Steel SMRSF	10	-	2.77	-	2.22
Concrete with Concrete SMRSF	9	-	2.06	-	1.65
Concrete with Concrete IMRSF	6	-	1.37	-	1.10
STEEL ECCENTRIC BRACED FRAMES (EBF)					
Columns	**	1.5	**	1.25	**
Beams and Diagonal Braces	10	-	2.77	-	2.22
Beams and Diagonal Braces, Dual System with Steel SMRSF	12	-	3.32	-	2.66

* Columns marked R9 are inelastic demand-capacity ratios directly from Reference 9. Columns marked A-7 are inelastic demand-capacity ratios calculated from Eq. (A-7).

** Values are the same as for beams and braces in this structural system

In general, there is reasonable agreement between the inelastic demand-capacity ratios from Reference 9 and those computed from Equation (A-7). For example, the DOD inelastic demand-capacity ratio for concrete shear walls is between the values for bearing and non-bearing walls from the equations. The DOD values are much lower than the values computed when shear walls act as a dual system with ductile moment-resisting space frames to resist seismic loads. The inelastic demand-capacity ratios for braced frames agree fairly well when the bracing carries no gravity loads. When bracing carries gravity loads, values for steel braced frames are in good agreement, but based on Equation (A-7), no inelastic behavior would be permitted for concrete braced frames or wood trusses. The DOD inelastic demand-capacity ratio for beams in a ductile moment-resisting frame fall between values from the equations for special and intermediate moment-resisting space frames (SMRSF and IMRSF as defined in Reference 8). However, the DOD values for columns are low compared to values derived from the code reduction coefficients.

Based upon the data presented in Table A-1, the inelastic demand-capacity ratios for seismic design and analysis of Moderate and High Hazard facilities presented in Table 4-7 (Chapter 4) have been selected. Because of the reasonable agreement with the DOD values from Reference 9 combined with the capability to distinguish between a greater number of structural systems, the values derived from Equation (A-7) have been given somewhat more weight for Table 4-7 than Reference 9 values. The only major exception is that Reference 9 values for columns have been utilized. Increased conservatism for columns as recommended in the DOD manual is retained. In addition, Reference 9 provides slightly different values for different members making up braced frames, and these differences are retained.

Appendix B

Commentary on Wind Design and Analysis Guidelines

Key points in the approach employed for the design and evaluation of facilities for extreme winds, hurricanes, and tornadoes are discussed in this appendix.

B.1 Wind Design Criteria

The philosophy employed in these guidelines is to establish common performance goals to be used as the basis for design or evaluation criteria for four categories of facilities. The performance goals are the same, whether the natural phenomena loading is from earthquakes, winds, or floods.

Natural phenomena hazard probability levels were then established along with sufficiently conservative design procedures to accomplish the performance goals. A consensus standard, ANSI/ANS 2.3-1983 (Ref. 82), provides guidelines for estimating tornado and extreme wind characteristics at nuclear power plant sites. This standard was <u>not</u> adopted in these guidelines by the Natural Phenomena Hazards Panel for the following reasons:

1. The document is intended for the siting of commercial nuclear power plants. Criteria is not necessarily appropriate for DOE facilities.
2. Site-specific hazard assessments were performed; it is not necessary or desirable to use regional criteria.
3. Although published in 1983, the ANS 2.3 document is based on 12 year old technology. Both the data and the methodologies used in the guidelines are more recent.
4. Although ANS 2.3 is a consensus document, it has <u>not</u> been approved by the U.S. Nuclear Regulatory Commission.

Instead of the ANSI/ANS 2.3 Standard, the uniform approach to wind design proposed in these guidelines is based on procedures in ANSI A58.1-1982 (Ref. 14). This consensus document is widely accepted as the most technologically sound wind load standard in the U.S.

The uniform approach to design for wind loads treats the three types of windstorms (straight wind, hurricanes, and tornadoes) the same. Since ANSI A58.1-1982 already treats straight winds and hurricanes the same, all that remains is to demonstrate the applicability of the approach to tornado resistant design. The procedure of ANSI A58.1-1982 is applied for determining wind pressures on buildings or net forces on other structures. The additional effects of atmospheric pressure change (APC) and missiles in tornadoes must also be considered.

The ANSI Standard addresses the physical characteristics of wind, including variation of wind speed with height and terrain roughness, effects of turbulence (through use of a gust response factor), and the variation of wind pressure over the surface of a building. Wind effects addressed in the ANSI Standard can be detected and measured on wind tunnel models and on full-sized buildings. Furthermore, evidence of the physical effects of wind found in wind tunnel and full-scale measurements are also found in windstorm damage. The appearance of damage from straight, hurricane, and tornado winds is very similar. The similarity suggests that wind pressure distribution on buildings is generally independent of the type of storm. One cannot look at a collapsed windward wall, or an uplifted roof, or damage at an eave or roof corner or wall corner and determine the type of windstorm that caused the damage. Table B-1 sites specific examples where the appearance of damage from the three types of storms is identical. The conclusion reached is that the proposed uniform approach is reasonable for estimating wind loads produced by straight winds, hurricane winds, or tornadoes.

Table B-1 Examples of Similar Damage from Straight Winds, Hurricanes, and Tornadoes

Type of Damage	Winds	Hurricanes	Tornadoes
Windward wall collapses inward	Mobile home, Big Spring, Texas 1973	A-frame, Hurricane Diana 1984	Metal building, Lubbock, Texas 1970
Leeward wall or side wall collapses outward	Warehouse, Big Spring, Texas 1973	Commercial building, Hurricane Celia 1970	Warehouse, Lubbock, Texas 1970
Roof	Warehouse, Joplin, Missouri 1973	Motel, Hurricane Frederick 1979	School, Wichita Falls, Texas 1979
Eaves	Mobile home, Big Spring, Texas 1973	A-frame, Hurricane Diana 1984	Metal building, Lubbock, Texas 1970
Roof corners	Residence, Arvine, California 1977	Residence, Hurricane Frederick 1979	Apartment building, Omaha, Nebraska 1975
Wall corners	Metal building, Arvine, California 1977	Flagship Motel, Hurricane Alicia 1983	Manufacturing building, Wichita Falls, Texas 1979
Internal pressure	Not applicable	Two-story office building, Cyclone Tracey, Darwin, Australia 1974	High school, Xenia, Ohio 1974

The tornado wind speeds in Reference 2 are gust speeds and must be converted to equivalent fastest-mile wind speeds before applying the ANSI Standard procedures. Conversions from tornado wind speeds to fastest-mile wind speeds are obtained from Table 5-2.

Table B-2 Relationship Between Tornado Wind Speeds and Fastest-Mile Wind Speeds

Tornado Wind Speed, mph (V_t)	Fastest-Mile Wind Speed, mph (V_{fm})
100	85
110	94
120	103
130	113
140	123
150	132
160	142
170	151
180	161
190	170
200	180
210	190
220	200
230	209
240	218
250	231
260	241
270	250
280	260
290	271
300	280

$$V_{fm} = 0.958\ V_t - 11.34$$

B.2 Load Combinations

The ratios of hazard probabilities to performance goal probabilities for the usage categories shown in Table B-3 are an approximate measure of the conservatism required in the design to achieve the performance goals. At the specified hazard probabilities which provide extreme wind loadings, the most conservatism is needed in the response evaluation and acceptance criteria for design of General Use and Important or Low Hazard facilities. Somewhat less conservatism is needed for Moderate and High Hazard facilities. The hazard probabilities specified for tornadoes are less than the performance goal probabilities. Hence, the performance goals are theoretically met with no conservatism in the design.

Table B-3 Ratio of Hazard Probabilities to Performance Goal Probabilities

Usage Category	Performance Goals	Hazard probability	Ratio of Hazard to Performance Probability
Extreme Winds			
General Use	10^{-3}	$2x10^{-2}$	20
Important or Low Hazard	$5x10^{-4}$	10^{-2} (1)	20
Moderate Hazard	10^{-4}	10^{-3}	10
High Hazard	10^{-5}	10^{-4}	10
Tornadoes			
Moderate Hazard	10^{-4}	$2x10^{-5}$	0.2
High Hazard	10^{-5}	$3x10^{-6}$ (2)	0.3

(1) $2x10^{-2}$ with I = 1.07 ~ 10^{-2}
(2) $2x10^{-5}$ with I = 1.35 ~ $3x10^{-6}$

Conservatism can be achieved in designs by specifying factors of safety for Allowable Stress Design (ASD) and load factors for Strength Design (SD). Consistent with the ratios in Table B-3, the loading conditions recommended for design of DOE facilities are given in Table B-4.

Table B-4 Recommended Wind and Tornado Load Combinations

Facility		General Use	Important or Low Hazard	Moderate Hazard	High Hazard
ASD	Extreme Winds	DL + W 0.75(DL + LL + W)	DL + W 0.75(DL + LL + W)	0.9(DL + W) 0.67(DL + LL + W)	0.9(DL + W) 0.67(DL + LL + W)
	Tornadoes			0.75(DL + W_t) 0.63(DL + LL + W_t)	0.75(DL + W_t) 0.63(DL + LL + W_t)
SD	Extreme Winds	0.9DL + 1.3W 1.2DL + 0.5LL + 1.3W	0.9DL + 1.3W 1.2DL + 0.5LL + 1.3W	DL + 1.3W 1.1DL + 0.5LL + 1.2W	DL + 1.3W 1.1DL + 0.5LL + 1.2W
	Tornadoes			DL + W_t DL + LL + W_t	DL + W_t DL + LL + W_t

ASD = Allowable Strength Design
Use allowable stress appropriate for building material
SD = Strength Design
Use ϕ factors appropriate for building material

DL = Dead load
LL = Live load
W = Extreme wind load
W_t = Tornado load, including APC if appropriate

Since the ratio of extreme wind hazard probability to performance goal probability for General Use and Important or Low Hazard facilities are the largest (20), designs for these categories should be the most conservative in terms of factors of safety for ASD and load factors for SD. The recommended combinations are essentially those given in Reference 14. The recommended load combination for Moderate and High Hazard facilities are slightly less conservative than for the General Use and Important or Low Hazard categories, because the ratio of extreme wind hazard probability to performance goal probability is less (10). The requirements have been reduced by approximately 10 percent.

The tornado hazard probabilities for both Moderate and High Hazard facilities are less than the performance goal probabilities. Hence, the ratios in Table B-3 are less than one. The tornado load combinations for both ASD and SD recognize that the performance goals are theoretically met and <u>no</u> conservatism in the load combination is required. This approach is consistent with criteria for commercial nuclear power plants as given in ACI 349 (Ref. 30) for concrete and ANSI/AISC N690-1984 (Ref. 31) for steel.

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