Chapter 2
Earthquake Design and Evaluation Criteria

2.1 Introduction

This chapter describes requirements for the design or evaluation of all classes of structures, systems, and components (SSCs) comprising DOE facilities for earthquake ground shaking. These classes of SSCs include safety class and safety significant SSCs per DOE-STD-3009-94 (ref. 1-6) and life-safety SSCs per Uniformed Building Codes. This material deals with how to establish Design/Evaluation Basis Earthquake (DBE) loads on various classes of SSCs; how to evaluate the response of SSCs to these loads; and how to determine whether that response is acceptable. This chapter also covers the importance of design details and quality assurance to earthquake safety. These earthquake design and evaluation provisions are equally applicable to buildings and to items contained within the building, such as equipment and distribution systems. These provisions are intended to cover all classes of SSCs for both new construction and existing facilities. These design and evaluation criteria have been developed such that the target performance goals of the NPH Implementation Guide are achieved. For more explanation see the Commentary (Appendix C) herein and the Basis Document (Ref. 2-1).

2.2 General Approach for Seismic Design and Evaluation

This section presents the approach upon which the specific seismic force and story drift provisions for seismic design and evaluation of structures, systems, and components in each Performance Category (as described in Section 2.3) is based. These provisions include the following steps:

1. Selection of earthquake loading
2. Evaluation of earthquake response
3. Specification of seismic capacity and drift limits, (acceptance criteria)
4. Ductile detailing requirements

It is important to note that the above four elements taken together comprise seismic design and evaluation criteria. Acceptable performance (i.e., achieving performance goals) can only be reached by consistent specification of all design criteria elements as shown in Figure 2-1. In order to achieve the target performance goals, these seismic design and evaluation criteria specify seismic loading in probabilistic terms. The remaining elements of the criteria (see Fig. 2-1) are deterministic design rules which are familiar to design engineers and

which have a controlled level of conservatism. This level of conservatism combined with the specification of seismic loading, leads to performance goal achievement.

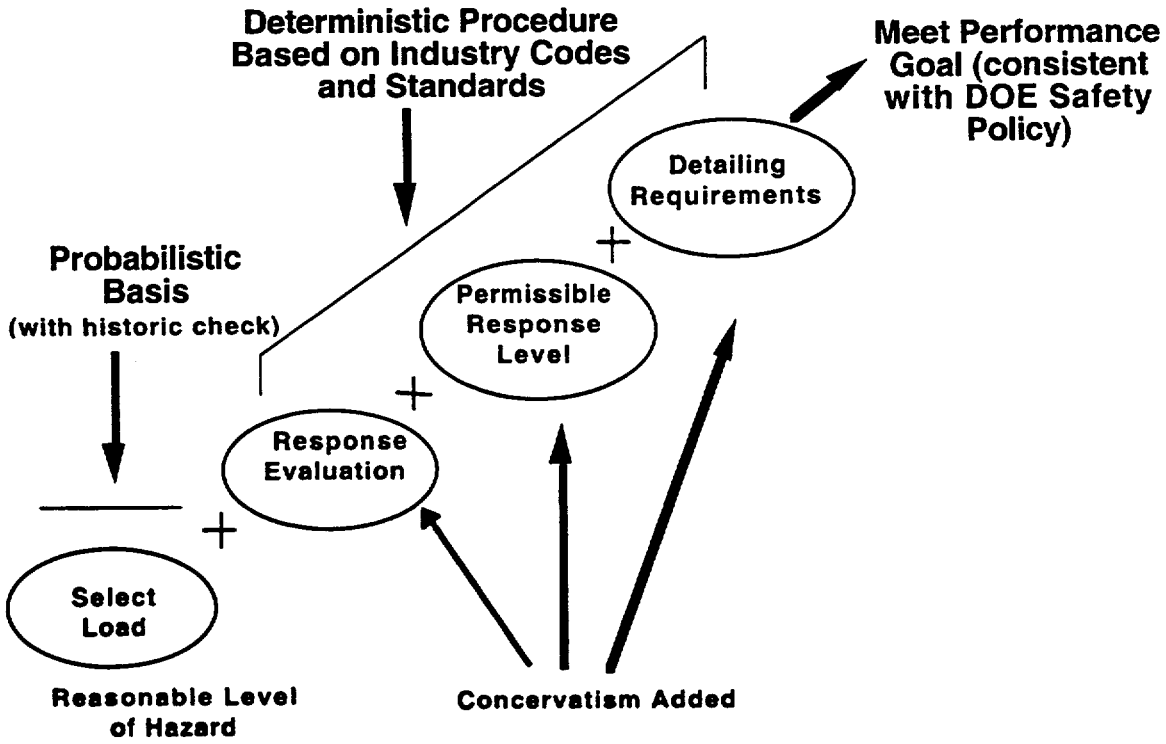


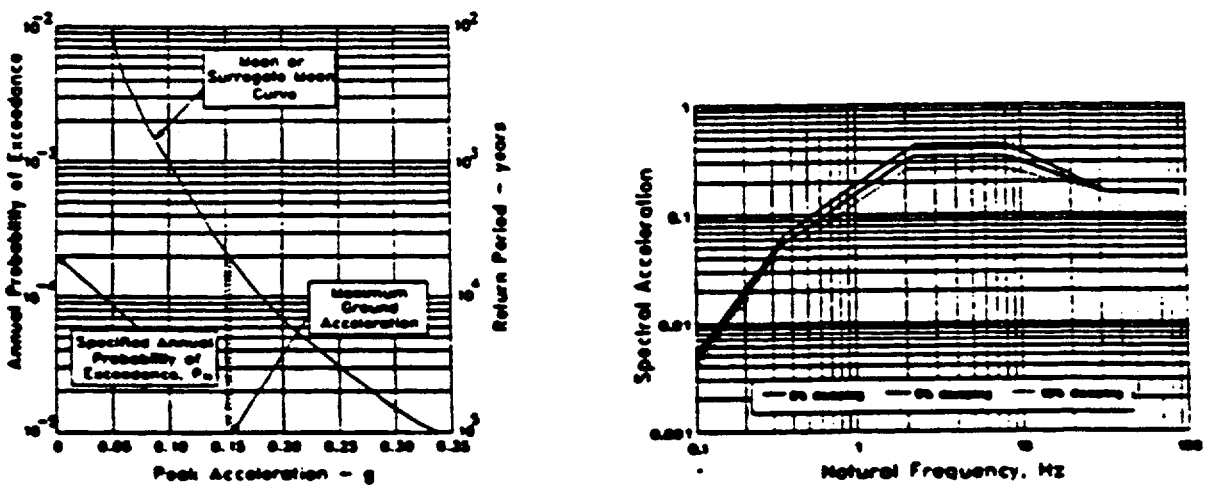
Figure 2-1. DOE-STD-1020 Combines Various Steps to Achieve Performance Goals

Criteria are provided for each of the four Performance Categories 1 to 4 as defined in the NPH Implementation Guide of DOE Order 420.1 and DOE-STD-1021 (Ref. 1-6). The criteria for Performance Categories 1 and 2 are similar to those from model building codes, with the exception that DOE requirements specify a 1000 year return period in the case of PC-2. Criteria for PC-3 are similar to those for Department of Defense Essential Facilities (Ref. C-5) Tri-Services Manual. Criteria for PC-4 approach the provisions for commercial nuclear power plants.

Seismic loading is defined in terms of a site-specified design response spectrum (the Design/Evaluation Basis Earthquake, [DBE]). Either a site-specific design response spectrum specifically developed for the site, or a generic design response spectrum that is appropriate or conservative for the site may be used. Seismic hazard estimates are used to establish the DBE per DOE-STD-1023 (REF. 2-22).

For each Performance Category, a mean annual exceedance probability for the DBE, P_H is specified from which the maximum ground acceleration (and/or velocity) may be determined from probabilistic seismic hazard curves, see Table 2-1. Evaluating maximum ground acceleration from a specified mean annual probability of exceedance is illustrated in Figure 2-2a. Earthquake input excitation to be used for design and evaluation by these provisions is defined by a median amplification smoothed and broadened design/evaluation response spectrum shape such as that shown in Figure 2-2b (from Ref. 2-2) anchored to the maximum ground acceleration and/or velocity. Such spectra are determined in accordance with DOE-STD-1023 (Ref. 2-22).

It should be understood that the spectra shown in Figure 2-2 or in-structure spectra developed from them represent inertial effects. They do not include differential support motions, typically called seismic anchor motion (SAM), of structures, equipment or distribution systems supported at two or more points. While SAM is not usually applicable to building design, it might have a significant effect on seismic adequacy of equipment or distribution systems.



a) Evaluating Peak Acceleration from Annual Probability of Exceedance with a Seismic Hazard Curve

b) Median Amplification, smoothed and broadened, Design/Evaluation Response Spectra

Figure 2-2. Earthquake Input Excitation is Defined by Maximum Ground Acceleration Anchoring Site-Specific Response Spectra

Table 2-1 Seismic Performance Categories and Seismic Hazard Exceedance Levels

Performance Category	Mean Seismic Hazard Exceedance Levels, P_H	Return Period
0	No Requirements	
1	$2x10^{-3}$	500yr
2	$1x10^{-3}$	1000yr
3	$5x10^{-4}$ ($1x10^{-3}$)[1]	2000yr (1000yr)[1]
4	$1x10^{-4}$ ($2x10^{-4}$)[1]	10,000yr (5000yr)[1]

[1] For sites such as LLNL, SNL-Livermore, SLAC, LBL, and ETEC, which are near tectonic plate boundaries.

Performance Category 2 and lower SSCs may be seismically designed or evaluated using the approaches specified in building code seismic provisions. However, for Performance Category 3 or higher, the seismic evaluation must be performed by a dynamic analysis approach. A dynamic analysis approach requires that:

1. The input to the SSC model be defined by either a design response spectrum, or a compatible time history input motion.
2. The important natural frequencies of the SSC be estimated, or the peak of the design response spectrum be used as input. Multi-mode effects must be considered.
3. The resulting seismic induced inertial forces be appropriately distributed and a load path evaluation (see Section C.4.2) for structural adequacy be performed.

The words "dynamic analysis approach" are not meant to imply that complex dynamic models must be used in the evaluation. Often equivalent static analysis models are sufficient if the above listed three factors are incorporated. However, use of such simplified models for

structures in Performance Category 3 or higher must be justified and approved by DOE. This dynamic analysis approach should comply with the seismic response analysis provisions of ASCE 4 (Ref. 2-3) except where specific exceptions are noted.

The maximum ground acceleration and ground response spectra determined in the manner illustrated in Figure 2-2 are used in the appropriate terms of the UBC equation for base shear. The maximum ground acceleration is also used in the UBC equation for seismic force on equipment and non-structural components. Use of modern site-specific earthquake ground motion data is considered to be preferable to the general seismic zonation maps from the UBC and should be applied according to the guidance provided in DOE-STD-1023 (Ref. 2-22). For structures, 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 criteria for Performance Category 2 and lower SSCs. This reduced seismic response is combined with non-seismic concurrent loads and then compared to code allowable response limits (or code ultimate limits combined with code specified load factors). The design detailing provisions from the UBC, which provide ductility, toughness, and redundancy, are also required such that SSCs can fully achieve potential inelastic energy absorption capability. Normally, relative seismic anchor motion (SAM) is not considered explicitly by model building code seismic provisions. However, SAM should be considered for SSCs in PC-2 or higher categories.

The Uniform Building Code (UBC) has been followed for Performance Categories 1 and 2 because it is believed that more engineers are familiar with this code than other model building codes. The Interagency Committee on Seismic Safety in Construction (ICSSC, Ref. 2-4) has concluded that the following seismic provisions are equivalent for a given DBE:

1. 1994 Uniform Building Code (Ref. 2-5)
2. 1991 NEHRP Recommended Provisions (Ref. 2-6)
3. 1993 BOCA National Building Code (Ref. 2-7)
4. 1994 SBCCI Standard Building Code (Ref. 2-8)

These other model building codes may be followed provided site-specific ground motion data is incorporated into the development of earthquake loading in a manner similar to that described in this document for the UBC.

For Performance Category 3 and 4 SSCs, these seismic design and evaluation criteria specify that seismic evaluation be accomplished by dynamic analysis. The recommended

approach is to perform an elastic response spectrum dynamic analysis to evaluate elastic seismic demand on SSCs. Inelastic energy absorption capability is allowed by permitting limited inelastic behavior. By these provisions, inelastic energy absorption capacity of structures is accounted for by the parameter, F_μ. However, strength and ductile detailing for the entire load path should be assured. 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. The same F_μ values are specified for both Performance Categories of 3 and 4. In order to achieve the conservatism appropriate for the different Performance Categories, the reduced seismic forces are multiplied by a scale factor. Scale factors are specified for Performance Category 3 and 4. The resulting factored seismic forces are combined with non-seismic concurrent loads and then compared to code ultimate response limits. The design detailing provisions from the UBC, which provide ductility, toughness, and redundancy, are also required such that SSCs can fully achieve potential inelastic energy absorption capability. Also, explicit consideration of relative seismic anchor motion (SAM) effects is required for Performance Category 3 and higher.

The overall DOE Seismic Design and Evaluation Procedure is shown in Figure 2-3. In addition to the general provisions described in this chapter, the topics discussed in Appendix C should be considered before commencing design or evaluation.

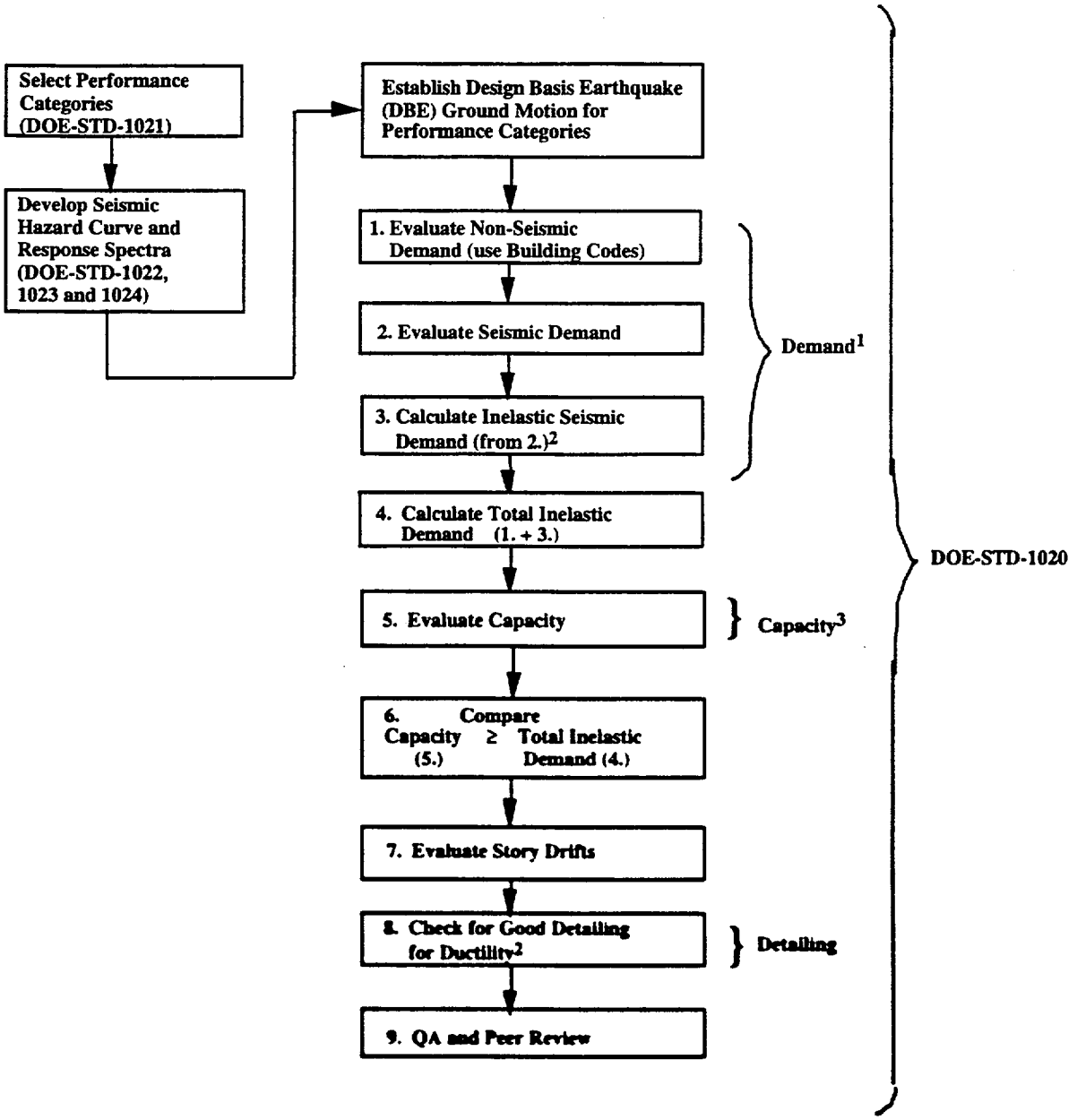
2.3 Seismic Design and Evaluation of Structures, Systems, and Components

- Select Performance Categories of structure, system, or component based on DOE-STD-1021 (Ref. 1-10).

- For sites with Performance Category 3 or 4 structures, systems, and components, obtain or develop a seismic hazard curve and design response spectra in accordance with DOE-STD-1023 (Ref 2-22) for all performance categories based on site characterization discussed in DOE-STD-1022 (Ref 1-15). In the interim, Eastern U.S. sites may use DOE-STD-1024. (Ref. 2-23)

- Establish design basis earthquake from P_H, (see Table 2-1) mean seismic hazard curve, and median response spectra.

  For sites with only PC-1 or 2 SSC, and no site-specific seismic hazard curve, obtain seismic coefficients from model building codes.



1. See Section C.4 for further discussion.
2. For evaluation of existing facilities, the strength and detailing of the entire load path must be checked prior to assignment of ductility reduction factors.
3. See Section C.5 for further discussion.

Figure 2-3. DOE Seismic Design and Evaluation Procedure

Minimum values of peak ground acceleration (PGA) shall be:

0.06g for Performance Category 3
0.10g for Performance Category 4

2.3.1 Performance Category 1 and 2 Structures, Systems, and Components.

Seismic design or evaluation of Performance Category 2 and lower SSCs is based on model building code seismic provisions. In these criteria, the current version of the Uniform Building Code shall be followed. Alternatively, the other equivalent model building codes may be used. All UBC seismic provisions shall be followed for Performance Category 2 and lower SSCs (with modifications as described below).

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

$$V = \frac{ZICW}{R_w} \tag{2-1}$$

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 which results in element forces which represent inelastic seismic demand, D_{SI}

The steps in the procedure for PC-1 and 2 SSCs are as follows:

- Evaluate element forces for non-seismic loads, D_{NS}, expected to be acting concurrently with an earthquake.
- Evaluate element forces, D_{SI}, for earthquake loads.

  a. Static force method, where V is applied as a load distributed over the height of the structure for regular facilities, or dynamic force method for irregular facilities as described in the UBC.

  b. In either case, the total base shear is given by Equation 2-1 where the parameters are evaluated as follows:

  1. Z is the peak ground acceleration from site-specific seismic hazard curves at the following exceedance probabilities if available:

Performance Category 1 - $2x10^{-3}$
Category 2 - $1x10^{-3}$

Otherwise, Z is obtained using UBC and adjusted per the procedures provided in DOE-STD-1023.

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. See Fig. 2-4 below:

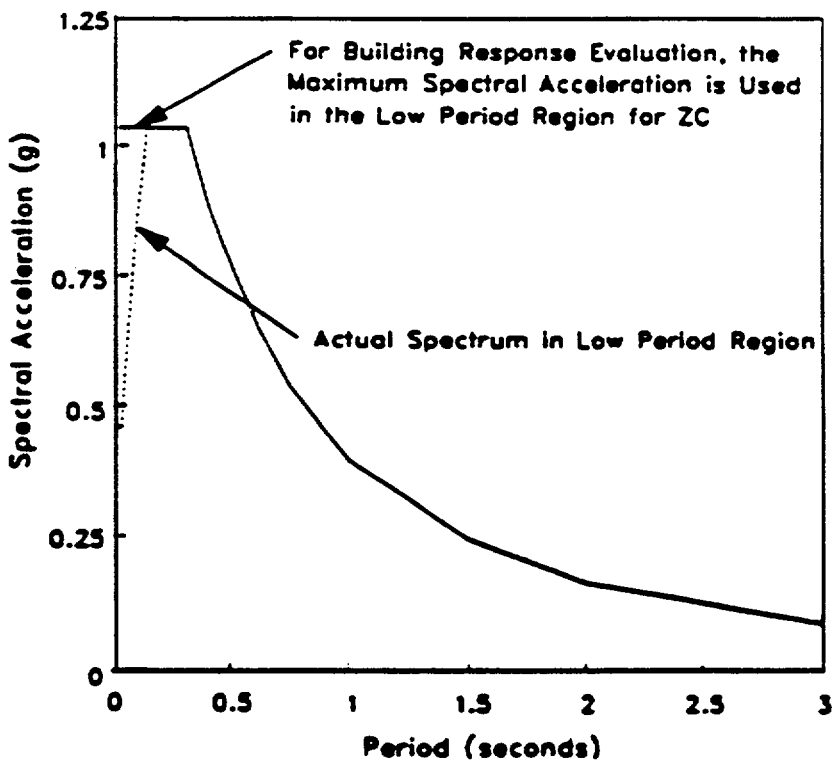


Figure 2-4. Example Design/Evaluation Earthquake Ground Motion Response Spectrum

For systems and components, spectral amplification is accounted for by C_p in the UBC equipment force equation as discussed in Section 2.4.1.

3. If a recent site-specific seismic hazard assessment is not available, it is acceptable to determine ZC from Table C-5 values and appropriate response spectra. For eastern U.S. sites DOE -STD-1024 provides guidance. If ZC, determined from a recent site-specific assessment is less than that given by UBC provisions, any significant differences with UBC must be justified. Final earthquake loads are subject to approval by DOE.

4. Importance factor, I, should be taken as:

   Performance Category 1, I = 1.0

   Performance Category 2, I = 1.25

5. For structures, reduction factors, R_W, are shown in Table 2-2. For systems and components, the reduction factor is implicitly included in C_p.

- Combine responses from various loadings (D_{NS} and D_{SI}) to evaluate demand, D_{TI}, by code specified load combination rules (e.g., load factors for ultimate strength design or unit load factors for allowable stress design).
- Evaluate capacities of SSCs, C_C, from code ultimate values when strength design is used (e.g., UBC Chapter 19 for reinforced concrete or LRFD for steel) or from allowable stress levels (with one-third increase) when allowable stress design is used. Minimum specified or 95% non-exceedance in-situ values for material strengths should be used for capacity estimation.
- Compare demand, D_{TI}, with capacity, C_C, for all SSCs. If D_{TI} is less than or equal to C_C, the facility satisfies the seismic force requirements If D_{TI} is greater than C_C, the facility has inadequate seismic resistance.

Table 2-2. Code Reduction Coefficients, R_W

Structural System (Terminology is identical to the UBC)	R_W
MOMENT RESISTING FRAME SYSTEMS - Beams	
Steel Special Moment Resisting Frame (SMRF)	12
Concrete SMRF	12
Concrete Intermediate Moment Frame (IMRF)	8
Steel Ordinary Moment Resting Frame	6
Concrete Ordinary Moment Resisting Frame	5
SHEAR WALLS	
Concrete or Masonry Walls	8(6)
Plywood Walls	9(8)
Dual System, Concrete with SMRF	12
Dual System, Concrete with Concrete IMRF	9
Dual System, Masonry with SMRF	8
Dual System, Masonry with Concrete IMRF	7
STEEL ECCENTRIC BRACED FRAMES (EBF)	
Beams and Diagonal Braces	10
Beams and Diagonal Braces, Dual System with Steel SMRF	12
CONCENTRIC BRACED FRAMES	
Steel Beams	8(6)
Steel Diagonal Braces	8(6)
Concrete Beams	8(4)
Concrete Diagonal Braces	8(4)
Wood Trusses	8(4)
Beams and Diagonal Braces, Dual Systems	
Steel with Steel SMRF	10
Concrete with Concrete SMRF	9
Concrete with Concrete IMRF	6

Note: Values herein assume good seismic detailing practice per the UBC 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.

- Evaluate story drifts (i.e., the displacement of one level of the structure relative to the level above or below due to the design seismic forces), including both translation and torsion. Calculated story drifts should not exceed $0.04/R_W$ times the story height nor 0.005 times the story height for buildings with a fundamental period less than 0.7 seconds. For more flexible 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 2-1; actual drift can be estimated by multiplying calculated drifts by 3 ($R_W/8$). These drifts limits may be exceeded when it is demonstrated that greater drift can be tolerated by both structural systems and non-structural elements.

- Elements of the facility shall be checked to assure that all detailing requirements of the UBC provisions are met. The basic UBC seismic detailing provisions must be met if Z is 0.11g or less. UBC Seismic Zone No. 2 provisions shall be met when Z is between 0.12 and 0.24g. UBC Seismic Zone Nos. 3 & 4 provisions shall be followed when Z is 0.25g or more.

- A quality assurance program consistent with model building code requirements shall be implemented for SSCs in Performance Categories 1 and 2. In addition, peer review shall be conducted for Performance Category 2 SSCs.

2.3.2 Performance Category 3 and 4 Structures, Systems, and Components

The steps in the procedure for PC-3 and 4 SSCs are as follows:

- Evaluate element forces, D_{NS}, for the non-seismic loads expected to be acting concurrently with an earthquake.

- Calculate the elastic seismic response to the DBE, D_s, using a dynamic analysis approach and appropriate damping values from Table 2-3. Response Level 3 is to be used only for justifying the adequacy of existing SSCs with adequate ductile detailing. Note that for evaluation of systems and components supported by the structure, in-structure response spectra are used. For PC-3 and PC-4 SSCs, the dynamic analysis must consider 3 orthogonal components of earthquake ground motion (two horizontal and one vertical). Responses from the various direction components shall be combined in accordance with ASCE 4. Include, as appropriate, the contribution from seismic anchor motion. To determine response of SSCs which use $F\mu > 1$, note that for fundamental periods lower than the period at which the maximum spectral amplification occurs, the maximum spectral acceleration should be used. For higher modes, the actual spectral accelerations should be used.

- Calculate the inelastic seismic demand element forces, D_{SI}, as

$$D_{SI} = SF \frac{D_S}{F_\mu} \qquad (2\text{-}2)$$

where: F_μ = Inelastic energy absorption factor from Table 2-4 for the appropriate structural system and elements having adequate ductile detailing

SF = Scale factor related to Performance Category
= 1.25 for PC-4
= 1.0 for PC-3

Variable scale factors, based on the slope of site-specific hazard curves, may be used as discussed in Appendix C to result in improved achievement of performance goals. SF is applied for evaluation of structures, systems, and components. At this time, $F\mu$ values are not provided for systems and components. It is recognized that many systems and components exhibit ductile behavior for which $F\mu$ values greater than unity would be appropriate (see Section C.4.4.2). Low $F\mu$ values in Table 2-4 are intentionally specified to avoid brittle failure modes.

- Evaluate the total inelastic-factored demand D_{TI} as the sum of D_{SI} and D_{NS} (the best-estimate of all non-seismic demands expected to occur concurrently with the DBE).

$$D_{TI} = D_{NS} + D_{SI} \qquad (2\text{-}3)$$

- Evaluate capacities of elements, C_C, from code ultimate or yield values

Reinforced Concrete

Use UBC Chapter 19

Steel

Use UBC Chapter 22 Standards

— LRFD provisions, or

— Plastic Design provisions, or

— Allowable Stress Design provision scaled by 1.4 for shear in members and bolts and 1.7 for all other stresses.

Refer to References 2-9 and 2-10 for related industry standards. Note that strength reduction factors, ϕ, are retained. Minimum specified or 95%

nonexceedence in-situ values for material strengths should be used to estimate capacities.

- The seismic capacity is adequate when C_C exceeds D_{TI}, i.e.:

$$C_C \geq D_{TI} \tag{2-4}$$

- 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 lateral seismic forces are not reduced by F_μ when computing story drifts). Calculated story drifts should not exceed 0.010 times the story height for structures with contribution to distortion from both shear and flexture. For structures in which shear distortion is the primary contributer to drift, such as those with low rise shear walls or concentric braced-frames, the calculated story drift should not exceed 0.004 times the story height. These drift limits may be exceeded when acceptable performance of both the structure and nonstructural elements can be demonstrated at greater drift.

- Check elements to assure that good detailing practice has been followed (e.g., see sect. C.4.4.2). Values of F_μ given in Table 2-4 are upper limit values assuming good design detailing practice and consistency with recent UBC provisions. Existing facilities may not be consistent with recent provisions, and, if not, must be assigned reduced F_μ. Basic UBC seismic detailing provisions shall be followed if the PGA at P_H is 0.11g or less. UBC Seismic Zone No. 2 provisions should be met when the PGA at P_H is between 0.12 and 0.24g. UBC Seismic Zone Nos. 3 & 4 provisions should be followed when the PGA at P_H is 0.25g or more.

- Implement peer review of engineering drawings and calculations (including proper applicaton of F_μ values), increased inspection and testing of new construction or existing facilities.

2.3.3 Damping Values for Performance Category 3 and 4 Structures, Systems, and Components

Damping values to be used in linear elastic analyses are presented in Table 2-3 at three different response levels as a function of D_T/C_C.

D_T is the elastically computed total demand,

$$D_T = D_{NS} + D_S \tag{2-5}$$

and C_C is the code specified capacity.

When determining the input to subcomponents mounted on a supporting structure, the damping value to be used in elastic response analyses of the supporting structure shall be based on the response level reached in the majority of the seismic load resisting elements of the supporting structure. This may require a second analysis.

In lieu of a second analysis to determine the actual response of the structure, Response Level 1 damping values may be used for generation of in-structure spectra. Response Level 1 damping values must be used if stability considerations control the design.

When evaluating the structural adequacy of an existing SSC, Response Level 3 damping may be used in elastic response analyses independent of the state of response actually reached, because such damping is expected to be reached prior to structural failure.

When evaluating a new SSC, damping is limited to Response Level 2. For evaluating the structural adequacy of a new SSC, Response Level 2 damping may be used in elastic response analyses independent of the state of response actually reached.

The appropriate response level can be estimated from the following:

Response Level	D_T/C_C
3**	≥1.0
2*	≈0.5 to 1.0
1*	≤ 0.5

* Consideration of these damping levels is required only in the generation of floor or amplified response spectra to be used as input to subcomponents mounted on the supporting structure. For analysis of structures including soil-structure interaction effects (sec C.4.3), D_T/C_C ratios for the best estimate case shall be used to determine response level

** Only to be used for justifying the adequacy of existing SSCs with adequate ductile detailing. However, functionality of SSCs in PC-3 and PC-4 must be given due consideration.

Table 2-3 Specified Damping Values

	Damping (% of critical)		
Type of Component	Response Level 1	Response Level 2	Response Level 3
Welded and friction bolted metal structures	2	4	7
Bearing-bolted metal structures	4	7	10
Prestressed concrete structures (without complete loss of prestress)	2	5	7
Reinforced concrete structures	4	7	10
Masonry shear walls	4	7	12
Wood structures with nailed joints	5	10	15
Distribution systems***	3	5	5
Massive, low-stressed components (pumps, motors, etc.)	2	3	–*
Light welded instrument racks	2	3	–*
Electrical cabinets and other equipment	3	4	5**
Liquid containing metal tanks			
Impulsive mode	2	3	4
Sloshing mode	0.5	0.5	0.5

* Should not be stressed to Response Level 3. Use damping for Response Level 2.

** May be used for anchorage and structural failure modes which are accompanied by at least some inelastic response. Response Level 1 damping values should be used for functional failure modes such as relay chatter or relative displacement issues which may occur at a low cabinet stress level

*** Cable trays more than one half full of loose cables may use 10% of critical damping

Table 2-4 Inelastic Energy Absorption Factors, F_μ

Structural System (terminology is identical to Ref. 2-5)	F_μ
MOMENT RESISTING FRAME SYSTEMS - Beams	
Steel Special Moment Resisting Frame (SMRF)	3.0
Concrete SMRF	2.75
Concrete Intermediate Moment Frame (IMRF)	1.5
Steel Ordinary Moment Resting Frame	1.5
Concrete Ordinary Moment Resisting Frame	1.25
SHEAR WALLS	
Concrete or Masonry Walls	
In-plane Flexure	1.75
In-plane Shear	1.5
Out-of-plane Flexure	1.75
Out-of plane Shear	1.0
Plywood Walls	1.75
Dual System, Concrete with SMRF	2.5
Dual System, Concrete with Concrete IMRF	2.0
Dual System, Masonry with SMRF	1.5
Dual System, Masonry with Concrete IMRF	1.4
STEEL ECCENTRIC BRACED FRAMES (EBF)	
Beams and Diagonal Braces	2.75
Beams and Diagonal Braces, Dual System with Steel SMRF	3.0
CONCENTRIC BRACED FRAMES	
Steel Beams	2.0
Steel Diagonal Braces	1.75
Concrete Beams	1.75
Concrete Diagonal Braces	1.5
Wood Trusses	1.75
Beams and Diagonal Braces, Dual Systems	
Steel with Steel SMRF	2.75
Concrete with Concrete SMRF	2.0
Concrete with Concrete IMRF	1.4
METAL LIQUID STORAGE TANKS	
Moment and Shear Capacity	1.25
Hoop Capacity	1.5

Note: 1. Values herein assume good seismic detailing practice per Reference 2-5, along with reasonably uniform inelastic behavior. Otherwise, lower values should be used.

2. F_μ for columns for all structural systems is 1.5 for flexure and 1.0 for axial compression and shear. For columns subjected to combined axial compression and bending, interaction formulas shall be used

3. Connections for steel concentric braced frames should be designed for at least 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 transferred to the brace by the structural system.

4. Connections for steel moment frames and eccentric braced frames and connections for concrete, masonry, and wood structural systems should follow Reference 2-5 provisions utilizing the prescribed seismic loads from these criteria 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.

5. F_μ for chevron, V, and K bracing is 1.5. K bracing requires special consideration for any building if Z is 0.25g or more.

2.4 Additional Requirements

2.4.1 Equipment and Distribution Systems

For Performance Category 2 and lower systems and components, the design or evaluation of equipment or non-structural elements supported within a structure may be based on the total lateral seismic force, F_p, as given by the UBC provisions (Ref. 2-5). For Performance Category 3 and higher systems and components, seismic design or evaluation shall be based on dynamic analysis, testing, or past earthquake and testing experience data. In any case, equipment items and non-structural elements must be adequately anchored to their supports unless it can be shown by dynamic analysis or by other conservative analysis and/or test that the equipment will be able to perform all of its safety functions without interfering with the safety functions of adjacent equipment. Anchorage must be verified for adequate strength and sufficient stiffness.

Evaluation by Analysis

By the UBC provisions for PC-1 and 2, parts of the structures, permanent non-structural 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:

$$F_p = ZI_pC_pW_p \tag{2-8}$$

where: W_p = the weight of element or component

C_p = a horizontal force factor as given by Table 16-O of the UBC for rigid elements, or determined from the dynamic properties of the element and supporting structure for nonrigid elements (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 16-O, but need not exceed 2.0)

The lateral force determined using Equation 2-8 shall be distributed in proportion to the mass distribution of the element or component. Forces determined from Equation 2-8 shall 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 shall be applied in the horizontal direction that results in the most critical loadings for design/evaluation.

Note that DOE-STD-1020 takes one exception to the UBC provisions. By the UBC for equipment located above grade, the value C_P for non-rigid or flexibly supported items is twice the value for rigid and rigidly supported equipment. However, by the UBC for equipment located at or below grade, the value C_P for non-rigid or flexibly supported items is the same as the value for rigid and rigidly supported equipment. By DOE-STD-1020 for equipment located at or below grade, the value C_P for non-rigid or flexibly supported items (except for piping, ducting or conduit systems made of ductile materials and connections) is specified to be twice the value for rigid and rigidly supported equipment. An alternative methodology is contained in the 1994 NEHRP Provisions (Ref. 2-24) which accounts for the dynamic properties of the equipment, the location of the equipment within the primary structure, and the response of the primary supporting structure.

For PC-3 and PC-4 subsystems and components, support excitation shall be represented by means of floor response spectra (also commonly called in-structure response spectra). Floor response spectra should be developed accounting for the expected response level of the supporting structure even though inelastic behavior is permitted in the design of the structure (see Section 2.3.3). It is important to account for uncertainty in the properties of the equipment, supporting structure, and supporting media when using in-structure spectra which typically have narrow peaks. For this purpose, the peak broadening or peak shifting techniques outlined in ASCE 4 shall be employed.

Equipment or distribution systems that are supported at multiple locations throughout a 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 to obtain the inertial loads. 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.

Seismic Anchor Motion

The seismic anchor motion (SAM) component for seismic response is usually obtained by conventional static analysis procedures. The resultant component of stress can be very significant if the relative motions of the support points are quite different. If all supports of a structural system supported at two or more points have identical excitation, then this component of seismic response does not exist. For multiply-supported components with different seismic inputs, support displacements can be obtained either from the structural response calculations of the supporting structure or from spectral displacement determined from the floor response

spectra. The effect of relative seismic anchor displacements shall be obtained by using the worst combination of peak displacements or by proper representation of the relative phasing characteristics associated with different support inputs. In performing an analysis of systems with multiple supports, the response from the inertial loads shall be combined with the responses obtained from the seismic anchor displacement analysis of the system by the SRSS rule $\left[R = \sqrt{(R_{inertia})^2 + (R_{SAM})^2}\right]$, where R = response parameter of interest.

Evaluation by Testing

Guidance for conducting testing is contained in IEEE 344 (Ref. 2-11). Input or demand excitation for the tested equipment shall be based on the seismic hazard curves at the specified annual probability for the Performance Category of the equipment (OBE provisions of Ref. 2-11 do not apply). When equipment is qualified by shake table testing, the DBE input to the equipment is defined by an elastic computed required-response-spectrum (RRS) obtained by enveloping and smoothing (filling in valleys) the in-structure spectra computed at the support of the equipment by linear elastic analyses. In order to meet the target performance goals established for the equipment, the Required Response Spectrum (RRS) must exceed the In-Structure Spectra by:

$$\text{RRS} \geq (1.1)(\text{In-Structure Spectra}) \qquad \text{for PC-2 and lower}$$

$$\text{RRS} \geq (1.4\text{SF})(\text{In-structure Spectra}) \qquad \text{for PC-3 and higher} \qquad (2\text{-}6)$$

where SF is the seismic scale factor from Equation 2-2.

The Test Response Spectrum (TRS) of test table motions must envelop the RRS. If equipment has been tested and shown to meet NRC requirements, then it need not be subjected to further testing.

Evaluation by Seismic Experience Data

For new design of systems and components, seismic qualification will generally be performed by analysis or testing as discussed in the previous sections. However, for existing systems and components, it is anticipated that many items will be judged adequate for seismic loadings on the basis of seismic experience data without analysis or testing. Seismic experience data has been developed in a usable format by ongoing research programs sponsored by the nuclear power industry. The references for this work are the Senior Seismic Review and Advisory Panel (SSRAP) report (Ref. 2-12) and the Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment (Ref. 2-13). Note that there are numerous restrictions ("caveats") on the use of this data as described in the SSRAP report and the GIP. It

is necessary to conduct either seismic analyses or shake table testing to demonstrate sufficient seismic capacity for those items that cannot be verified by seismic experience data or for items that are not obviously inherently rugged for seismic effects. There is an ongoing DOE program on the application of experience data for the evaluation of existing systems and components at DOE facilities. Currently, use of experience data is permitted for existing facilities and for the items specified in the two references, (Ref. 2-12) and (Ref. 2-13).

Anchorage and Supports

Adequate strength of equipment anchorage requires consideration of tension, shear, and shear-tension interaction load conditions. The strength of cast-in-place anchor bolts and undercut type expansion anchors shall be based on UBC Chapter 19 provisions (Ref. 2-5) for Performance Category 2 and lower SSCs and on ACI 349 provisions (Ref. 2-14) for Performance Category 3 and higher SSCs. For new design by AC1 349 provisions, it is required that the concrete pullout failure capacity be greater than the steel cast-in-place bolt tensile strength to assure ductile behavior. For evaluation of existing cast-in-place anchor bolt size and embedment depth, it is sufficient to demonstrate that the concrete pullout failure capacity is greater than 1.5 times the seismic induced tensile load. For existing facility evaluation, it may be possible to use relaxed tensile-shear interaction relations provided detailed inspection and evaluation of the anchor bolt in accordance with Reference 2-15 is performed.

The strength of expansion anchor bolts should generally be based on design allowable strength values available from standard manufacturers' recommendations or sources such as site-specific tests or Reference 2-15. Design-allowable strength values typically include a factor of safety of about 4 on the mean ultimate 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 2-15. A factor of safety of 3 is appropriate for this situation. When anchorage is modified or new anchorage is designed, design-allowable strength values including the factor of safety of 4 shall be used. For strength considerations of welded anchorage, AISC allowable values (Ref. 2-10) multiplied by 1.7 shall be used. Where shear in the member governs the connection strength, capacity shall be determined by multiplying the AISC allowable shear stress by 1.4.

Stiffness of equipment anchorage shall 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 and reduce its natural frequency, possibly increasing dynamic response. In addition, flexibility can lead to high stresses in anchorage components and failure of the anchorage or equipment sheet metal.

2.4.2 Evaluation of Existing Facilities

It is anticipated that these criteria would also be applied to evaluations of existing facilities. General guidelines for the seismic evaluation of existing facilities are presented in National Institute of Standards and Technology documents (Refs. 2-16 and 2-17), a DOD manual (Ref. 2-18), and in ATC-14, "Evaluating the Seismic Resistance of Existing Buildings" (Ref. 2-19) and ATC-22, "A Handbook for Seismic Evaluation of Existing Buildings" (Ref. 2-22). In addition, guidelines for upgrading and strengthening equipment are presented in Reference 2-23. Also, guidance for evaluation of existing equipment by experience data is provided in Reference 2-13. 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.

Once the as-is condition of a facility has been verified and deficiencies or weak links have been identified, detailed seismic evaluation and/or upgrading of the facility as necessary 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-is configuration is evaluated instead of several configurations in an iterative manner (as is often 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. Input from safety personnel and/or accident analyses should be used as an aid in determining safety priorities.

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 criteria presented in Sections 2.3.1 or 2.3.2 and good seismic design practice had been employed, 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 criteria of this chapter, a back-fit analysis should be conducted. Several alternatives can be considered:

1. If an existing SSC is close to meeting the criteria, a slight increase in the annual risk to natural phenomena hazards can be allowed within the tolerance of meeting the target performance goals (See Section 1.3). Note that reduced criteria for seismic evaluation of existing SSCs is supported in Reference 2-16. As a result, some relief in the criteria can be allowed by performing the evaluation using hazard exceedance probability of twice the value recommended in Table 2-1 for the Performance Category of the SSC being considered.
2. The SSC may be strengthened such that its seismic resistance capacity is sufficiently increased to meet these seismic criteria. When upgrading is required it should be designed for the original Performance Goal.
3. The usage of the facility may be changed such that it falls within a less hazardous Performance 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 SSC may be shown to be adequate. Alternatively, a probabilistic assessment might be undertaken in order to demonstrate that the performance goals can be met.

Requirements of Executive order 12941 (Ref. 1-6), as discussed in the Implementation Guide are to be implemented.

2.4.3 Basic Intention of Dynamic Analysis Based Deterministic Seismic Evaluation and Acceptance Criteria

The basic intention of the deterministic seismic evaluation and acceptance criteria defined in Section 2.3 is to achieve less than a 10% probability of unacceptable performance for a structure, system, or component (SSC) subjected to a Scaled Design/Evaluation Basis Earthquake (SDBE) defined by:

$$SDBE = (1.5SF)(DBE) \qquad (2\text{-}7)$$

where SF is the appropriate seismic scale factor from Equation 2-2.

The seismic evaluation and acceptance criteria presented in this section has intentional and controlled conservatism such that the target performance goals are achieved. The amount of intentional conservatism has been evaluated in Reference 2-1 such that there should be less than 10% probability of unacceptable performance at input ground motion defined by a scale

factor of 1.5SF times the DBE. Equation 2-7 is useful for developing alternative evaluation and acceptance criteria which are also based on the target performance goals such as inelastic seismic response analyses. To evaluate items for which specific acceptance criteria are not yet developed, such as overturning or sliding of foundations, or some systems and components; this basic intention must be met. If a nonlinear inelastic response analysis which explicitly incorporates the hysteretic energy dissipation is performed, damping values that are no higher than Response Level 2 should be used to avoid the double counting of this hysteretic energy dissipation which would result from the use of Response Level 3 damping values.

2.5 Summary of Seismic Provisions

Table 2-5 summarizes recommended earthquake design and evaluation provisions for Performance Categories 1 through 4. Specific provisions are described in detail in Section 2.3. The basis for these provisions is described in Reference 2-1.

Table 2-5 Summary of Earthquake Evaluation Provisions

	Performance Category (PC)			
	1	2	3	4
Hazard Exceedance Probability, P_H	$2x10^{-3}$	$1x10^{-3}$	$5x10^{-4}$ ($1x10^{-3}$)[1]	$1x10^{-4}$ ($2x10^{-4}$)[1]
Response Spectra	Median amplification (no conservative bias)			
Damping for Structural Evaluation	5%		Table 2-3	
Acceptable Analysis Approaches for Structures	Static or dynamic force method normalized to code level base shear		Dynamic analysis	
Analysis approaches for systems and components	UBC Force equation for equipment and non-structural elements (or more rigorous approach)		Dynamic analysis using in-structure response spectra (Damping from Table 2-3)	
Importance Factor	I=1.0	I=1.25	Not used	
Load Factors	Code specified load factors appropriate for structural material		Load factors of unity	
Scale Factors	Not Used		SF = 1.0	SF = 1.25
Inelastic Energy Absorption Ratios	Accounted for by R_w from Table 2-2		F_μ from Table 2-4 by which elastic response is reduced to account for permissible inelastic behavior	
Material Strength	Minimum specified or 95% non-exceedance in-situ values			
Structural Capacity	Code ultimate strength or allowable behavior level		Code ultimate strength or limit-state level	
Quality Assurance Program	Required within a graded approach (i.e., with increasing rigor ranging from UBC requirements from PC-1 to nuclear power plant requirements for PC-4)			
Peer Review	Not Required	Required within a graded approach (i.e., with increasing rigor ranging from UBC requirements from PC-2 to nuclear power plant requirements for PC-4)		

[1]For sites such as LLNL, SNL-Livermore, SLAC, LBL, & ETEC which are near tectonic plate boundaries

2.6 References

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Chapter 3
Wind Design and Evaluation Criteria

3.1 Introduction

This chapter presents a uniform approach to wind load determination that is applicable to the design of new and evaluation of existing structures, systems and components (SSCs). As discussed in Appendix D.1, a uniform treatment of wind loads is recommended to accommodate straight, hurricane, and tornado winds. SSCs are first assigned to appropriate Performance Categories by application of DOE-STD-1021. Criteria are recommended such that the target performance goal for each category can be achieved. Procedures according to the wind load provisions of ASCE 7 (Ref. 3-1) are recommended for determining wind loads produced by straight, hurricane and tornado winds. The straight wind/tornado hazard models for DOE sites published in Reference 3-2 are used to establish site-specific criteria for 25 DOE sites. For other sites, the wind/tornado hazard data shall be determined in accordance with DOE-STD-1023.

The performance goals established for Performance Categories 1 and 2 are met by model codes or national standards (see discussion in Appendix B). These criteria do not account for the possibility of tornado winds because wind speeds associated with straight winds typically are greater than tornado winds at annual exceedance probabilities greater than approximately 1×10^{-4}. Since model codes specify winds at probabilities greater than or equal to 1×10^{-2}, tornado design criteria are specified only for SSCs in Performance Categories 3 and higher, where hazard exceedance probabilities are less than 1×10^{-2}.

In determining wind design criteria for Performance Categories 3 and higher, the first step is to determine if tornadoes should be included in the criteria The decision logically can be made on the basis of geographical location, using historical tornado occurrence records. However, since site specific hazard assessments are available for the DOE sites, a more quantitative approach can be taken. Details of the approach are presented in Appendix D The annual exceedance probability at the intersection of the straight wind and tornado hazard curves is used to determine if tornadoes should be a part of the design criteria If the exceedance probability at the intersection of the curves is greater than or equal to 2×10^{-5} then tornado design criteria are specified. By these criteria, tornado wind speeds are determined at 2×10^{-5} for PC-3 and 2×10^{-6} for PC-4. If the exceedance probability is less than 2×10^{-5} only the effects of straight winds or hurricanes need be considered. For straight winds and hurricanes, wind speeds are determined at 1×10^{-3} for PC-3 and 1×10^{-4} for PC-4.

3.2 Wind Design Criteria

The criteria presented herein meets or exceeds the target performance goals described in DOE 5480.28 for each Performance Category. SSCs 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 Performance Category, but at different geographical locations, will have different wind speeds specified to achieve the same performance goal.

The minimum wind design criteria for each Performance Category are summarized in Table 3-1. The recommended basic wind speeds for straight wind, hurricanes and tornadoes are contained in Table 3-2 for laboratories, reservations, and production facilities. All wind speeds are fastest-mile, which is consistent with the ASCE 7 approach. Importance factors as given in ASCE 7 should be used were applicable. Importance factors are used to obtain wind speeds equivalent to 1×10^{-2} annual exceedance probability for Performance Category 2 and to account for hurricanes within 100 miles of the coastline in all Performance Categories.

Degrees of conservatism are introduced in the design process by means of load combinations. The combinations are given in the appropriate material-specific national concensus design standard, e.g. AISC Steel Construction Manual. Designers will need to exercise judgment in choosing the most appropriate combinations in some situations. Designs or evaluations shall be based on the load combination causing the most unfavorable effect. For PC-3 and 4 the load combination to be used should invoke either wind or tornado depending on which speed is specified in Table 3-2.

Most loads, other than dead loads, vary significantly with time. When these variable loads are combined with dead loads, their combined effect could be sufficient to reduce the risk of unsatisfactory performance to an acceptably low level. When more than one variable load is considered, it is unlikely that they will all attain their maximum value at the same time. Accordingly, some reduction in the total of the combined load effects is appropriate. This reduction is accomplished through load combination multiplication factors as given in the appropriate material-specific national concensus design standard.

Table 3-1 Summary Of Minimum Wind Design Criteria

	Performance Category	1	2	3	4
Wind	Hazard Annual Probability of Exceedance	$2x10^{-2}$	$2x10^{-2}$	$1x10^{-3}$	$1x10^{-4}$
	Importance Factor*	1.0	1.07	1.0	1.0
	Missile Criteria	NA	NA	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	Hazard Annual Probability of Exceedance	NA	NA	$2x10^{-5}$	$2x10^{-6}$
	Importance Factor*	NA	NA	I = 1.0	I = 1.0
	APC	NA	NA	40 psf @ 20 psf/sec	125 psf @ 50 psf/sec
	Missile Criteria	NA	NA	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 ASCE 7, Table 5 (Ref. 3-1) for importance factors to be used for all categories if the facility is prone to hurricanes or within 100 miles of Gulf of Mexico or Atlantic coastlines.

Table 3-2 Recommended Basic Wind Speeds for DOE Sites, in miles per hour

Performance Category	Fastest-Mile Wind Speeds at 10m Height					
	1	2	3		4	
	Wind	Wind	Wind	Tornado[4]	Wind	Tornado[4]
DOE PROJECT SITES	$2x10^{-2}$	$2x10^{-2}$	$1x10^{-3}$	$2x10^{-5}$	$1x10^{-4}$	$2x10^{-6}$
Kansas City Plant, MO	72	72	--	144	--	198
Los Alamos National Laboratory, NM	77	77	93	--	107	--
Mound Laboratory, OH	73	73	--	136	--	188
Pantex Plant, TX	78	78	--	132	--	182
Rocky Flats Plant, CO	109	109	138	(3)	161	(3)
Sandia National Laboratories, NM	78	78	93	--	107	--
Sandia National Laboratories, CA	72	72	96	--	113	--
Pinellas Plant, FL	93	93	130	--	150	--
Argonne National Laboratory--East, IL	70(1)	70(1)	--	142	--	196
Argonne National Laboratory--West, ID	70(1)	70(1)	83	--	95	--
Brookhaven National Laboratory, NY	70(1)	70(1)	--	95(2)	--	145
Princeton Plasma Physics Laboratory, NJ	70(1)	70(1)	--	103	--	150
Idaho National Engineering Laboratory	70(1)	70(1)	84	--	95	--
Feed Materials Production Center, OH	70(1)	70(1)	--	139	--	192
Oak Ridge, X-10, K-25, and Y-12, TN	70(1)	70(1)	--	113	--	173
Paducah Gaseous Diffusion Plant, KY	70(1)	70(1)	--	144	--	198
Portsmouth Gaseous Diffusion Plant, OH	70(1)	70(1)	--	110	--	166
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 Lab., CA	72	72	96	--	113	--
LLNL, Site 300, CA	80	80	104	--	125	--
Energy Technology & Engineering Center, CA	70(1)	70(1)	--	95(2)	--	111
Stanford Linear Accelerator Center, CA	72	72	95	--	112	--
Savannah River Site, SC	78	78	--	137	--	192

NOTES:

(1) Minimum straight wind speed.

(2) Minimum tornado speed.

(3) Although straight winds govern at Rocky Flats, because the potential for a tornado strike is high, it is recommended that facilities be designed for tornado missiles. APC need not be considered.

(4) Tornado speed includes rotational and translational effects.

(5) Hurricane effects adjustments as per Table 3-1.

3.2.1 Performance Category 1

The performance goals for Performance Category 1 SSCs are consistent with objectives of ASCE 7 Building Class I, Ordinary Structures. Similar criteria in model building codes such as the current Uniform Building Code (Ref. 3-3) are also consistent with the performance goal and may be used as an alternative criteria. The wind-force resisting system of structures should not collapse under design load. Survival without collapse implies that occupants should be able to find an area of relative safety inside the structure during an extreme wind event. Breach of structure envelope is acceptable, since confinement is not essential. Flow of wind through the structure and water damage are acceptable. Severe loss, including total loss, is acceptable, so long as the structure does not collapse and occupants can find safe areas within the building.

In ASCE 7 wind design criteria is based on an exceedance probability of 2×10^{-2} per year. The importance factor is 1.0, except if the site is within 100 miles of the Gulf of Mexico or Atlantic coastline, a slightly higher importance factor is recommended to account for the additional threat of hurricanes.

Distinctions are made in ASCE 7 between buildings and other structures and between main wind-force resisting systems and 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 SSCs should be classified as Exposure B, C, or D as defined in ASCE 7. Gust response factors (G) and velocity pressure exposure coefficients (K) should be used according to the rules of the ASCE 7 procedures.

Wind pressures are calculated on walls and roofs of enclosed structures by using appropriate pressure coefficients specified in ASCE 7. Internal pressures on components and cladding develop as a result of unprotected openings, or openings created by wind forces or missiles. The worst cases of combined internal and external pressures should be considered in wind design as required by ASCE 7.

SSCs in Performance Category 1 may be designed by either allowable stress design (ASD) or strength design (SD). Load combinations shall be considered to determine the most

unfavorable effect on the SSC being considered. When using ASD methods, customary allowable stresses appropriate for the material shall be used as given in the applicable material design standard (e.g. see Reference 3-4 for steel).

The SD method requires that the nominal strength provided be greater than or equal to the strength required to carry the factored loads. Appropriate material strength reduction factors should be applied to the nominal strength of the material being used. See Reference 3-5 for concrete or Reference 3-6 for steel for appropriate load combinations and strength reduction factors.

3.2.2 Performance Category 2

Performance Category 2 SSCs are equivalent to essential facilities (Class II), as defined in ASCE 7 or model building codes. The structure shall not collapse at design wind speeds. Complete integrity of the structure envelope is not required because no significant quantities of toxic or radioactive materials are present. However, breach of the SSC containment is not acceptable if the presence of wind or water interferes with the SSCs function.

An annual wind speed exceedance probability of $2x10^{-2}$ is specified for this Performance Category, but an importance factor of 1.07 in effect lowers the annual exceedance probability to $1x10^{-2}$. For those sites located within 100 miles of the Gulf of Mexico or Atlantic coastlines, ASCE 7 prescribes a slightly higher importance factor to account for the additional threat of hurricane winds.

Once the design wind speeds are established and the importance factors applied, the determination of wind loads on Performance Category 2 SSCs is identical to that described for Performance Category 1 SSCs. ASD or SD methods may be used as appropriate for the material being used. The load combinations described for Performance Category 1 are the same for Performance Category 2.

3.2.3 Performance Category 3

The performance goal for Performance Category 3 SSCs requires more rigorous criteria than is provided by national standards or model building codes. In some geographic regions, tornadoes must be considered.

Straight Winds and Hurricanes

For those sites where tornadoes are not a viable threat, 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 in ASCE 7 to account for additional threat of hurricane winds.

Once the design wind speeds are established and the importance factors applied, determination of Performance Category 3 wind loads is identical to Performance Category 1, except as noted below. SSCs in Performance Category 3 may be designed or evaluated by ASD or SD methods, as appropriate for the material used in construction. Because the hazard exceedance probability in Performance Category 3 contributes a larger percentage to the total probabilistic performance goal than in Performance Categories 1 or 2, less conservatism is needed in the Performance Category 3 design and evaluation criteria. This trend is different for seismic design as discussed in Chapter 2 and Appendix C. (See Appendix D for further explanation.) Thus, the load combinations given in the applicable material-specific national concensus design standard may be reduced by 10 percent. In combinations where gravity load reduces wind uplift, the reduction in conservatism is achieved by modifying only the gravity load factor.

When using ASD, allowable stresses shall be determined in accordance with applicable codes and standards (e.g. see Reference 3-4 for steel). Load combinations shall be evaluated to determine the most unfavorable effect of wind on the SSCs being considered. The SD load combinations shall be used along with nominal strength and strength reduction factors.

A minimum missile criteria is specified to account for objects or debris that could be picked up by straight winds, hurricanes or weak tornadoes. A 2x4 timber plank weighing 15 lbs is the specified missile. This missile represents a class of missiles transported by straight winds, hurricanes and weak tornadoes. Recommended impact speed is 50 mph at a maximum height of 30 ft above ground. The missile will break annealed 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 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 wall construction. See Table 3-3 for recommended wall barriers (Ref. 3-7).

Table 3-3 Recommended Straight Wind Missile Barriers for Performance Categories 3 and 4

Missile Criteria	Recommended Missile Barrier
2x4 timber plank 15 lb @ 50 mph (horiz.)	8-in. CMU wall with trussed horiz joint reinf @ 16 in. on center
max. height 30 ft. above ground Performance Category 3	Single wythe brick veneer with stud wall
max. height 50 ft. above ground Performance Category 4	4-in. concrete slab with #3 rebar @ 6 in. on center each way in middle of slab

Tornadoes

For those sites requiring design for tornadoes, the criteria are based on site-specific studies, as presented in Reference 3-2. An annual exceedance probability of $1x10^{-3}$, which is the same for straight wind, could be justified. As explained in Appendix D, a lower value is preferred because (1) the straight wind hazard curve gives wind speeds larger than the tornado hazard curve and (2) a lower hazard probability can be specified without placing undue hardship on the design. The basic tornado wind speed associated with an annual exceedance probability of $2x10^{-5}$ is recommended for Performance Category 3. The wind speed obtained from the tornado hazard curve is converted from peak gust to fastest-mile; use importance factor of 1.0 for Performance Category 3.

With the wind speed converted to fastest-mile wind and an importance factor of 1.0, the equations in ASCE 7 Table 4 should be used to obtain design wind pressures on SSCs. *Exposure Category C should always be used with tornado winds regardless of the actual terrain roughness.* Unconservative results will be obtained with exposure B. Tornadoes traveling over large bodies of water are waterspouts, which are less intense than land-based tornadoes. Thus, use of exposure category D also is not necessary. The velocity pressure exposure coefficient and gust response factor are obtained from ASCE 7. External pressure coefficients are used to obtain tornado wind pressures on various surfaces of structures. Net pressure coefficients are applicable to systems and components. On structures, a distinction is made between main wind-force resisting systems and components and cladding.

If a structure is not intentionally sealed to maintain an internal negative pressure for confinement of hazardous materials, or, if openings greater than one square foot per 1000 cubic feet of volume are present, or, if openings of this size can be caused by missile perforation, then the effects of internal pressure should be considered according to the rules of ASCE 7 If a

structure is sealed, then atmospheric pressure change (APC) associated with the tornado vortex should be considered instead of internal pressures (see Table 3-1 for APC values).

The maximum APC pressure occurs at the center of the tornado vortex where the wind speed is theoretically zero. A more severe loading condition occurs at the radius of maximum tornado wind speed, which is some distance from the vortex center. At the radius of maximum wind speed, the APC may be one-half its maximum value. Thus, a critical tornado load combination on a sealed building is one-half maximum APC pressure combined with 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. A rapid rate of pressure change, which can accompany a rapidly translating tornado, should be analyzed to assure that it does not damage safety-related ventilation systems. Procedures and computer codes are available for such analyses (Ref. 3-8).

When using ASD methods, allowable stresses appropriate for the materials shall be used. Since in this case, the hazard probability satisfies the performance goal, little or no additional conservatism is needed in the design. Thus, for ASD the tornado wind load combinations are modified to negate the effect of safety factors. For example, the combinations from ASCE 7 become:

(a) $0.63\ (D + W_t)$
(b) $0.62\ (D + L + L_r + W_t)$
(c) $0.62\ (D + L + L_r + W_t + T)$ (3-1)

Along with nominal material strength and strength reduction factors, the following SD load combinations for Performance Category 3 shall be considered:

(a) $D + W_t$
(b) $D + L + L_r + W_t$
(c) $D + L + L_r + W_t + T$ (3-2)

where:

W_t = tornado loading, including APC, as appropriate.

The notation and rationale for these load combinations are explained in Appendix D.

Careful attention should be paid to the details of construction. Continuous load paths shall be maintained; redundancy shall be built into load-carrying structural systems; ductility shall be provided in elements and connections to prevent sudden and catastrophic failures.

Two tornado missiles are specified as minimum criteria for this Performance Category. The 2x4-in. timber plank weighing 15 lbs is assumed to travel in a horizontal direction at speeds up to 100 mph. The horizontal speed is effective up to a height of 150 ft above ground level. If carried to great heights by the tornado winds, the timber plank can achieve a terminal vertical speed of 70 mph in falling to the ground. The horizontal and vertical speeds are assumed to be uncoupled and should not be combined. Table 3-4 describes wall and roof structures that will resist the postulated timber missile. A second missile to be considered is a 3-in. diameter standard steel pipe, which weighs 75 lbs. Design horizontal impact speed is 50 mph; terminal vertical speed is 35 mph. The horizontal speed of the steel pipe is effective up to a height of 75 ft above ground level. Table 3-4 summarizes certain barrier configurations that have been successfully tested to resist the pipe missile. Although wind pressure, APC and missile impact loads can occur simultaneously, the missile impact loads can be treated independently for design and evaluation purposes.

Table 3-4 Recommended Tornado Missile Barriers for Performance Category 3

Missile Criteria	Recommended Missile Barrier
Horizontal Component: 2x4 timber plank 15 lb @ 100 mph max. height 150 ft. above ground	8-in. CMU wall with one #4 rebar grouted in each vertical cell and trussed horiz joint reinf @ 16 in. on center
	Single wythe brick veneer attached to stud wall with metal ties
	4 in. concrete slab with #3 rebar @ 6 in. on center each way in middle of slab
Vertical Component: 2x4 timber plank 15 lb @ 70 mph	4 in. concrete slab with #3 rebar @ 6 in. on center each way in middle of slab
Horizontal Component: 3-in. diameter steel pipe 75 lb @ 50 mph max. height 75 ft. above ground	12-in. CMU wall with #4 rebar in each vertical cell and grouted, #4 rebar horizontal @ 8 in. on center
	Nominal 12-in. wall consisting of 8-in. CMU with #4 rebar in each vertical cell and grouted; #4 rebar horizontal @ 8 in. on center; single wythe brick masonry on outside face, horizontal ties @ 16 in. on center
	9.5- in. reinforced brick cavity wall with #4 rebar @ 8 in. on center each way in the cavity; cavity filled with 2500 psi concrete, horizontal ties @ 16 in. on center
	8-in. concrete slab with #4 rebar @ 8 in. on center each way placed 1.5 in. from each face
Vertical Component: 3-in. diameter steel pipe 75 lb @ 35 mph	6-in. concrete slab with #4 rebar @ 12 in. on center each way 1.5 in. from inside face

3.2.4 Performance Category 4

The performance goal for Performance Category 4 requires more conservative criteria than Performance Category 3. In some geographic regions, tornadoes must be considered.

Straight Winds and Hurricanes

For those sites where tornadoes are not a viable threat, the recommended basic wind speed is based on an annual exceedance probability of $1x10^{-4}$. 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 in ASCE 7 to account for the additional threat of hurricanes.

Once the design wind speeds are established and the importance factors applied, determination of Performance Category 4 wind loads is identical to Performance Category 3, except as noted below. SSCs in category Performance Category 4 may be designed or evaluated by ASD or SD methods, as appropriate for the material being used in construction. As with Performance Category 3, the wind hazard exceedance probability contributes a larger percentage of the total probabilistic performance goal than Performance Categories 1 or 2. Less conservatism is needed in the design and evaluation procedure. The degree of conservatism for Performance Category 4 is the same as Performance Category 3. Thus, the load combinations for both the ASD and SD are the same for Performance Categories 3 and 4.

Although the design wind speeds in Performance Category 4 are larger than Performance Category 3, the same missiles are specified (Table 3-3), except the maximum height above ground is 50 ft instead of 30 ft for Performance Category 4.

Tornadoes

For those sites requiring design for tornadoes, the criteria are based on site-specific studies as presented in Reference 3-2. Again, as with Performance Category 3, an annual exceedance probability of $1x10^{-4}$ could be justified. However, for the same reasons given for Performance Category 3, a lower value is recommended. The basic tornado wind speed associated with an annual exceedance probability of $2x10^{-6}$ and an importance factor of 1.0 is recommended. [Note: In UCRL-15910, the design wind speed was determined by selecting a wind speed associated with an exceedance probability of $2x10^{-5}$ and multiplying by an importance factor of 1.35]. The latter approach gives a more consistent exceedance probability

for all sites, although the design wind speeds are essentially the same. Once the basic tornado wind speed is determined for the specified annual exceedance probability and converted to fastest-mile, the procedure is as described for Performance Category 3, except as noted below.

Three tornado missiles are specified for Performance Category 4: a timber plank, a steel pipe and an automobile. The 2x4 timber plank weighs 15 lbs and is assumed to travel in a horizontal direction at speeds up to 150 mph. The horizontal component of the timber 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 vertical speed of 100 mph as it falls to the ground. 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 up to a height of 100 ft above ground level. The horizontal and vertical speeds of the plank and pipe are uncoupled and should not be combined. The third missile is a 3000-lb automobile that is assumed to roll and tumble along the ground at speeds up to 25 mph. Table 3-5 lists wall barrier configurations that have been tested and successfully resisted the timber and pipe missile. Impact of the automobile can cause excessive structural response to SSCs. Impact analyses should be performed to determine specific effects. In structures, collapse of columns, walls or frames may lead to further progressive collapse. Procedures for structural response calculations for automobile impacts is given in References 3-9, 3-10 and 3-11. Although wind pressure, APC, and missile impact loads can occur simultaneously, the missile impact loads can be treated independently for design and evaluation purposes.

Table 3-5 Recommended Tornado Missile Barriers for Performance Category 4

Missile Criteria	Recommended Missile Barrier
Horizontal Component: 2x4 timber plank 15 lb @ 150 mph max. height 200 ft. above ground	6 in. concrete slab with #4 rebar @ 6 in. on center each way in middle of slab
	8-in. CMU wall with one #4 rebar grouted in each vertical cell and horiz trussed joint reinf @ 16 in. on center
Vertical Component. 2x4 timber plank 15 lb @ 100 mph	4 in. concrete slab with #3 rebar @ 6 in. on center each way in middle of slab
Horizontal Component. 3-in. diameter steel pipe 75 lb @ 75 mph max. height 100 ft. above ground	10-in. concrete slab with #4 rebar @ 12 in. on center each way placed 1.5 in. from each face
Vertical Component. 3-in. diameter steel pipe 75 lb @ 50 mph	8-in. concrete slab with #4 rebar @ 8 in. on center each way placed 1.5 in. from inside face

3.2.5 Design Guidelines

Reference 3-12 provides guidelines and details for achieving acceptable wind resistance of SSCs. Seven principles should be followed in developing a design that meets the performance goals:

(a) Provide a continuous and traceable load path from surface to foundation
(b) Account for all viable loads and load combinations
(c) Provide a redundant structure that can redistribute loads when one structural element is overloaded
(d) Provide ductile elements and connections that can undergo deformations without sudden and catastrophic collapse
(e) Provide missile resistant wall and roof elements
(f) Anchor mechanical equipment on roofs to resist specified wind and missile loads
(g) Minimize or eliminate the potential for windborne missiles

3.3 Evaluation of Existing SSCs

The objective of the evaluation process is to determine if an existing SSC meets the performance goals of a particular Performance Category.

The key to the evaluation of existing SSCs is to identify potential failure modes and to calculate the wind speed to cause the postulated failure. A critical failure mechanism could be the failure of the main wind-force resisting system of a structure or a breach of the structure envelope that allows release of toxic materials to the environment or results in wind and water damage to the building contents. The structural system of many old facilities (25 to 40 years old) have considerable reserve strength because of conservatism used in the design, which may have included a design to resist abnormal effects. However, the facility could still fail to meet performance goals if breach of the building envelope is not acceptable.

The weakest link in the load path of an SSC generally determines the adequacy or inadequacy of the performance of the SSC under wind load. Thus, evaluation of existing SSCs normally should focus on the strengths of connections and anchorages and the ability of the wind loads to find a continuous path to the foundation or support system.

Experience from windstorm damage investigations provide the best guidelines for anticipating the potential performance of existing SSCs under wind loads. Reference 3-13 provides a methodology for estimating the performance of existing SSCs. The approach is directed primarily to structures, but can be adapted to systems and components as well. The methodology described in Reference 3-13 involves two levels of evaluation. Level I is essentially a screening process and should normally be performed before proceeding to Level II, which is a detailed evaluation. The Level II process is described below. The steps include:

(a) Data collection
(b) Analysis of element failures
(c) Postulation of failure sequence
(d) Comparison of postulated performance with performance goals

3.3.1 Data Collection

Construction or fabrication drawings and specifications are needed to make an evaluation of potential performance in high winds. A site visit and walkdown is usually required to verify that the SSCs are built according to plans and specifications. Modifications not shown on the drawings or deteriorations should be noted.

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

3.3.2 Analysis of Element Failures

After determining the as-is condition and the material properties, various element failures of the SSCs are postulated. Nominal strength to just resist the assumed element failure is calculated. Since the nominal strength is at least equal to the controlling load combination, the wind load to cause the postulated failure can be calculated. Knowing the wind load, the wind speed to produce the wind load is determined using the procedures of ASCE 7 and working backwards. Wind speeds to cause all plausible failure modes are calculated and tabulated. The weakest link is determined from the tabulation of element failures. These are then used in the next step to determine the failure sequence.

3.3.3 Postulation of Failure Sequence

Failure caused by wind is a progressive process, initiating with an element failure. Examples are failure of a roof to wall connection, inward or outward collapse of an overhead door, window glass broken by flying roof gravel. Once the initial element failure occurs at the lowest calculated wind speed, the next event in the failure sequence can be anticipated. For example, if a door fails, internal pressure inside the building will increase causing larger outward acting pressure acting on the roof. The higher pressures could then lead to roof uplift creating a hole in the roof itself. With the door opening and roof hole, wind could rapidly circulate through the structure causing collapse of partition walls, damage to ceilings or ventilation systems or transportation of small objects or debris in the form of windborne missiles. Each event in the sequence can be associated with a wind speed. All obvious damage sequences should be examined for progressive failure.

3.3.4 Comparison of Postulated Failures with Performance Goals

Once the postulated failure sequences are identified, the SSC performance is compared with the stated performance goals for the specified Performance Category. The general SSC evaluation procedures described in Apendix B(Figure B-2) are followed. If an SSC is able to survive wind speeds associated with the performance goal, the SSC meets the goal. If the performance criteria are not met, then the assumptions and methods of analyses can be modified to eliminate conservatism introduced in the evaluation methods. The acceptable hazard probability levels can be raised slightly, if the SSC comes close to meeting the performance goals. Otherwise, various means of retrofit should be examined. Several options are listed below, but the list is not exhaustive:

(a) Add x-bracing or shear walls to obtain additional lateral load resisting capacity

(b) Modify connections in steel, timber or prestressed concrete construction to permit them to transfer moment, thus increasing lateral load resistance in structural frames

(c) Brace a relatively weak structure against a more substantial one

(d) Install tension ties that run from roof to foundation to improve roof anchorage

(e) 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 structure envelope or to reduce the consequences of missile perforation, the following general suggestions are presented:

(a) Install additional fasteners to improve cladding anchorage
(b) Provide interior barriers around sensitive equipment or rooms containing hazardous materials
(c) Eliminate windows or cover them with missile-resistant grills
(d) Erect missile resistant barriers in front of doors and windows
(e) Replace ordinary overhead doors with heavy-duty ones that will resist the design wind loads and missile impacts. The door tracks must also be able to resist the wind loads.

Each class of SSC will likely have special situations that need attention. Personnel who are selected to evaluate existing facilities should be knowledgable of the behavior of various SSC classes subjected to extreme winds.

3.4 References

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3-5. Los Alamos National Laboratory, **TVENT,** Los Alamos, NM, 1979.

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3-8. American Society of Civil Engineers, **Structural Analysis and Design of Nuclear Plant Facilities,** Manuals and Reports on Engineering Practice - No. 58, 1980.

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Chapter 4
Flood Design and Evaluation Criteria

4.1 Flood Design Overview

The flood design and evaluation criteria seek to ensure that safety structures, systems and components (SSCs) at DOE sites satisfy the performance goals described in the NPH Implementation Guide for DOE Order 420.1. These criteria consider the design of SSCs for regional flood hazards (i.e., river flooding) and local precipitation that effects roof design and site drainage. This chapter describes the flood criteria, presents the design basis flood (DBFL) that must be considered in flood design, presents the criteria for the design of civil engineering systems (e.g., structures, site drainage, roof systems and roof drainage, etc.) and presents alternative design strategies for flood hazards. Guidance is also provided to evaluate existing SSCs that may not be located above the DBFL, to assess whether the performance goals are satisfied. Determination of the DBFL shall be accomplished in accordance with DOE-STD-1023 (Ref. 4-1).

Table 4-1 provides the flood criteria for Performance Categories 1 through 4. The criteria are specified in terms of the flood hazard input, hazard-annual probability, design requirements, and emergency operation plan requirements. The hazard annual probability levels in Table 4-1 correspond to the mean hazard.

Evaluation of the flood design basis for SSCs consists of:

1. determination of the DBFL for each flood hazard as defined by the hazard annual probability of exceedance and applicable combinations of flood hazards,
2. evaluation of the site stormwater management system (e.g., site runoff and drainage, roof drainage),
3. development of a flood design strategy for the DBFL that satisfies the criteria performance goals (e.g., build above the DBFL, harden the facility), and
4. design of civil engineering systems (e.g., buildings, buried structures, site drainage, retaining walls, dike slopes, etc.) to the applicable DBFL and design requirements.

Each of these areas is briefly described in the following subsections

Table 4-1 Flood Criteria Summary

Item	Performance Category			
	1	2	3	4
Flood Hazard Input	Flood insurance studies or equivalent input, including the combinations in Table 4-2	Site probabilistic hazard analysis, including the combinations in Table 4-2	Site probabilistic hazard analysis, including the combinations in Table 4-2	Site probabilistic hazard analysis, including the combinations in Table 4-2
Mean Hazard Annual Probability	$2x10^{-3}$	$5x10^{-4}$	$1x10^{-4}$	$1x10^{-5}$
Design Requirements	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 criteria.			
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.	

4.1.1 Design Basis Flood (DBFL)

As part of the flood hazard assessment[1] that is performed for a site, the sources of flooding (e.g., rivers, lakes, local precipitation) and the individual flood hazards (e.g., hydrostatic forces, ice pressure, hydrodynamic loads) are identified. A site or individual SSC 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 stormwater management system to handle the runoff due to local (on-site or near site) 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. Therefore, at a site there may be multiple DBFLs that are considered. For sites with potential for river flooding a DBFL is determined for river flooding and for local precipitation which determines the design of the site stormwater management systems. (Note, for sites located on rivers or streams, the meteorologic and hydrologic events that produce intense local precipitation are often distinct from those which produce high river flows). In this instance, various aspects of the design for a SSC may be determined by different flood hazards. As a result, the term DBFL is used in a general sense that applies to the multiple flood hazards that may be included in the design basis.

[1] Guidelines for conducting a probabilistic flood hazard assessment are contained in (DOE-STD-1023). The analysis includes an evaluation of the hydrologic and hydraulic characteristics of a site and site region. As part of the probabilistic assessment, an evaluation of uncertainty is also performed.

Table 4-2 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, as a minimum to the 2-year wind acting in the most favorable direction (Ref. 4-2), coincident with the peak flood or as determined in a probabilistic analysis that considers the joint occurrence of river flooding and wind generated waves.
	3	Ice forces (Refs. 4-2 and 4-3) and Case 1.
	4	Evaluate the potential for erosion, debris, etc. due to the primary hazard.
Dam Failure	1	All modes of dam failure must be considered (i.e., overtopping, seismically induced failure, random structural failures, upstream dam failure, etc.)
	2	Wind-waves corresponding, as a minimum to the 2-year wind acting in the most favorable direction (Ref. 4-2), coincident with the peak flood or as determined in a probabilistic analysis that considers the joint occurrence of river flooding and wind generated waves.
	3	Evaluate the potential for erosion, debris, etc. due to the primary hazard.
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 applicable regulations.
Storm Surge, Seiche (due to hurricane, seiche, squall lines, etc.)	1	Tide effects corresponding to the mean high tide above the MLW** level (if not included in the hazard analysis).
	2	Wave action and Case 1. Wave action should include static and dynamic effects and potential for erosion (Ref. 4-2)
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 applicable regulations
Tsunami	1	Tide effects corresponding to the mean high tide above the MLW** level (if not included in the hazard analysis).

* Events are added to the flood level produced by the primary hazard
** MLW-Mean Low Water.

The DBFL for a SSC for a 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 4-1), including the combination of flood hazards (e g . river flooding and wind-wave action) given in Table 4-2, and

2. Corresponding loads associated with the DBFL peak-hazard level and applicable load combinations (e.g., hydrostatic and/or hydrodynamic forces, debris loads).

The first item is determined as part of the probabilistic flood hazard assessment. Limited flood hazard assessments for some DOE sites have been conducted (see Refs. 4-4, 4-5, and 4-6). Flood loads are assessed for the DBFL on a SSC-by-SSC basis.

Table 4-2 defines the flood design basis events that must be considered. The events listed in Table 4-2 should be considered as part of the site flood hazard assessment. For example, if a river is a source of flooding, wind waves must be considered. The DBFL is determined by entering the flood hazard curve which includes the combination of events in Table 4-2. For example, at a site located on an ocean shore, the flood hazard curve should include the effects of storm surge, tides and wind-waves.

If the hazard annual probability for a primary flood hazard is less than the design basis hazard annual probability for a given Performance Category (see Table 4-1), it need not be considered as a design basis event. For instance, if the hazard annual probability for Performance Category 1 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 $2x10^{-3}$.

4.1.2 Flood Evaluation Process

The following describes the steps involved in the evaluation of SSCs. The procedure is general and applies to new and existing construction. It is oriented toward the evaluation of individual SSCs. However, due to the nature of flood events (i.e., river flooding may inundate a large part of a site and thus many SSCs simultaneously), it may be possible to perform an evaluation for the entire site or a group of SSCs.

The flood evaluation process is illustrated in Figure 4-1. It is divided into the consideration of regional flood hazards and local precipitation. For new construction, design practice (see Section 4.1.3) is to construct the SSC above the DBFL, thus avoiding the flood hazard and eliminating the consideration of flood loads as part of the design. The design of the site stormwater management system and structural systems (i.e., roofs) for local precipitation must be adequate to prevent flooding that may damage a SSC or interrupt operations to the extent that the performance goals are not satisfied.

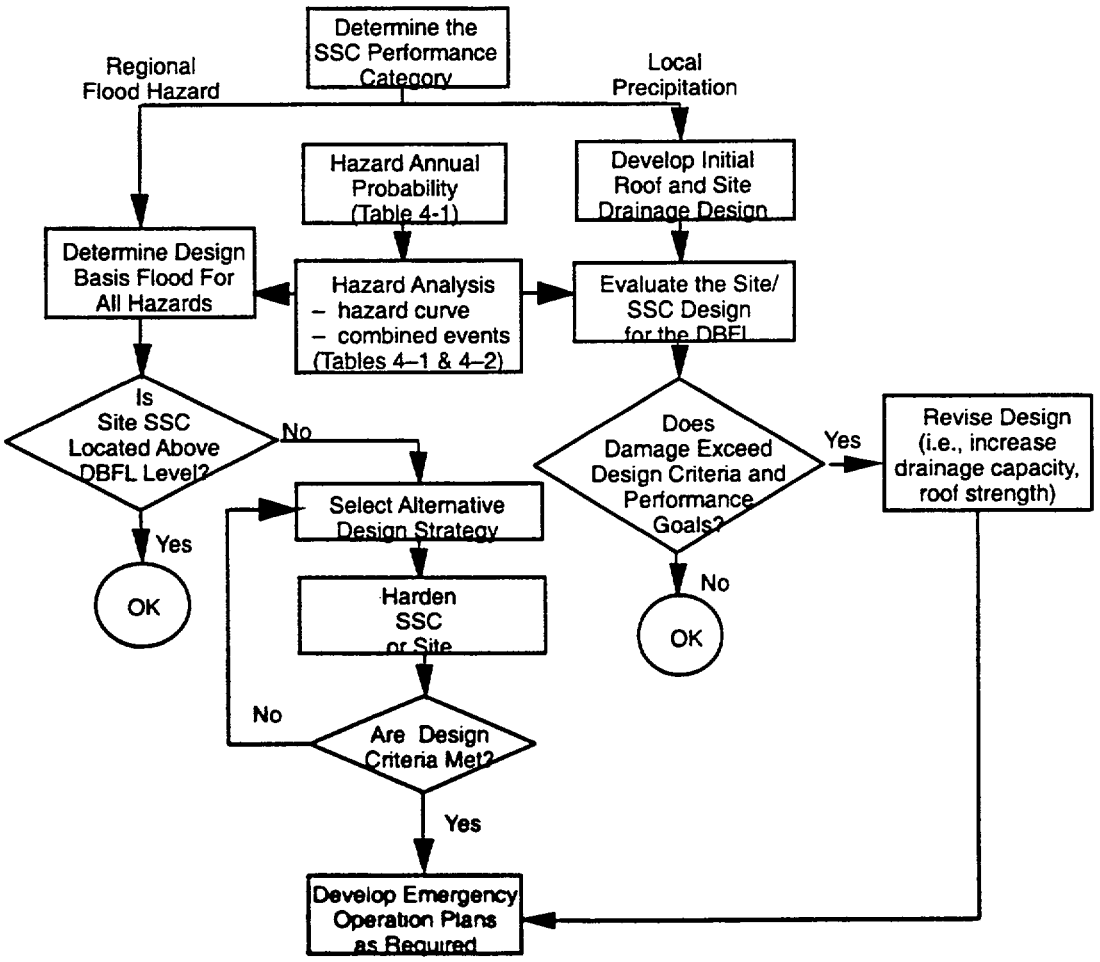

Figure 4-1 Flood Evaluation Process

To perform the flood evaluation for a SSC, the results of a flood screening analysis (as a minimum) or a probabilistic flood hazard analysis should be available (Refs 4-4 to 4-6). The steps in the flood evaluation process include:

1. Determine the SSC Performance Category (see Chapter 2 and DOE-STD-1021).

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

2. Determine the DBFL for each type or source of flooding (see Tables 4-1 and 4-2). The assessment of flood loads (e.g., hydrostatic and hydrodynamic loads) or other effects (e.g., scour, erosion) is made on an SSC-by-SSC basis.

3. For new construction locate the SSC above the DBFL, if possible. If this cannot be done, proceed to Step 4.

4. Develop a design strategy to mitigate flood hazards that impact the SSC. Options include hardening the SSC, modifying the flood path, and developing emergency operation plans to provide for occupant safety and to secure vulnerable areas. The flood hazard must be mitigated such that the performance goals are met.

5. If the SSC is located below the DBFL level (even if the SSC has been hardened), emergency procedures must be provided to evacuate personnel and to secure the SSC prior to the arrival of the flood (see Step 10).

Evaluation for Local Precipitation

6. Develop an initial site-drainage system and roof-system drainage plan and structural design per applicable regulations. Typical stormwater management systems are designed for not less than the 25-year, 6-hour storm. The minimum storm sewer size is typically 12 inches and the minimum culvert size 15 inches. For roof drain systems, the minimum pipe size for laterals and collectors are typically 4 inches. Stormwater management systems usually have sufficient capacity to ensure that runoff from the 100 year, 6-hour design storm will not exceed a depth of 0.87 feet at any point within the street right-of-way or extend more than 0.2 feet above the top of the curb in urban streets.

7. Perform a hydrological analysis for the site to evaluate the performance of the site stormwater management system (considering roof drainage and man-made and natural watercourses) for the DBFL local precipitation for each SSC. The site analysis must determine the level of flooding (if any) at each SSC. Guidelines for performing a hydrological analysis are contained in DOE-STD-1023 and DOE-STD-1022.

   For SSCs where flooding occurs, the engineer must assess whether the performance goals are satisfied. If the SSC performance is unsatisfactory, a modification of the site stormwater management system is required (see Step 9). Due to the different Performance Category DBFLs, this step may be performed for a number of flood events.

8. Evaluate the drainage and structural design of roof systems for the DBFL local precipitation. The structural design of the roof system will satisfy design criteria for loads due to ponding that result from clogged/blocked drains and snow and ice loads. These were either developed during the design of existing facilities or will be those from applicable regulations. If the design criteria for the roof is exceeded (i.e., deflection, stress allowables), the design must be revised (see Step 9).

9. If the DBFL for a SSC due to local precipitation produces levels of flooding such that the performance goals (i.e., damage level due to inundation or exceedance of design criteria allowables), are not satisfied, design modifications must be developed. The design modifications must provide additional capacity (i.e., runoff capacity, additional strength) to satisfy the performance goals. Alternative design strategies are discussed in Section 4.1.3.

10. For SSCs that are impacted by the DBFL, emergency operation plans must be developed to provide for the safety of personnel and to secure critical areas to satisfy performance goals.

In principle, each SSC is designed in accordance with the requirements for the applicable Performance Category. However, because floods have a common-cause impact on SSCs that are in proximity to one another, the design basis for the most critical SSC may govern the design for other SSCs or for the entire site. Stated differently, it may be more realistic economically and functionally to develop a design strategy that satisfies the performance goals of the most critical SSC and simultaneously that of other SSCs. For example, it may be feasible to harden a site (e.g., construct a levee system), thus protecting all SSCs. Conversely, it may be impractical to develop a design strategy that protects the entire site when SSC locations vary substantially (i.e., they are at significantly different elevations or there are large spatial separations).

The possible structural or functional interaction between SSCs should be considered as part of the evaluation process. For example, if an SSC in Performance Category 4 requires emergency electric power in order to satisfy the performance goals, structures that house emergency generators and fuel should be designed to the DBFL for the Performance Category 4 SSC. In general, a systematic review of a site for possible structural or functional

dependencies is required. As an aid to the review, the analyst can develop a logic model that displays the functional/structural dependencies between SSCs.

4.1.3 Flood Design Strategies

The basic design strategy for SSCs in Performance Categories 2 to 4 (excluding local precipitation), is to construct the SSC above the DBFL. When this can be done, flood hazards are not considered in the design basis except that possible raised ground water level must be considered. The flood criteria have been established with this basic strategy in mind. Note that local precipitation is an exception since all sites must consider this hazard in the design of the site stormwater management system, roof systems, etc.

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

1. Situate the SSC above the DBFL level,
2. Modify the flood, or
3. Harden the site or SSC to mitigate the effects of the DBFL such that the performance goals are satisfied, and
4. Establish emergency operation plans to safely evacuate employees and secure areas with hazardous, mission-dependent, or valuable materials.

If an SSC is situated above the DBFL, the performance goals are readily satisfied. If an SSC is located below the DBFL, alternatives can be considered to modify the magnitude of the flood or mitigate its effects such that the likelihood of damage and interruption of operations is acceptably low (i.e., performance goals are satisfied). In addition, emergency operation plans must be developed that establish the procedures to be followed to recognize/identify the flood hazard in a timely manner and 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 do not adequately limit the level of damage and interruption to facility operations.

Under certain circumstances the flood can be modified to limit the magnitude of the hazard. Alternatives include the construction of detention ponds that provide for the collection and controlled release of runoff on-site, modification of stream channels, etc.

The strategy of hardening a SSC or site and providing emergency operation plans is secondary to siting facilities above the DBFL level because some probability of damage does exist and SSC operations may be interrupted. If it is determined that a SSC may be impacted by the DBFL and thus must be hardened, the designer must determine the flood loads associated with the DBFL. The design of flood mitigation systems (i.e., exterior walls, flood-proof doors, etc.) must be conducted in accordance with the requirements specified in applicable regulations.

The evaluation of the site stormwater management system and roof design (i.e., drainage and structural capacity) differs somewhat from that for other flood hazards. First, all sites must be designed for the effects of local precipitation. Secondly, from the perspective of the performance goals, the adequacy of the site stormwater management system is measured in terms of the impact of local flooding on SSCs at the site. For example, the initial design of the site stormwater management system may correspond to the 25-year rainfall 6-hour storm. If the DBFL for a SSC corresponds to a $5x10^{-4}$ rainfall, the site stormwater management system design clearly does not meet this criterion. 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. The actual impact of the DBFL precipitation on the SSC is assessed by conducting a hydrologic evaluation for the site that accounts for natural and man-made watercourses on site, roof drainage, etc. The analysis may conclude that flooding is limited to streets and parking lots. If temporary flooding in these areas does not significantly affect the operation and safety of the SSC, then it may be concluded that the design of the site-drainage system (i.e., for the 25-year rainfall) is adequate. Conversely, if flooding does result in significant damage which impairs the operation or safety of SSCs, appropriate measures must be taken to satisfy the performance goals. This may include increasing the capacity of the drainage system, constructing detention ponds on site, or hardening an SSC against the effects of flooding caused by local precipitation.

4.2 Flood Design Criteria

Unlike design strategies for seismic and wind hazards, it is not always possible to provide margin in the flood design of a SSC. For example, the simple fact that a site is inundated (even if structural damage does not occur), will cause significant disruption (e.g., down time during the flood, clean-up). This is often unacceptable in terms of the economic impact and disruption of the mission-dependent function of the site. Under these circumstances, there is no margin, as used in the structural sense that can be provided when a site or SSC is inundated. Therefore, the SSC must be kept dry and operations must not be interrupted in order to satisfy the performance goals. Since a risk reduction cannot, in general,

be specified, the hazard annual probability is set to the performance goal probability of damage with the exception of Performance Category 1. For Performance Category 1, a risk reduction corresponding to a factor of 2 is defined. This risk reduction is based on the limited warning time that is required to evacuate personnel from an area that may be flooded (Ref. 4-7).

The DBFL for Performance Category 1 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 this Performance Category it is not necessary that a detailed site-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 Performance Categories 2 through 4, a comprehensive site-specific flood hazard assessment should be performed, unless the results of a screening analysis (see References 4-4 and 4-5) demonstrates that the performance goals are satisfied.

4.2.1 Performance Category 1

The performance goal for Performance Category 1 specifies that occupant safety be maintained and that the probability of severe structural damage be less than or about 10-3 per year. The mean hazard annual probability of exceedance is $2x10^{-3}$. In addition, event combinations that must be considered are listed in Table 4-2.

To meet the performance goal for this category, two requirements must be met: (1) the building structural system must be capable of withstanding the forces associated with the DBFL, and (2) adequate time for warning must be available to ensure that building occupants can be evacuated (i.e., 1 to 2 hours, Ref. 4-7). If the building is located above the DBFL, then structural and occupant safety requirements are met.

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

1. Modifying the flood or providing flood protection for the site or for the specific structure, such that severe structural damage does not occur, and
2. Developing emergency procedures in order to provide adequate warning and evacuation capability to provide for the safety of building occupants.

For structural loads applied to roofs, exterior walls, etc., the applicable regulations should be used.

4.2.2 Performance Category 2

The performance goal for Performance Category 2 is to limit damage and interruption of operations while also maintaining occupant safety. The DBFL is equal to the flood whose annual probability of exceedance is $5x10^{-4}$ per year including the event combinations listed in Table 4-2. For purposes of establishing the DBFL for Performance Category 2, a site-specific hazard assessment should be performed. This analysis must include the uncertainty in the hazard assessment in order to obtain an accurate estimate of the mean-annual probability level.

SSCs in this category should be located above the DBFL. For SSCs that cannot be located above the DBFL, an acceptable design can be achieved by the same measures described for Performance Category 1. Emergency operation plans must be developed to provide for occupant safety and to mitigate the damage to mission-dependent SSCs. 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 SSCs in Performance Category 1, applicable regulations should be used to incorporate flood loads in the building design.

4.2.3 Performance Category 3

The performance goal for Performance Category 3 is continued function of the facility, including confinement of hazardous materials and occupant safety. SSCs 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 4-2.

If SSCs in this category cannot be constructed above the DBFL level, a design must be developed that provides continued facility operation. The strategy must mitigate the flood (i.e., modifying the flood, hardening the facility, building a levee to prevent flood encroachment) to an extent that facility operations can continue. A higher level of protection is required for SSCs in Performance Category 3 as compared to Categories 1 and 2. Limited damage and interruption of operations may be acceptable for Performance Categories 1 and 2, however, for Performance Category 3 the DBFL must be mitigated such that the flood does not impact operations.

The design of Performance Category 3 SSCs that may be impacted by the DBFL should be based on the loads (i.e., hydrostatic forces) and other hazards (i.e., ice forces, debris) that occur. If mitigation systems such as watertight doors, sealants, etc. are used, manufacturer

specifications should be applied. Section 4.3 describes design requirements for flood-mitigation systems such as levees, dikes, etc.

For SSCs 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 who may have 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.

4.2.4 Performance Category 4

The performance goals for Performance Category 4 are basically the same as for Performance Category 3. However, a higher confidence is required that the performance goals are met. SSCs in this category should be located above flood levels whose mean-annual probability of exceedance is 10^{-5}, including the combinations of events listed in Table 4-2.

4.3 Flood Design Practice for SSCs Below the DBFL Elevation

For SSCs 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.

4.3.1 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. 4-8 to 4-12), including the forces due to ice formation on bodies of water.

4.3.2 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 applicable regulations. The design criteria are to be used in conjunction with flood loads and effects derived from the SSCs DBFL (see Tables 4-1 and 4-2).

4.3.2.1 Performance Categories 1 and 2

Facilities that are subject to flood loads should be designed according to provisions in applicable regulations. Design loads and load combinations are determined from the DBFL. Load factors specified in or other applicable regulations shall be used.

4.3.2.2 Performance Categories 3 and 4

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. 4-13). Design loads and load combinations are determined from the DBFL. Load factors specified in applicable regulations shall be used.

4.3.3 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 other applicable regulations. Applicable local regulations must be considered in the design of the site stormwater management system. A typical 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 SSC. 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 SSC. The analyst may choose to evaluate the site stormwater management system for the highest category DBFL (as a limiting case). If the results of this analysis demonstrate that flooding does not compromise the site SSCs, then it may be concluded that the site stormwater management system 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 in accordance with applicable 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 are specified by reference 4-15 or other applicable local regulations. The roof should be designed to consider the maximum depth of water that could accumulate if the primary-drainage system is blocked (Refs. 4-14, and 4-15).

Roof-drainage systems should be designed according to applicable regulations. The drainage system should be verified as part of the site analysis for the DBFL (discussed above). 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 secondary 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 applicable codes. Design criteria for snow and rain-on-snow loads are defined in the model building codes and standards.

DOE criteria specify the importance factors that should be used to scale snow loads in the design. In the design of roof systems for snow loads, the importance factor for Performance Categories 1 and 2 is 1.0. For Performance Categories 3 and 4 an importance factor of 1.2 should be used.

4.3.4 Flood Protection and Emergency Operations Plans

For SSCs 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 an SSC must withstand, various hardening systems may be considered. These include,

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

The design of structural systems (i.e., exterior building walls) shall be developed in accordance with applicable regulations. Waterproofing requirements are also given in

applicable design standards. Guidelines for the design of earth structures such as levees, seawalls, etc. are provided in References 4-8, 4-16, and 4-17. 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. 4-12 and 4-16).

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 an emergency operation plan 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 - program to insure 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. The development of the emergency operation plan should be coordinated with the results of the flood hazard assessment and local agencies responsible for flood forecasting. The availability of warning time will vary depending on the type of flood hazard and local forecasting capabilities. Specific information on flood emergency procedures can be found in Reference 4-18.

4.4 Considerations for Existing Construction

Existing SSCs may not be situated above the DBFL as defined by these criteria. In this case, an SSC should be reviewed to determine the level of flooding, if any, that can be sustained, without exceeding the performance goal requirements. This is referred to as the Critical Flood Elevation (CFE). If the CFE is higher than the DBFL, then the performance goals 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 example, it may be possible to wet proof an SSC, thus allowing some level of flooding to occur.

For each SSC, there is a critical elevation, which if exceeded, causes damage or disruption such that the performance goal is not satisfied. The CFE may be located:

- below grade due to the structural vulnerability of exterior walls or instability due to uplift pressures,
- at the elevation of utilities that support SSCs, or
- at the actual base elevation of an SSC.

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 an SSC, it may be determined that greater flood depths must occur to cause damage (e.g., critical equipment or materials may be located above the first floor). If the CFE for an SSC exceeds the DBFL, then the performance goal is satisfied. If the CFE does not exceed the DBFL, options must be considered to harden the SSC, change the Performance Category, etc.

For Performance Categories 3 and 4, 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 operations (i.e, uninterrupted access) as well as the damage to the physical systems.

4.5 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 to require a risk assessment. These 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 one of the four Performance Categories.

The objective of a risk assessment is to evaluate the risk of damage to SSCs important for maintaining safety and site operations. 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 4-19 to 4-22.

4.6 References

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Appendix A
Terminology and Definitions

Note: - Definitions common to DOE Order 420.1, the accompanying Implementation Guide, this Standard, and other NPH Standards are contained in the NPH Implementation Guide.

Annual Probability of Exceedance - The likelihood of natural phenomena hazards must be evaluated on a probabilistic basis for this performance goal based NPH criteria. The frequency of occurrence of parameters describing the external hazard severity (such as earthquake ground acceleration, wind speed, or 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. A given event of return period, T, is equally likely to occur any year, thus the probability of that event being exceeded in any one year is 1/T. The annual probability of exceedance, p, of an event is the reciprocal of the return period of that event (i.e., $p = 1/T$). 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.

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:

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

where EP and p vary from 0 to 1, 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^{-3}	0.05
10^{-4}	0.005
10^{-5}	0.0005

Hence, an event with a 10^{-3} annual probability of exceedance (1000 year return period) has a 5 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.

Performance Goal is the annual probability of exceedance of acceptable behavior limits used as a target to develop NPH design and evaluation criteria. Goals for structure, system, component (SSC) performance during natural phenomena hazards have been selected and expressed in terms of annual probability of exceedance. Numerical values of annual probabilities of exceedance for performance goals depend on SSC characteristics. For example, probability values specified for normal use SSCs are consistent with performance obtained through the use for model building code provisions for natural phenomena hazards. Probability values specified for hazardous use SSCs approach performance obtained through the use of nuclear power plant NPH criteria. Acceptable behavior limits considered in the performance goal also depend on the SSC characteristics. For example, the acceptable behavior limits for normal use SSCs is major damage but limited in extent to below that at which occupants are endangered. However, the acceptable behavior limits for hazardous use SSCs is lesser damage such that the facility can perform its function.

Performance goal probability values apply to each natural phenomena hazard (NPH) individually. Hence, the annual probability of exceedance of acceptable behavior limits for all NPH would be somewhat larger than the performance goal value if structures, systems, and components are designed exactly to the criteria in this document for all NPH.

Natural Phenomena Hazard Curves - The likelihood of earthquake, wind, and flood hazards at DOE sites can be defined by graphical relationships between ground acceleration, wind speed, or water elevation and 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 as illustrated in Figure A-1. There is considerable uncertainty in natural phenomena hazard curves which is not indicated by the single curve shown in Figure A-1. The means of accounting for this uncertainty is discussed in the different chapters on individual natural phenomena hazards

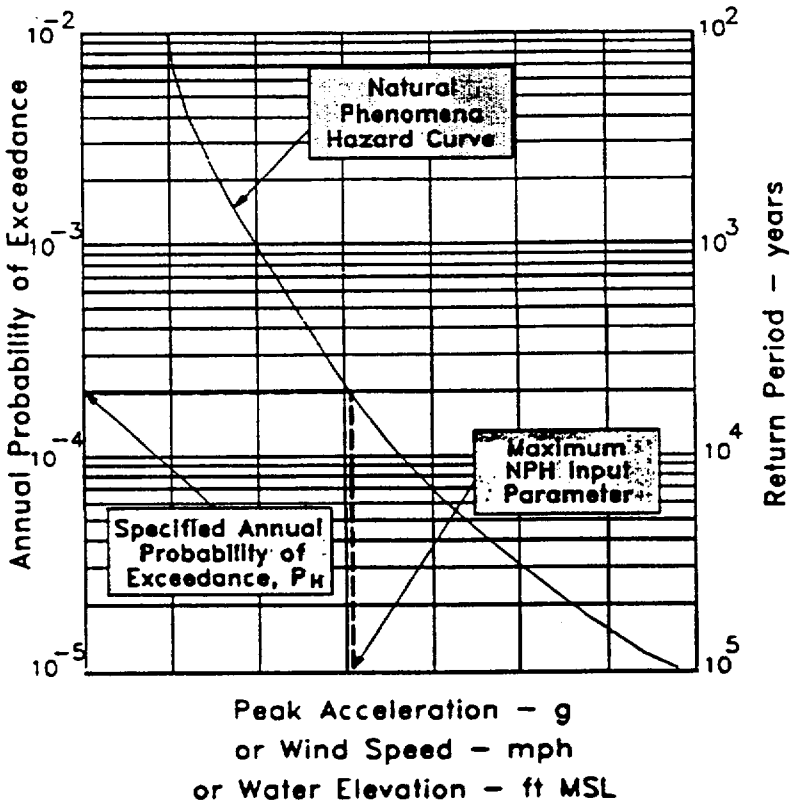

Figure A-1 Example Probabilistic Natural Phenomena Hazard Curve

Appendix B
Commentary on General NPH Design and Evaluation Criteria

B.1 NPH Design and Evaluation Philosophy

The natural phenomena hazard (NPH) design and evaluation criteria presented in this document (DOE-STD-1020) implement the requirements of DOE Order 420.1, "Facility Safety" (Ref. B-1) and the associated Implementation Guides: "Implementation Guide for the Mitigation of Natural Phenomena Hazards for DOE Nuclear Facilities and Non-nuclear Facilities" (Ref. B-2), "Implementation Guide for Nonreactor Nuclear Safety Design Criteria and Explosives Safety Criteria" (Ref. B-3), and "Implementation Guide for Use with DOE Orders 420.1 and 440.1 Fire Safety Program" (Ref. B-4) which 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 target performance goals expressed as an annual probability of exceedance of acceptable behavior limits (i.e., behavior limits beyond which damage/failure is unacceptable). DOE Order 420.1 and the associated Implementation Guides establish a graded approach for NPH requirements by defining performance categories (numbered 0 through 4) each with a qualitative performance goal for behavior (i.e., maintain structural integrity, maintain ability to function, maintain confinement of hazardous materials) and a qualitative target probabilistic performance goal. DOE-STD-1020 provides four sets of NPH design and evaluation criteria (explicit criteria are not needed for Performance Category 0). These criteria range from those provided by model building codes for Performance Category 1 to those approaching nuclear power plant criteria for Performance Category 4.

DOE-STD-1020 employs the graded approach by following the philosophy of probabilistic performance goal-based design and evaluation criteria for natural phenomena hazards. Target performance goals range from low probability of NPH-induced damage/failure to very high confidence of extremely low probability of NPH-induced damage/failure. In this manner, structures, systems, and components (SSCs) are governed by NPH criteria which are appropriate for the potential impact on safety, mission, and cost of those SSCs 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. SSCs containing hazardous materials which, in the event of damage, threaten public safety or the environment, and/or which have been determined to require special consideration, 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 model building code design and evaluation procedures). For ordinary SSCs of relatively low

cost, there is typically no need or requirement to add conservatism to the design beyond that of model building codes. For these SSCs, it is also typically not cost-effective to strengthen structures more than required by model building codes that consider extreme loads due to natural phenomena hazards.

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 annual probability of exceedance of SSC 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 criteria specify hazard annual probabilities of exceedance, response evaluation methods, and permissible behavior criteria for each natural phenomena hazard and for each performance category such that desired performance goals are achieved for either design or evaluation. The ratio of the hazard annual probability of exceedance and the performance goal annual probability of exceedance is called the risk reduction ratio, R_R in DOE-STD-1020. This ratio 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 ($R_R = 1$), 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 ($R_R > 1$). In the criteria 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 is to aim at the performance goal probability values which are applicable to each natural phenomena hazard separately.

Design and evaluation criteria are presented in Chapters 2, 3, and 4 for earthquake, wind, and flood hazards, respectively. These criteria are deterministic procedures that establish SSC loadings from probabilistic natural phenomena hazard curves; specify acceptable methods for evaluating SSC response to these loadings; provide acceptance criteria to judge whether computed SSC response is acceptable; and to provide detailing requirements such that behavior is as expected as illustrated in Figure B-1. These criteria are intended to apply equally for design of new facilities and for evaluation of existing facilities. In addition, the criteria are intended to cover buildings, equipment, distribution systems (piping, HVAC, electrical raceways, etc.), and other structures.

DOE-STD-1020 primarily covers (1) methods of establishing load levels on SSCs 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 NPH load levels and NPH response evaluation and acceptance criteria, the importance of these parts of the design/evaluation process should not be underestimated. Quality assurance and peer review are briefly addressed in Section 1.4, 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. Reference B-5 describes earthquake design considerations including detailing for ductility. Reference B-6 gives structural details for wind design.

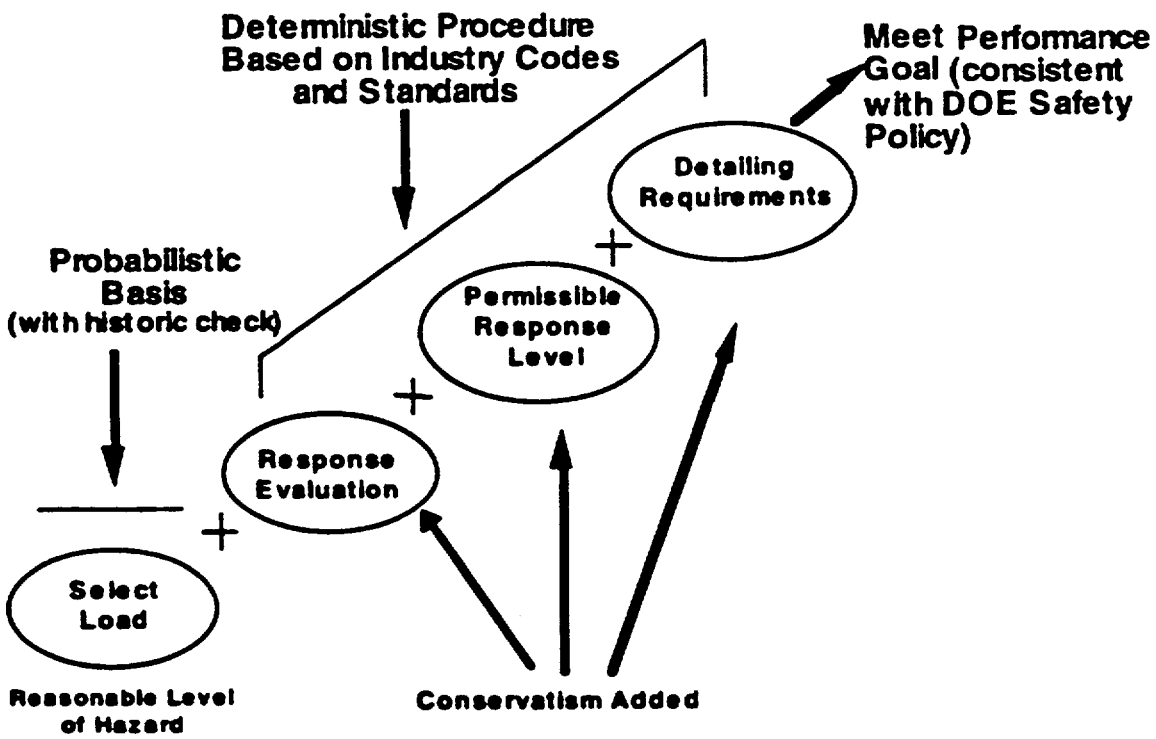

Figure B-1 DOE-STD-1020 Combines Various Methods to Achieve Performance Goals

B.2 Graded Approach, Performance Goals, and Performance Categories

As stated above, DOE Order 420.1 and the associated Implementation Guides establish a graded approach in which NPH requirements are provided for various performance categories each with a specified performance goal. The motivation for the graded approach is that it enables design or evaluation of DOE structures, systems, and components to be performed in a manner consistent with their importance to safety, importance to mission, and cost. There are only a few "reactor" facilities in the DOE complex and many facilities with a wide variety of risk potential, mission, and cost. Also, the graded approach enables cost-benefit studies and establishment of priorities for existing facilities. There are few new designs planned for the DOE complex and the evaluation of existing facilities requires cost benefit considerations and prioritizing upgrading and retrofit efforts. Finally, the graded approach is common practice by model building codes such as the Uniform Building Code (Ref. B-7), Department of Defense earthquake provisions (Ref. B-8), and even by the Nuclear Regulatory Commission which provides graded criteria from power plants to other licensed nuclear facilities.

The motivation for the use of probabilistic performance goals by the NPH Implemenation Guide for DOE Order 420.1 and DOE-STD-1020 is that accomplish the graded approach using a quantified approach consistent with the variety of DOE facilities as well as meeting the risk-based DOE safety policy. Furthermore, the use of probabilistic performance goals enables the development of consistent criteria both for all natural phenomena hazards (i.e., earthquakes, winds, and floods) and for all DOE facilities which are located throughout the United States. The use of performance goal based criteria is becoming common practice as: it is embedded in recent versions of the Uniform Building Code and in the DOD seismic provisions for essential buildings; it has been used for DOE new production reactor NPH criteria; and it has been utilized in recent Nuclear Regulatory Commission applications such as for the advanced light water reactor program and for revisions to commercial reactor geological siting criteria in 10CFR100, Appendix A.

Five performance categories are specified in the Implementation Guide for DOE Order 420.1 for design/evaluation of DOE structures, systems, and components for natural phenomena hazards ranging from 0 through 4. Table B-1 presents both the qualitative and quantitative descriptions of the performance goals for each performance category. Both the qualitative description of acceptable NPH performance and the quantitative probability value for each performance category are equally significant in establishing these NPH

design and evaluation criteria within a graded approach. SSCs are to be placed in categories in accordance with DOE-STD-1021-93 (Ref. B-9) Additional guidance on performance categorization is available in Reference B-10.

As mentioned previously, the quantitative performance goal probability values are applicable to each natural phenomena hazard (earthquake, wind, and flood) individually. The earthquake and flood design and evaluation criteria presented in this document are aimed at meeting the target performance goals given in Table B-1. The extreme wind and tornado design and evaluation criteria presented in this document are conservative compared to earthquake and flood criteria in that they are aimed at lower probability levels than the target performance goals in Table B-1. It is estimated that for extreme winds, the probabilities of exceeding acceptable behavior limits are less than one order of magnitude smaller than the performance goals in Table B-1. For tornado criteria, the probabilities of exceeding acceptable behavior limits are greater than one but less than two orders of magnitude smaller than the performance goals for Performance Categories 3 and 4. This additional conservatism in wind and tornado criteria for design and evaluation of DOE facilities is consistent with common practice in government and private industry. Furthermore, this additional conservatism can be accommodated in the design and evaluation of SSCs without significantly increasing costs. SSCs in Performance Categories 3 and 4 should be designed for tornadoes at certain sites around the country where tornado occurrences are high. The tornado hazard probability must be set lower than necessary to meet the performance goals in order for tornadoes rather than straight winds or hurricanes to control the design criteria.

Table B-1 Structure, System, or Component (SSC) NPH Performance Goals for Various Performance Categories

Performance Category	Performance Goal Description	NPH Performance Goal Annual Probability of Exceeding Acceptable Behavior Limits, P_f
0	No Safety, Mission, or Cost Considerations	No requirements
1	Maintain Occupant Safety	$\leq 10^{-3}$ of the onset of SSC[1,2] damage to the extent that occupants are endangered
2	Occupant Safety, Continued Operation with Minimum Interruption	$\leq 5 \times 10^{-4}$ of SSC damage to the extent that the component cannot perform its function
3	Occupant Safety, Continued Operation, Hazard Confinement	$\leq 10^{-4}$ of SSC damage to the extent that the component cannot perform its function
4	Occupant Safety, Continued Operation, Confidence of Hazard Confinement	$\leq 10^{-5}$ of SSC damage to the extent that the component cannot perform its function

(1) These performance goals are for each natural phenomena hazard (earthquake, wind, and flood).
(2) SSC refers to structure, distribution system, or component (equipment).

The design and evaluation criteria for SSCs in Performance Categories 0, 1, and 2 are similar to those given in model building codes. Performance Category 0 recognizes that for certain lightweight equipment items, furniture, etc., and for other special circumstances where there is little or no potential impact on safety, mission, or cost, design or evaluation for natural phenomena hazards may not be needed. Assignment of an SSC to Performance Category 0 is intended to be consistent with, and not take exception to, model building code NPH provisions. Performance Category 1 criteria include no extra conservatism against natural phenomena hazards beyond that in model building codes that include earthquake, wind, and flood considerations. Performance Category 2 criteria are intended to maintain the capacity to function and to keep the SSC operational in the event of natural phenomena hazards. Model building codes would treat hospitals, fire and police stations, and other emergency-handling facilities in a similar manner to DOE-STD-1020 Performance Category 2 NPH design and evaluation criteria.

Performance Category 3 and 4 SSCs handle significant amounts of hazardous materials or have significant programmatic impact. Damage to these SSCs could potentially endanger worker and public safety and the environment or interrupt a significant mission. As a result, it is very important for these SSCs to continue to function in the event of natural phenomena hazards, such that the hazardous materials may be controlled and confined. For these categories, there must be a very small likelihood of damage due to natural phenomena hazards. DOE-STD-1020 NPH criteria for Performance Category 3 and higher SSCs are more conservative than requirements found in model building codes and are similar to DOD criteria for high risk buildings and NRC criteria for various applications as illustrated in Table B-2. Table B-2 illustrates how DOE-STD-1020 criteria for the performance categories defined in DOE Order 420.1 and the associated Implementation Guides compare with NPH criteria from other sources.

Table B-2 Comparison of Performance Categories from Various Sources

Source	SSC Categorization			
DOE-STD-1020 - DOE Natural Phenomena Hazard Criteria	1	2	3	4
Uniform Building Code	General Facilities	Essential Facilities	-	
DOD Tri-Service Manual for Seismic Design of Essential Buildings	-	-	High Risk	-
Nuclear Regulatory Commission	-		Evaluation of NRC Fuel Facilities	Evaluation of Existing Reactors

The design and evaluation criteria presented in this document for SSCs subjected to natural phenomena hazards have been specified to meet the performance goals presented in Table B-1. The basis for selecting these performance goals and the associated annual probabilities of exceedance are described briefly in the remainder of this section.

For *Performance Category 1 SSCs,* the primary concern is preventing major structural damage or collapse that would endanger personnel. A performance goal annual probability of exceedance of about 10^{-3} of the onset of significant damage is appropriate for this category. This performance is considered to be consistent with model building codes (Refs. B-7, B-11, B-12, and B-13), at least for earthquake and wind considerations. The primary concern of model building codes is preventing major structural failure and maintaining life safety under major or severe earthquakes or winds. Repair or replacement of the SSC or the ability of the SSC to continue to function after the occurrence of the hazard is not considered.

Performance Category 2 SSCs are of greater importance due to mission-dependent considerations. In addition, these SSCs may pose a greater danger to on-site personnel than Performance Category 1 SSCs because of operations or materials involved. The performance goal is to maintain both capacity to function and occupant safety. Performance Category 2 SSCs should allow relatively minor structural damage in the event of natural phenomena hazards. This is damage that results in minimal interruption to 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 or equipment damage, with the SSC being able to function with minimal interruption. This performance goal is slightly more severe than that corresponding to the design criteria for essential facilities (e.g., hospitals, fire and police stations, centers for emergency operations) in accordance with model building codes (e.g., Ref. B-7).

Performance Category 3 and higher SSCs 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 SSC damage so that hazardous materials can be controlled and confined, occupants are protected, and functioning of the SSC is not interrupted. The performance goal for Performance Category 3 and higher SSCs is to limit damage such that DOE safety policy is achieved. For these categories, damage must typically be limited in confinement barriers (e.g., buildings, glove boxes, storage canisters, vaults), ventilation systems and filtering, and monitoring and control equipment in the event of an occurrence of severe earthquakes, winds, or floods. In addition, SSCs can be placed in Performance Categories 3 or 4 if improved performance is needed due to cost or mission requirements.

For Performance Category 3 SSCs, an appropriate performance goal has been set at an annual probability of exceedance of about 10^{-4} of damage beyond which hazardous material confinement and safety-related functions are impaired. For Performance Category 4 SSCs, a reasonable performance goal is an annual probability of exceedance of about 10^{-5} of damage beyond which hazardous material confinement and safety-related functions are impaired. These performance goals approaches and approximates, respectively, at least for earthquake considerations, the performance goal for seismic-induced core damage associated with design of commercial nuclear power plants (Refs. B-14, B-15, B-16, and B-17). Annual frequencies of seismic core damage from published probabilistic risk assessments (PRA) of recent commercial nuclear plants have been summarized in Reference B-18. 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 Performance Category 4 performance goals given in the NPH Implementation Guide for DOE Order 420.1 are consistent with Reference B-18 information.

B.3 Evaluation of Existing Facilities

New SSCs can be designed by these criteria, but existing SSCs may not meet these NPH provisions. 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 criteria presented in this document.

For existing SSCs, an assessment must be made for the 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 SSCs 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. Prioritization criteria for evaluation and upgrade of existing DOE facilities are currently being developed.

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

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

If SSC 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 SSC upgrades than to expend effort on further analytical studies. Note that the actions in Table B-2 need not necessarily be accomplished in the order shown.

Evaluations of existing SSCs must follow or, at least, be measured against the NPH criteria provided in this document. For SSCs not meeting these criteria and which cannot be easily remedied, budgets and schedule for required strengthening must be established on a prioritized basis. As mentioned previously, prioritization criteria for evaluation and upgrade of existing DOE facilities are currently being developed. Priorities should be established on the basis of performance category, cost of strengthening, and margin between as-is SSC capacity and the capacity required by the criteria. For SSCs which are close to meeting criteria, it is probably not cost effective to strengthen the SSC in order to obtain a small reduction in risk. As a result, some relief in the criteria is allowed for evaluation of existing SSCs. It is permissible to perform such evaluations using natural phenomena hazard exceedance probability of twice the value specified for new design. For example, if the natural phenomena hazard annual probability of exceedance for the SSC under consideration was 10^{-4}, it would be acceptable to reconsider the SSC at hazard annual probability of exceedance of 2×10^{-4}. This would have the effect of slightly reducing the seismic, wind, and flood loads in the SSC evaluation. This amount of relief is within the tolerance of meeting the target performance goals and is only a minor adjustment of the corresponding NPH design and evaluation criteria.

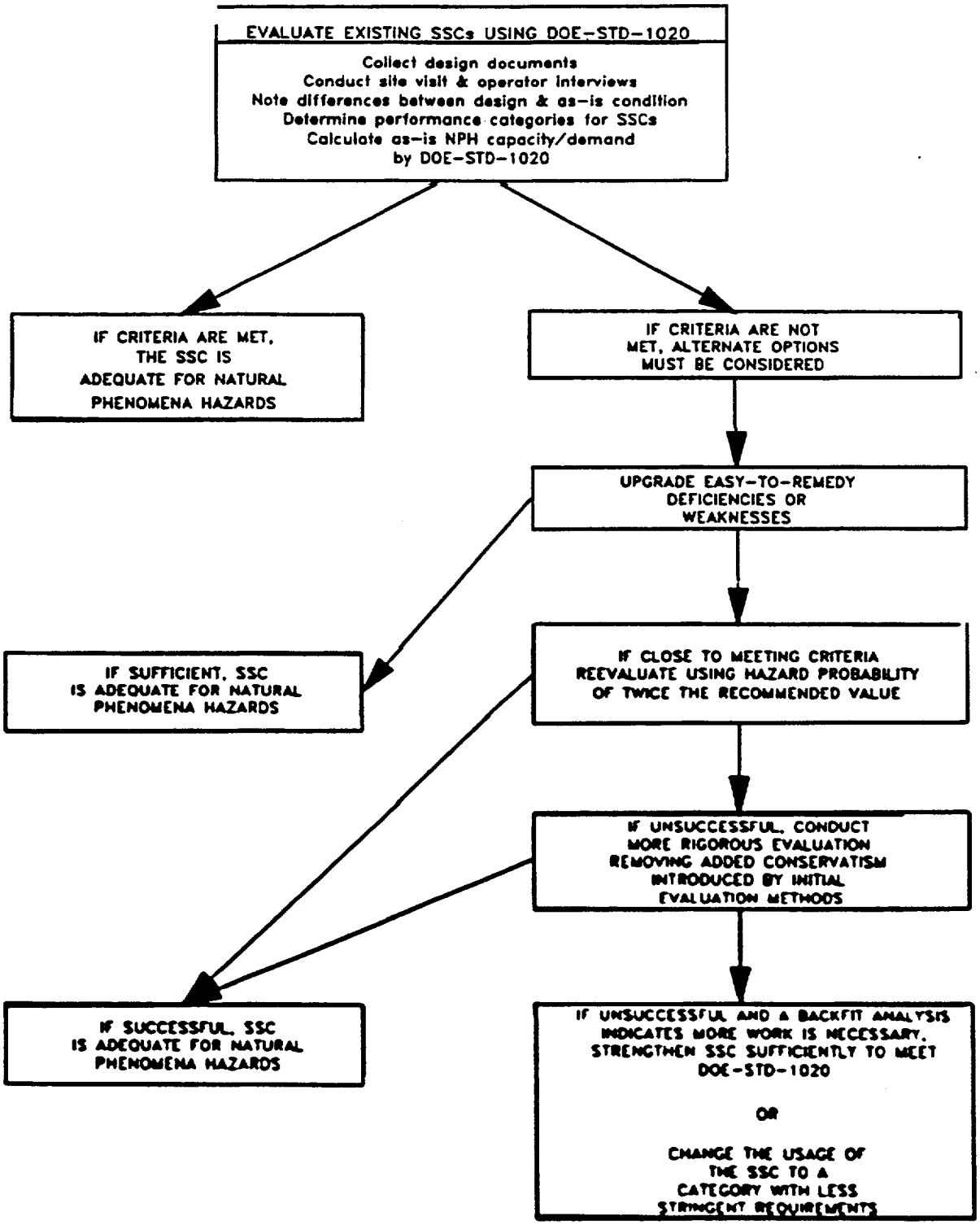

Figure B-2 Evaluation Approach for an Existing SSC

B.4 References

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Appendix C
Commentary on Earthquake Design and Evaluation Criteria

C.1 Introduction

Earthquake design and evaluation criteria for DOE structures, systems, and components are presented in Chapter 2 of this standard. Commentary on the DOE earthquake design and evaluation provisions is given in this appendix. Specifically, the basic approach employed is discussed in Section C.2 along with meeting of target performance goals, seismic loading is addressed in Section C.3, evaluation of seismic response is discussed in Section C.4, capacities and good seismic design practice are discussed in Section C.5, special considerations for systems and components and for existing facilities are covered in Sections C. 6 and C. 7, respectively, and quality assurance and peer review are addressed in Section C.8. Alternate seismic mitigation measures are discussed in Section C.9.

These seismic criteria use the target performance goals of the NPH Implementation Guide for DOE Order 420.1 (Ref. C-67) to assure safe and reliable performance of DOE facilities during future potential earthquakes. Design of structures, systems, and components to withstand earthquake ground motion without significant damage or loss of function depends on the following considerations:

1. The SSC must have sufficient strength and stiffness to resist the lateral loads induced by earthquake ground shaking. If an SSC is designed for insufficient lateral forces or if deflections are unacceptably large, damage can result, even to well-detailed SSCs.
2. Failures in low ductility modes (e.g., shear behavior) or due to instability that tend to be abrupt and potentially catastrophic must be avoided. SSCs must be detailed in a manner to achieve ductile behavior such that they have greater energy absorption capacity than the energy content of earthquakes.
3. Building structures and equipment which are base supported tend to be more susceptible to earthquake damage (because of inverted pendulum behavior) than distributed systems which are supported by hangers with ductile connections (because of pendulum restoring forces).
4. The behavior of an SSC 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. Also, the designer must consider both relative displacement and inertia (acceleration) induced seismic failure modes.
5. SSCs 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.

By the NPH Implementation Guide for DOE Order 420.1 (Ref. C-67) and this standard, probabilistic performance goals are used as a target for formulating deterministic seismic design criteria. Table C-1 defines seismic performance goals for structures, systems, or components (SSCs) assigned to Performance Categories 1 through 4. SSCs are to be assigned to performance categories in accordance with DOE-STD-1021-93 (Ref. C-26). The seismic performance goals are defined in terms of a permissible annual probability of unacceptable performance P_F (i.e., a permissible failure frequency limit). Seismic induced unacceptable performance should have an annual probability less than or approximately equal to these goals.

Table C-1 Structure, System, or Component (SSC) Seismic Performance Goals for Various Performance Categories

Performance Category	Performance Goal Description	Seismic Performance Goal Annual Probability of Exceeding Acceptable Behavior Limits, P_F
1	Maintain Occupant Safety	$\simeq 10^{-3}$ of the onset of SSC(1) damage to the extent that occupants are endangered
2	Occupant Safety, Continued Operation with Minimum Interruption	$\simeq 5 \times 10^{-4}$ of SSC damage to the extent that the component cannot perform its function
3	Occupant Safety, Continued Operation, Hazard Confinement	$\simeq 10^{-4}$ of SSC damage to the extent that the component cannot perform its function
4	Occupant Safety, Continued Operation, Confidence of Hazard Confinement	$\simeq 10^{-5}$ of SSC damage to the extent that the component cannot perform its function

(1) SSC refers to structure, distribution system, or component (equipment).

The performance goals shown in Table C-1 include both quantitative probability values and qualitative descriptions of acceptable performance. The qualitative descriptions of expected performance following design/evaluation levels of earthquake ground motions are expanded in Table C-2. These descriptions of acceptable performance are specifically tailored to the needs in many DOE facilities.

The performance goals described above are achieved through the use of DOE seismic design and evaluation provisions which include: (1) lateral force provisions; (2) story drift/damage control provisions; (3) detailing for ductility provisions; and (4) quality assurance provisions. These provisions are comprised of the following four elements taken together: (1) seismic loading; (2) response evaluation methods; (3) permissible response levels; and (4) ductile detailing requirements. Acceptable performance (i.e., achieving performance goals) can only be reached by consistent specification of all design criteria elements as shown in Figure C-1.

Table C-2 Qualitative Seismic Performance Goals

PC	Occupancy Safety	Concrete Barrier	Metal Liner	Component Functionality	Visible Damage
1	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation	Confinement not required.	Confinement not required.	Component will remain anchored, but no assurance it will remain functional or easily repairable.	Building distortion will be limited but visible to the naked eye.
2	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation	Concrete walls will remain standing but may be extensively cracked; they may not maintain pressure differential with normal HVAC. Cracks will still provide a tortuous path for material release. Don't expect largest cracks greater than 1/2 inch.	May not remain leak tight because of excessive distortion of structure.	Component will remain anchored and majority will remain functional after earthquake. Any damaged equipment will be easily repaired.	Building distortion will be limited but visible to the naked eye.
3	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation	Concrete walls cracked; but small enough to maintain pressure differential with normal HVAC. Don't expect largest cracks greater than 1/8 inch.	Metal liner will remain leak tight.	Component anchored and functional.	Possibly visible local damage but permanent distortion will not be immediately apparent to the naked eye.
4	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation	Concrete walls cracked; but small enough to maintain pressure differential with normal HVAC. Don't expect largest cracks greater than 1/8 inch.	Metal liner will remain leak tight.	Component anchored and functional.	Possibly visible local damage but permanent distortion will not be immediately apparent to the naked eye.

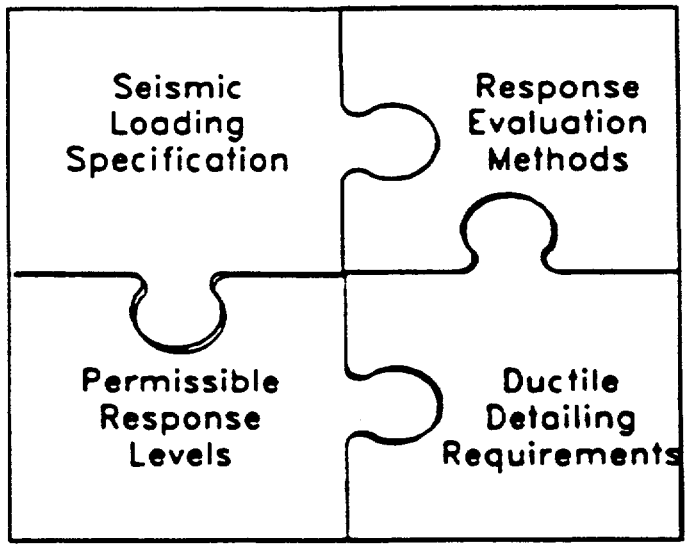


Figure C-1 Consistent Specification of All Seismic Design/Evaluation Criteria Elements

C.2 Basic Approach for Earthquake Design and Evaluation and Meeting Target Performance Goals

C.2.1 Overall Approach for DOE Seismic Criteria

Structure/component performance is a function of: (1) the likelihood of hazard occurrence and (2) the strength of the structure or equipment item. Consequently, seismic performance depends not only on the earthquake probability used to specify design seismic loading, but also on the degree of conservatism used in the design process as illustrated in Figure C-2. 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 Performance Category 1) and for nuclear power plants (similar to DOE Performance Category 4). For consistency with these other uses, the approach in this standard specifies the use of conservative design and evaluation procedures coupled with a hazard definition consistent with these procedures.

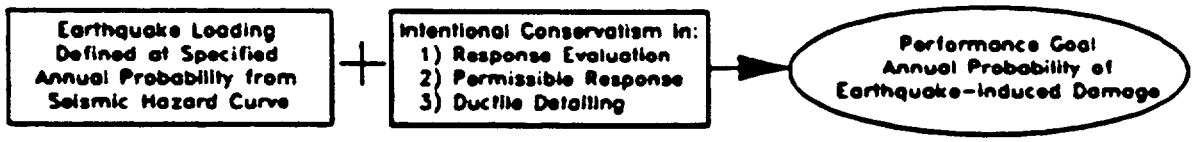


Figure C-2 Performance Goal Achievement

The performance goals for Performance Category 1 SSCs are consistent with goals of model building codes for normal facilities; the performance goals for Performance Category 2 SSCs are slightly more conservative than the goals of model building codes for important or essential facilities. For seismic design and evaluation, model 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 Performance Category 3 SSC's are consistent with DOE essential facilities and Pu handling facilities. The perform-

ante goals for Performance Category 4 SSC's approach those used for nuclear power plants. For these reasons, this standard specifies seismic design and evaluation criteria for PC-1 and PC-2 SSC's corresponding closely to model building codes and seismic design and evaluation criteria for both PC-3 and PC-4 SSC's based on dynamic analysis methods consistent with those used for similar nuclear facilities.

By this standard, the DBE is defined at specified hazard probability P_H and the SSC is designed or evaluated for this DBE using an adequately conservative deterministic acceptance criteria. To be adequately conservative, the acceptance criteria must introduce an additional reduction in the risk of unacceptable performance below the annual risk of exceeding the DBE. The ratio of the seismic hazard exceedance probability, P_H to the performance goal probability P_F is defined herein as the risk reduction ratio R_R, given by:

$$R_R = \frac{P_H}{P_F} \qquad \text{(C-1)}$$

The required degree of conservatism in the deterministic acceptance criteria is a function of the specified risk reduction ratio. Table C-3 provides a set of seismic hazard exceedance probabilities, P_H and risk reduction ratios, R_R for Performance Categories 1 through 4 required to achieve the seismic performance goals specified in Table C-1. Note that Table C-3 follows the philosophy of:

1) gradual reduction in hazard annual exceedance probability
2) gradual increase in conservatism of evaluation procedure as one goes from Performance Category 1 to Performance Category 4 (PC 1 to PC 4).

Table C-3 Seismic Performance Goals & Specified Seismic Hazard Probabilities

Performance Category	Target Seismic Performance Goal, P_F	Seismic Hazard Exceedance Probability, P_H	Risk Reduction Ratio, R_R
1	1×10^{-3}	2×10^{-3}	2
2	5×10^{-4}	1×10^{-3}	2
3	1×10^{-4}	5×10^{-4} (1×10^{-3})[1]	5 (10)[1]
4	1×10^{-5}	1×10^{-4} (2×10^{-4})[1]	10 (20)[1]

1 For sites such as LLNL, SNL-Livermore, SLAC, LBL, and ETEC which are near tectonic plate boundaries.

Different structures, systems, or components may have different specified performance goal probabilities, P_F. It is required that for each structure, system, or component, either: (1) the performance goal category; or (2) the hazard probability (P_H) or the DBE together with the appropriate R_R factor will be specified in a design specification or imple-

mentation document that invokes these criteria. As shown in Table 2-3, 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 to achieve the required risk reduction ratio, R_R. In earthquake evaluation, there are many places where conservatism can be introduced, including:

1. Maximum design/evaluation ground acceleration and velocity.
2. Response spectra amplification.
3. Damping.
4. Analysis methods.
5. Specification of material strengths.
6. Estimation of structural capacity.
7. Load or scale factors.
8. Importance factors.
9. Limits on inelastic behavior.
10. Soil-structure interaction (except for frequency shifting due to SSI).
11. Effective peak ground motion.
12. Effects of a large foundation or foundation embedment.

For the earthquake evaluation criteria in this standard, conservatism is intentionally introduced and controlled by specifying (1) hazard exceedance probabilities, (2) load or scale 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 Performance Category 2 and lower SSCs because the UBC approach (which includes these factors) is followed for these categories. Importance factors are not used for Performance Category 3 and higher SSCs. However, a seismic scale factor SF is used to provide the difference in risk reduction ratio R_R between Performance Categories 3 and 4. Material strengths and structural capacities specified for Performance Category 3 and higher SSCs correspond to ultimate strength code-type provisions (i.e., ACI 318-89 for reinforced concrete, LRFD, or AISC Chapter N for steel). Material strengths and structural capacities specified for Performance Category 2 and lower SSCs correspond to either ultimate strength or allowable stress code-type provisions. It is recognized that such provisions introduce conservatism. In addition, 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 differences in seismic evaluation criteria among categories in terms of load and importance factors, limits on inelastic behavior, and other factors by this standard are summarized below:

1.	PC 1 and PC 2	From PC 1 to PC 2, seismic hazard exceedance probability is lowered and importance factor is increased. All other factors are held the same.
2.	PC 2 and PC 3	From PC 2 to PC 3, load and importance factors are eliminated, damping is generally increased, and limits on inelastic behavior are significantly reduced. All other factors are essentially the same, although static force evaluation methods are allowed for PC 2 SSCs and dynamic analysis is required for PC 3 SSCs.
3.	PC 3 and PC 4	From PC 3 to PC 4, seismic hazard exceedance probability is lowered and a seismic scale factor is used. All other factors are held the same.

The basic intention of the deterministic seismic evaluation and acceptance criteria presented in Chapter 2 is to achieve less than a 10% probability of unacceptable performance for a structure, system, or component (SSC) subjected to a Scaled Design/Evaluation Basis Earthquake (SDBE) defined by:

$$SDBE = (1.5SF)(DBE) \tag{C-2}$$

where SF is the appropriate seismic scale factor (SF is 1.0 for PC 3 and 1.25 for PC 4). The seismic evaluation and acceptance criteria presented in this standard has intentional and controlled conservatism such that the required risk reduction ratios, R_n, and target performance goals are achieved. The amount of intentional conservatism has been evaluated in Reference C-20 as that there should be less than 10% probability of unacceptable performance at input ground motion defined by a scale factor of 1.5SF times the DBE. Equation C-2 is useful for developing alternative evaluation and acceptance criteria which are also based on the target performance goals.

It is permissible to substitute alternate acceptance criteria for those criteria defined in Chapter 2 so long as these alternate criteria will also reasonably achieve less than about a 10% probability of unacceptable performance for the combination of the SDBE defined by Equation C-2 with the best-estimate of the concurrent non-seismic loads. This relief is permitted to enable one to define more sophisticated alternate acceptance criteria than those presented in Chapter 2 when one has a sufficient basis to develop and defend this alternate criteria.

C.2.2 Influence of Seismic Scale Factor

The target performance goals of the Implementation Guide for DOE Order 420.1 are the basis of the seismic design and evaluation criteria presented in this standard. It is known that for PC 1 and PC 2, target performance goals, P_n, of 1×10^{-4} and 5×10^{-4}, respec-

tively, are met relatively closely. However, for PC 3 and PC 4, target performance goals, P_F, of 1 x10^{-4} and 1 x 10^{-5}, respectively, are met in a more approximate manner as illustrated in this section. The variability in performance goal achievement can be most significantly attributed to the uncertainty in the slopes of seismic hazard curves from which DBE ground motion is determined. Seismic hazard curve slope does not have a significant effect on performance for PC 1 and PC 2 because P_F and P_H do not differ greatly (i.e. $R_R = P_H/P_F =$ 2).

Over any ten-fold difference in exceedance probabilities, seismic hazard curves may be approximated by:

$$H(a) = Ka^{-k_H} \tag{C-3}$$

where H(a) is the annual probability of exceedance of ground motion level "a," K is a constant, and k_H is a slope parameter. Slope coefficient, A_R is the ratio of the increase in ground motion corresponding to a ten-fold reduction in exceedance probability. AR is related to k_H by:

$$k_H = \frac{1}{\log(A_R)} \tag{C-4}$$

The Basis for Seismic Provisions of DOE-STD-1020 (Ref. C-20) presents estimates of seismic hazard curve slope ratios A_R for typical U.S. sites over the annual probability range of 10^{-4} to 10^{-6}. For eastern U.S. sites, A_R typically falls within the range of 2 to 4 although A_R values as large as 6 have been estimated. For California and other high seismic sites near tectonic plate boundaries with seismicity dominated by close active faults with high recurrence rates, A_R typically ranges from 1.5 to 2.25. For other western sites with seismicity not dominated by close active faults with high recurrence rates such as INEL, LANL, and Hanford, A_R typically ranges from 1.75 to 3.0. Therefore, seismic design/evaluation criteria should be applicable over the range of A_R from 1.5 to 6 with emphasis on the range from 2 to 4.

DOE seismic design and evaluation criteria presented in Chapter 2 is independent of A_R and, thus, does not reflect its effect on meeting target goals. The performance of structures, systems, and components in terms of annual probability of exceeding acceptable behavior limits can be evaluated by convolution of seismic hazard and seismic fragility curves. Seismic fragility curves describe the probability of unacceptable performance versus ground motion level. The fragility curve is defined as being lognormally distributed

and is expressed in terms of two parameters: a median capacity level, C_{50}, and a logarithmic standard deviation, β. β expresses the uncertainty in the capacity level and generally lies within the range of 0.3 to 0.6. For DBE ground motion specified at annual probability, P_H, it is shown in Ref. C-20 that the risk reduction ratio, R_R, between the annual probability of exceeding the DBE and the annual probability of unacceptable performance is given by:

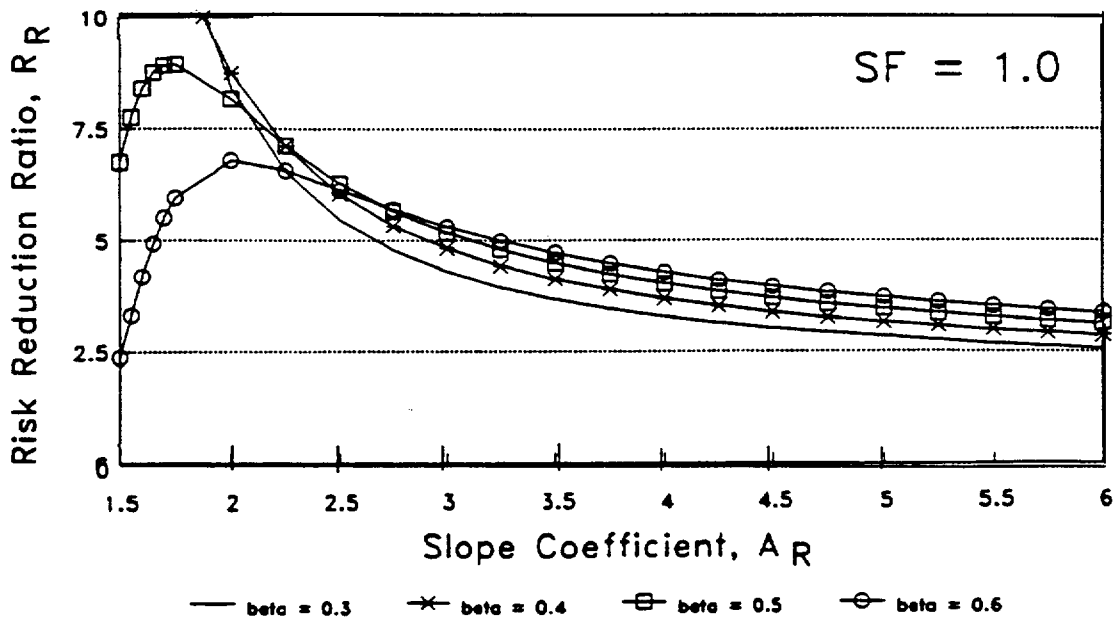
$$R_R = (C_{50}/\mathrm{DBE})^{k_H} e^{-\frac{1}{2}(k_H\beta)^2} \tag{C-5}$$

where C_{50} and β define the seismic fragility curve and DBE and k_H define the seismic hazard curve.

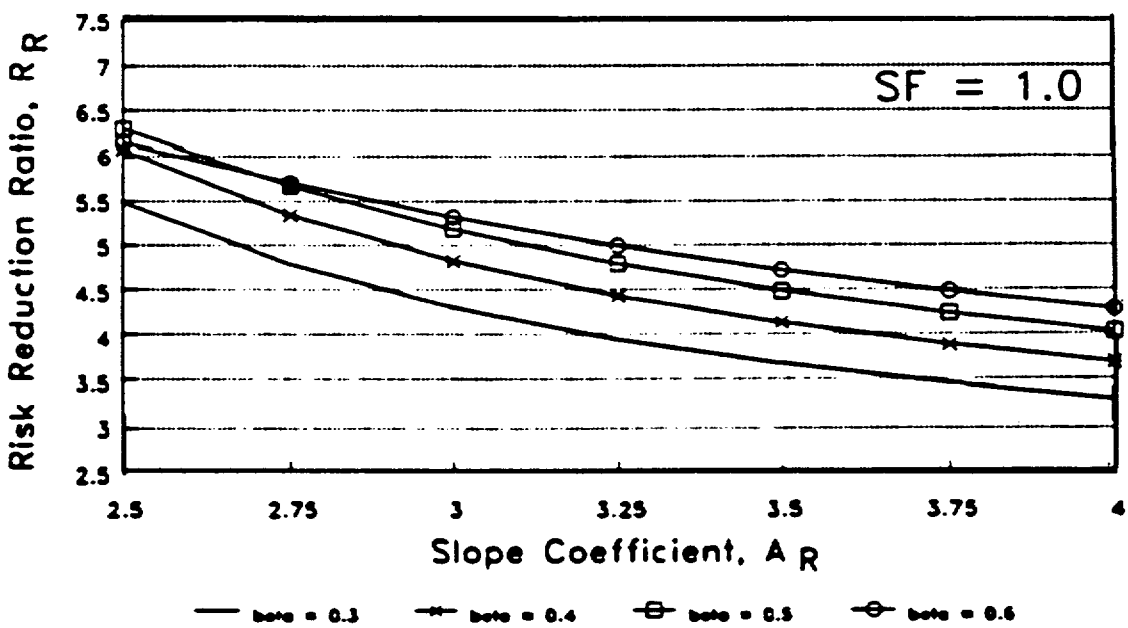
Using the basic criterion of DOE-STD-1020 that target performance goals are achieved when the minimum required 10% probability of failure capacity, C_{10} is equal to 1.5 times the seismic scale factor, SF, times the DBE ground motion, Equation (C-5) may be rewritten as:

$$R_R = (1.5\mathrm{SF})^{k_H} e^{\left[1.282 k_H\beta - \frac{1}{2}(k_H\beta)^2\right]} \tag{C-6}$$

Equation (C-6) demonstrates the risk reduction ratio achieved by DOE seismic criteria as a function of hazard curve slope, uncertainty, β, and seismic scale factor, SF. Note from Table C-3 that for Performance Category 4 (not near tectonic plate boundaries), the hazard probability is 1×10^{-4} and the performance goal is 1×10^{-4} such that the target risk reduction ratio, R_R is 10 and for Performance Category 3, the hazard probability is 5×10^{-4} and the performance goal is 1×10^{-4} such that the target risk reduction ratio, R_R is 5. The actual risk reduction ratios from Equation (C-6) versus slope coefficient A_H are plotted in Figures C-3 and C-4 for Performance categories 3 and 4, respectively. In these figures, SF of 1.0 is used for PC 3 and SF of 1.25 is used for PC 4 and the range of β from 0.3 to 0.6 has been considered. For the hazard curves considered by DOE-STD-1024-92 (Ref. C-13), A_H values average about 3.2 in the probability range associated with PC 3 and about 2.4 in the probability range associated with PC 4. More recent seismic hazard studies (Ref. C-6) gives A_H values which average about 3.8 in the probability range associated with PC 3 and about 3.0 in the probability range associated with PC 4. As a result, Figure C-3 includes a blown-up view for the 2.5 to 4 AR range and Figure C-4 includes a blown-up view for the 2 to 3 A_H range.

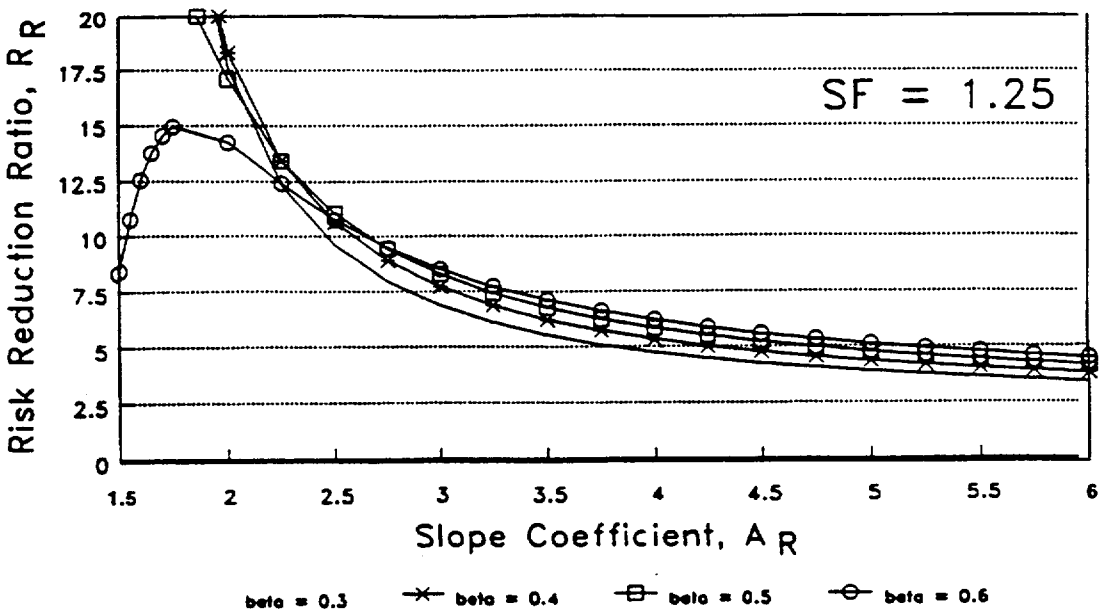


a) A_R from 1.5 to 6

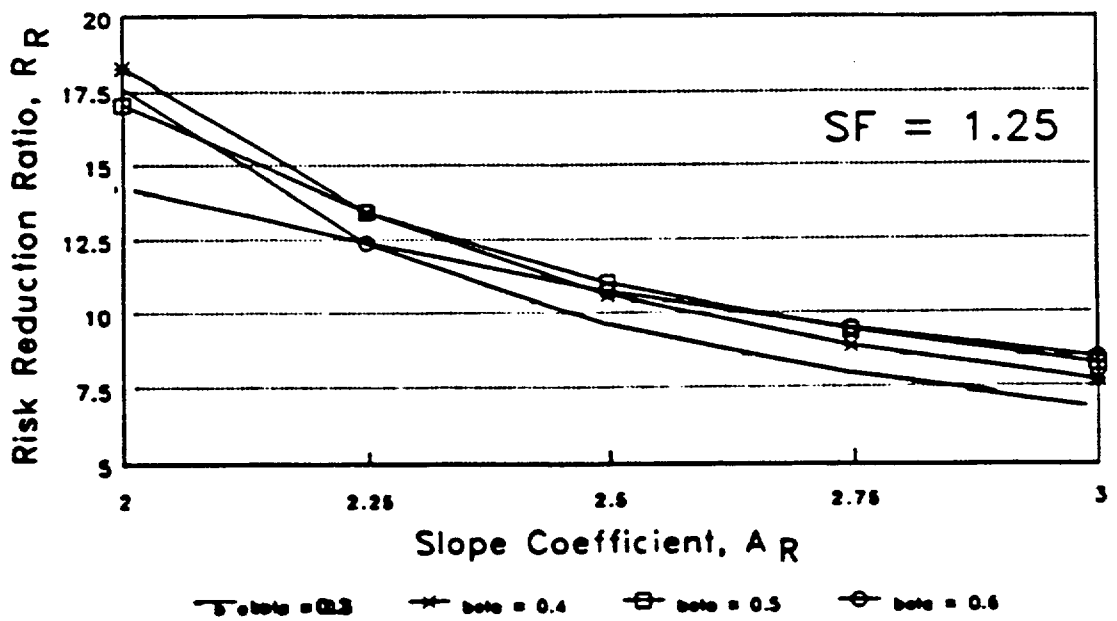


b) A_R from 2.5 to 4

Figure C-3 Value of R_R vs A_R for SF = 1.0 (PC 3)



a) A_R from 1.5 to 6



b) A_R from 2 to 3

Figure C-4 Value of R_R vs A_R for SF = 1.25 (PC 4)

Figure C-3 demonstrates that for SF = 1.0, risk reduction ratios between about 3 and 10 are achieved over the A_R range from 2 to 6. These risk reduction ratios support achieving performance goals between about $2x10^{-4}$ to $5x10^{-5}$. In the primary region of interest of A_R between 2.5 and 4, risk reduction ratios from 4 to 6 are achieved as compared to the target level of 5 for PC 3 and sites not near tectonic plate boundaries. Figure C-4 demonstrates that for SF = 1.25, risk reduction ratios between about 3 and 20 are achieved over the A_R range from 2 to 6. These risk reduction ratios support achieving performance goals between about $3x10^{-5}$ to $5x10^{-4}$. In the primary region of interest of A_R between 2 and 3, risk reduction ratios from about 8 to 17 are achieved as compared to the target level of 10 for PC 4 and sites not near tectonic plate boundaries.

The risk reduction ratio achieved may be improved by using a variable formulation of SF which is a function of A_R. In order to justify use of the variable scale factor approach, the site specific hazard curve must have a rigorous pedigree. Reference C-20 demonstrates that the SF factors shown in Figure C-5 give the best fit of R_R over the A_R range of primary interest from about 2 to about 6. The use of the scale factors given in Figure C-5 combined with Equation C-6 improves the R_R values compared to target values as shown in Figures C-6 and C-7 for PC 3 (R_R = 5) and PC 4 (R_R = 10), respectively. Figures C-6 and C-7 demonstrate that when the variable scale factors from Figure C-5 are used, risk reduction factors achieved are within about 10% of the target values of 5 and 10, respectively. As a result, target performance goals would be met within about the same 10%.

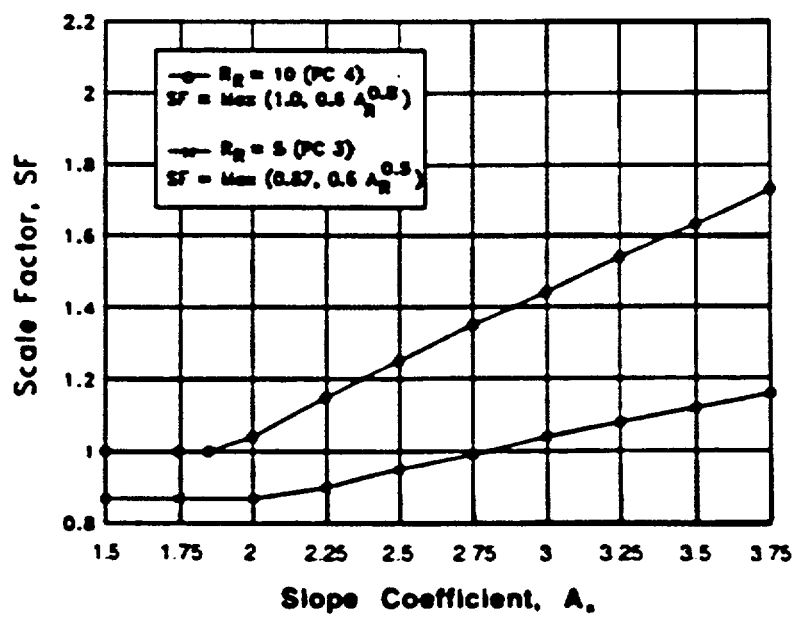


Figure C-5 Variable II Seismic Scale Factor for PC 3 and PC 4

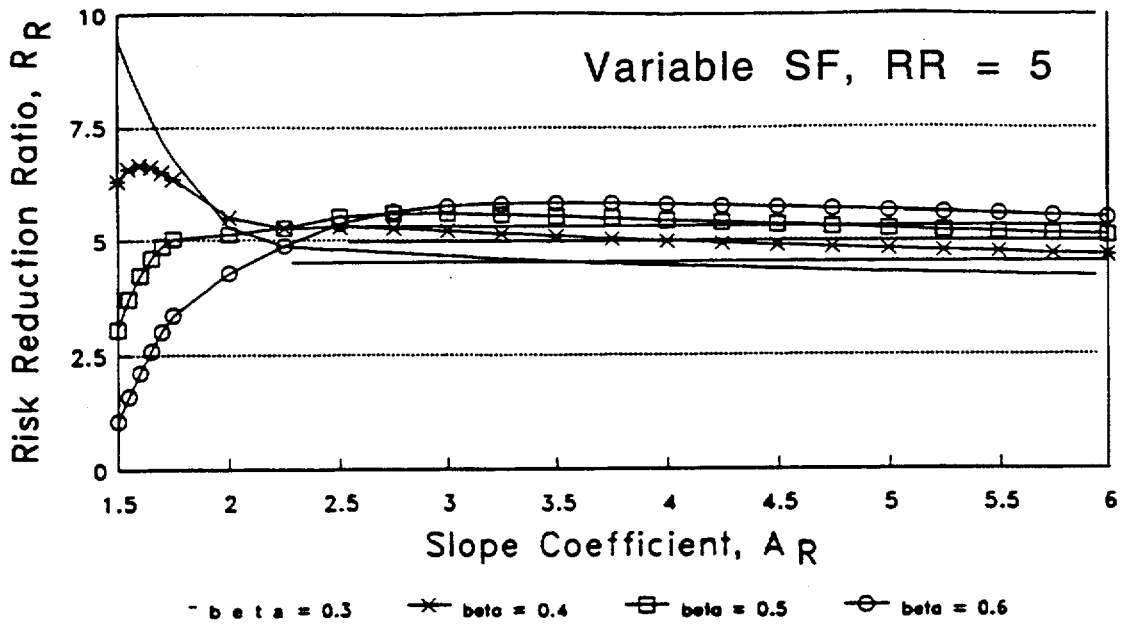


Figure C-6 Value of R_R vs A_R for Variable SF (Fig. C-5 for PC 3)

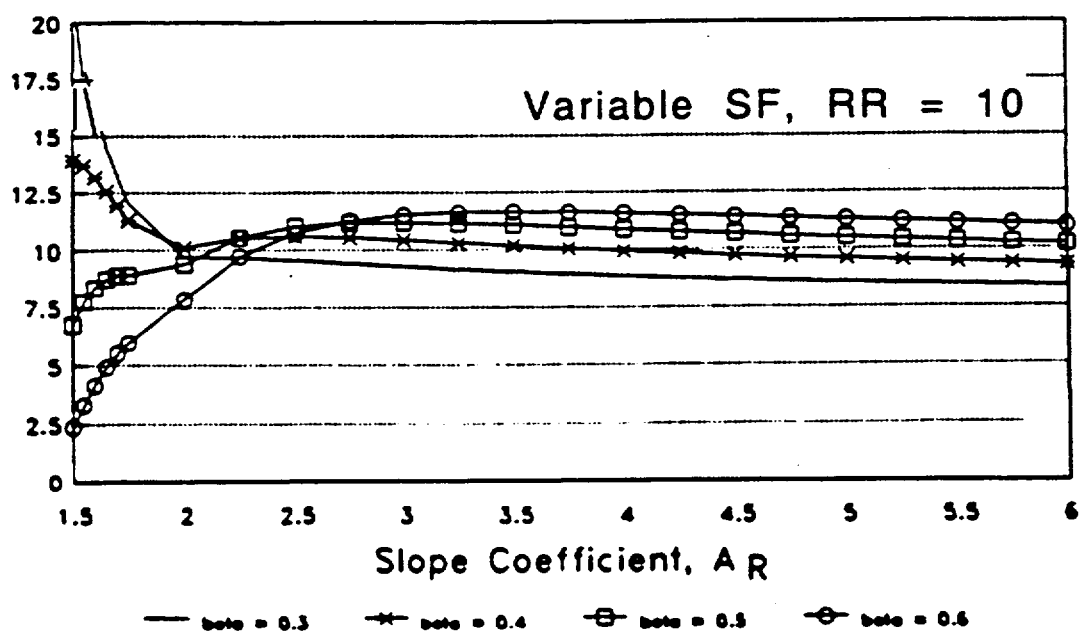


Figure C-7 Value of R_R vs AR for Variable SF (Fig. C-5 for PC 4)

For sites near tectonic plate boundaries for which A_R is in the range of about 1.5 to 2.25, such as LLNL, SNL-Livermore, SLAC, LBL, and ETEC. Figures C-3a and C-4a demonstrate that larger risk reduction ratios are achieved than the target levels of 5 for PC 3 and 10 for PC 4, respectively. Therefore, it is acceptable to use twice the hazard probabilities for these sites combined with the appropriate constant scale factors. Hence, for sites near tectonic plate boundaries, target performance goals may be adequately achieved with hazard probabilities and seismic scale factors of $1x10^{-3}$ and 1.0 for PC 3 and $2x10^{-4}$ and 1.25 for PC 4.

C.3 Seismic Design/Evaluation Input

The seismic performance goals presented in Tables C-1 and C-2 are achieved by defining the seismic hazard in terms of a site-specified design response spectrum (called herein, the Design/Evaluation Basis Earthquake, [DBE]). Either a site-specific design response spectrum specifically developed for the site, or a generic design response spectrum that is appropriate or conservative for the site may be used as the site-specified design response spectrum. Probabilistic seismic hazard estimates are used to establish the DBE. These hazard curves define the amplitude of the ground motion as a function of the annual probability of exceedance P_H of the specified seismic hazard.

For each performance category, an annual exceedance probability for the DBE, P_H is specified from which the maximum ground acceleration (or velocity) may be determined from probabilistic seismic hazard curves. Evaluating maximum ground acceleration from a specified annual probability of exceedance is illustrated in Figure C-8. Earthquake input excitation to be used for design and evaluation by these provisions is defined by a median amplification smoothed and broadened design/evaluation response spectrum shape such as that shown in Figure C-8 anchored to this maximum ground acceleration. Note that the three spectra presented in Figure C-8 are identical; the top spectrum has spectral acceleration plotted against natural frequency on a log scale, the middle spectrum is on what is termed a tripartite plot where spectral velocities and displacements as well as accelerations are shown, and the bottom spectrum has spectral acceleration plotted against natural period on a linear scale.

It should be understood that the spectra shown in Figure C-8 represent inertial effects. They do not include relative or differential support motions of structures, equipment, or distribution systems supported at two or more points typically referred to as seismic anchor motion (SAM). While SAM is not usually applicable to building design, it might have a significant effect on seismic adequacy of equipment or distribution systems.

Seismic design/evaluation criteria based on target probabilistic performance goals requires that Design/Evaluation Basis Earthquake (DBE) motions be based on probabilistic seismic hazard assessments. in accordance with DOE Order 420.1 and the associated NPH Implementation Guide (Refs. C-27 and C-67), it is not required that a site-specific probabilistic seismic hazard assessment be conducted if the site includes only Performance Category 2 and lower SSCs. If such an assessment has not been performed, it is acceptable to determine seismic loads (as summarized in Section C.3. 2. 2) from the larger of those determined in accordance with the UBC (Ref. C-2) and with UCRL-53582, Rev. 1 (Ref. C-14). Design/evaluation earthquake ground motion determined from a recent site-specific probabilistic seismic hazard assessment is considered to be preferable to the UBC for determining ZC. Therefore, the DBE response spectrum for Performance Category 2 and lower may be developed from a new probabilistic seismic hazard assessment following the guidance given herein for Performance Category 3 and higher. However, when design/evaluation earthquake ground motion is based on recent site-specific geotechnical studies and the resulting seismic loads are less than that determined by the UBC, the differences must be justified and approval of seismic loads must be obtained from DOE.

For design or evaluation of SSCs in Performance Category 3 and higher, it is strongly recommended that a modern site-specific seismic hazard assessment be performed to provide the basis for DBE ground motion levels and response spectra. DOE Order 420.1 and the associated NPH Implementation Guide (Refs. C-27 and C-67), require that the need for updating the site seismic hazard assessment be reviewed at least every 10 years. The DOE seismic working group interim standard, DOE-STD-1024-92 (Ref. C-13), indicates that the approach used for the seismic hazard assessments summarized in UCRL-53582 (Ref. C-14), which are more than 10 years old, are out of date relative to the current state of the art. However, in accordance with DOE-STD-1024-92, it is permissible to establish DBE ground motion levels and response spectra for Performance Categories 3 and 4 based on UCRL-53582 in the interim until a modern site-specific seismic hazard assessment becomes available. DBE ground motion levels for Performance Categories 3 and 4 based on UCRL-53582 are also provided in Section C.3.2.2.

Minimum values of the DBE are provided in Section 2.3 to assure a minimum level of seismic design at all DOE sites. Such a minimum level of seismic design is believed to be necessary due to the considerable uncertainty about future earthquake potential in the lower seismicity regions of the United States where most DOE sites are located.

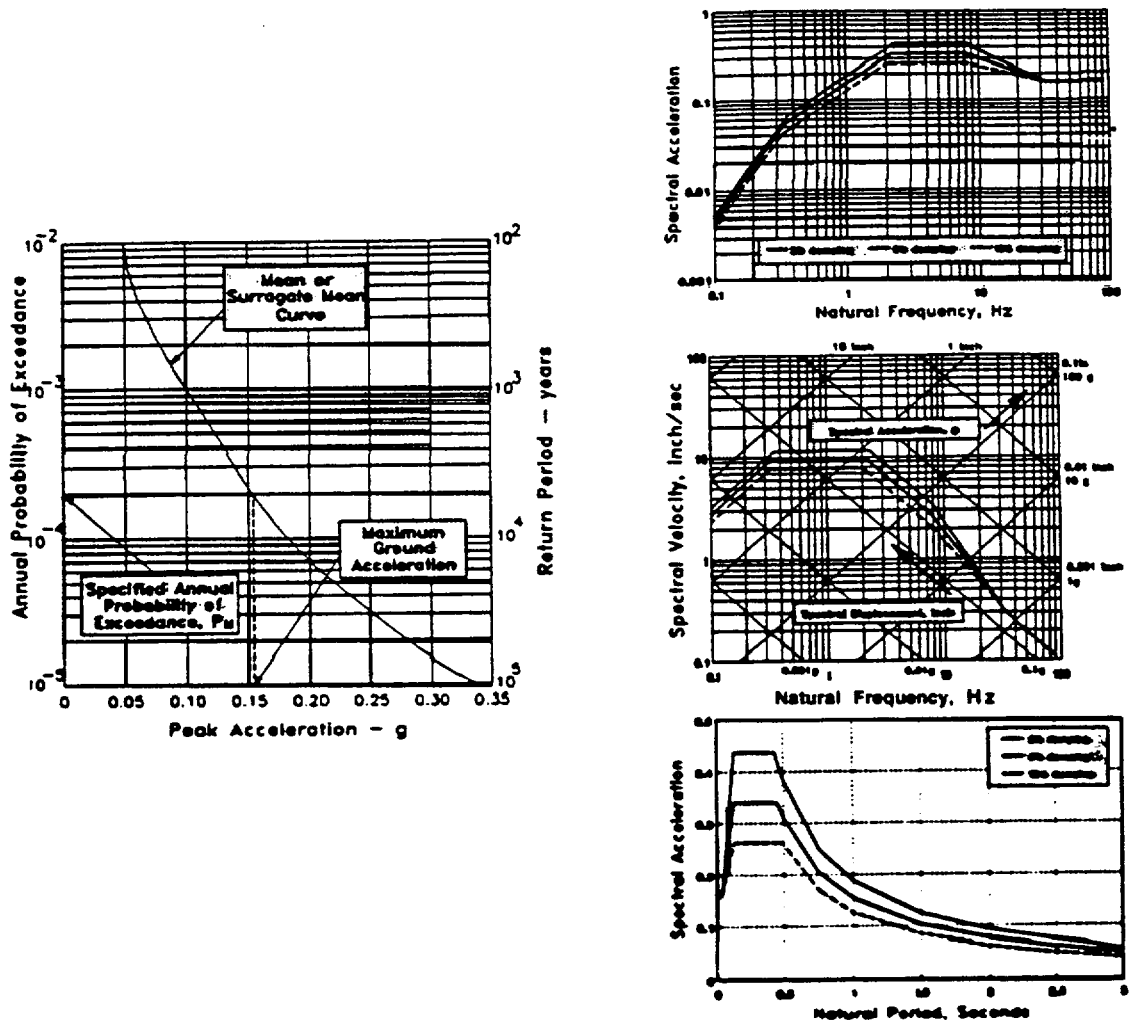


Figure C-8 Earthquake Input Excitation is Defined by Maximum Ground Acceleration Anchoring Site-Specific Response Spectra

C.3.1 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 used the UBC seismic hazard definition, other building code requirements, or 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 have been developments in building codes in which the seismic hazard provisions are based upon a consistent annual probability of exceedance for all regions of the U.S. In 1978, ATC-3 provided probabilistic-based seismic hazard provisions (Ref. C-1). From the ATC-3 provisions, changes to the UBC (Ref. C-2) and the development of the National Earthquake Hazards Reduction Program (NEHRP, Ref. C-3) have resulted. A probabilistic-based seismic zone map was incorporated into the UBC beginning with the 1988 edition. Canada and the U.S. Department of Defense have adopted this approach (Refs. C-4 and C-5). 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^{-4}. For instance, UBC (Ref. C-2), ATC-3 (Ref. C-1), and NEHRP (Ref. C-3) 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 1 $x10^{-2}$ 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. C-5) 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 $1x10^{-4}$ 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. C-6) 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 $0.1x10^{-4}$ and $10x10^{-4}$ as is illustrated in Figure C-9. The probability level of SSE design spectra (between 5 and 10 hz) at the 69 eastern U.S. nuclear power plants considered by Ref. C-6 fall within the above stated range. Figure C-9 also demonstrates that for 2/3 of these plants the SSE spectra corresponds to probabilities between about $0.4x10^{-4}$ and $2.5x10^{-4}$. Hence, the specified hazard probability level of $1x10^{-4}$ in this standard is consistent with SSE levels.

These seismic hazard definitions specified in this standard are appropriate as long as the seismic design or evaluation of the SSCs for these earthquake levels is conservatively performed. The level of conservatism of the evaluation for these hazards should increase

as one goes from Performance Category 1 to 4 SSCs. The conservatism associated with Performance Categories 1 and 2 should be consistent with that contained in the UBC (Ref. C-2), ATC-3 (Ref. C-1), or NEHRP (Ref. C-3) for normal or essential facilities, respectively. The level of conservatism in the seismic evaluation for Performance Category 4 SSCs should approach that used for nuclear power plants when the seismic hazard is designated as shown above. The criteria contained herein 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 procedures and acceptance criteria as one goes from Performance Category 1 to Performance Category 4.

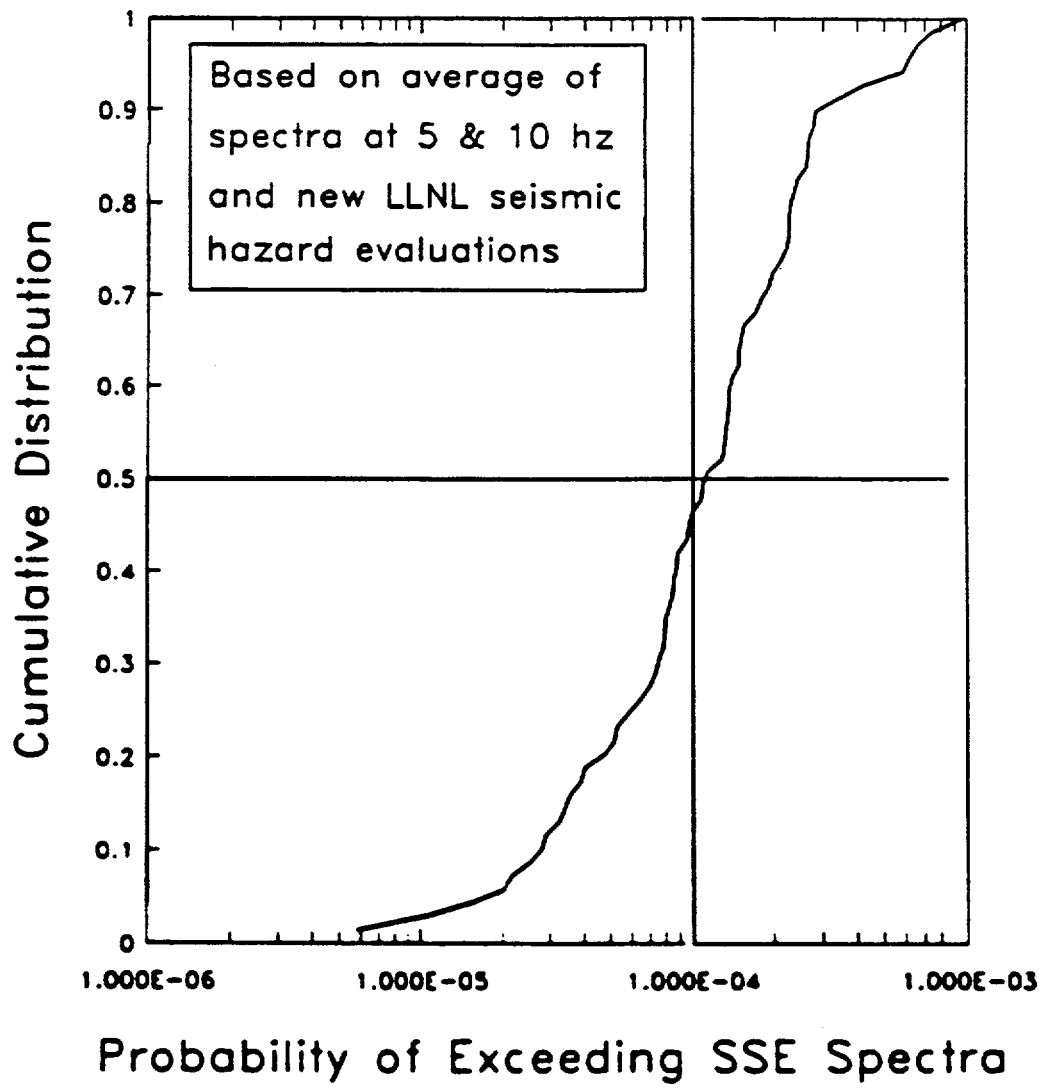


Figure C-9 Probability of Exceeding SSE Response Spectra

C.3.2 Earthquake Ground Motion Response Spectra

Design/evaluation Basis Earthquake (DBE) response spectra generally have the shape shown in Figure C-8. The DBE 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 spectral shapes are necessary in order to provide practical input. DBE ground motion at the site is defined in terms of smooth and broad frequency content response spectra in the horizontal and vertical directions defined at a specific control point. In most cases, the control point should be on the free ground surface. However, in some cases it might be preferable to define the DBE response spectra at some other location. One such case is when a soft (less than 750 feet/second shear wave velocity), shallow (less than 100 feet) soil layer at the ground surface is underlain by much stiffer material. In this case, the control point should be specified at the free surface of an outcrop of this stiffer material. Wherever specified, the breadth and amplification of the DBE response spectra should be either consistent with or conservative for the site soil profile, and facility embedment conditions.

Ideally, it is desirable for the DBE response spectrum to be defined by the mean uniform hazard response spectrum (UHS) associated with the seismic hazard annual frequency of exceedance, P_{ie}, over the entire frequency range of interest (generally 0.5 to 40 Hz). However, currently considerable controversy exists concerning both the shape and amplitude of mean UHS.

First, many existing mean UHS shapes are not consistent with response spectrum shapes derived from earthquake ground motion recordings. The DBE response spectrum should be consistent in shape with response spectrum shapes from ground motion recorded at similar sites for earthquakes with magnitudes and distances similar to those which dominate the seismic hazard at the specified annual frequency. Unless it can be demonstrated that the mean UHS shape is consistent with the response spectrum shapes obtained from appropriate ground motion records, the mean UHS should not be used.

Second, even for a specified ground motion parameter such as peak ground acceleration (PGA) or peak ground velocity (PGV), the mean estimate for a given hazard exceedance probability tends to be unstable between different predictors and tends to be driven by extreme upper bound models. Mean estimates should be used only when such estimates are stable. Mean estimates outside the range of 1.3 to 1.7 times the median

estimate are likely to suffer from the above problems. Because of these issues with regard to both mean estimates and UHS, the Department of Energy has published a standard (DOE-STD-1024-92) on the use of probabilistic seismic hazard estimates (Reference C-13).

Preferably, the median deterministic DBE response spectrum shape should be site-specific and consistent with the expected earthquake magnitudes, and distances, and the site soil profile and embedment depths. When a site-specific response spectrum shape is unavailable then a median standardized spectral shape such as the spectral shape defined in NUREG/CR-0098 (Reference C-15) may be used so long as such a shape is either reasonably consistent with or conservative for the site conditions.

C.3.2.1 DBE Response Spectra at High Frequencies

The C factor in the UBC base shear equation 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 Performance Category 2 and lower SSCs, earthquake loading is evaluated from the base shear equation in accordance with UBC seismic provisions with the exception that the ZC is determined from input design/evaluation response spectra. ZC as given by UBC provisions is plotted as a function of both natural period and natural frequency on Figure C-10. Also, Figure C-10 includes a typical design/evaluation spectra.

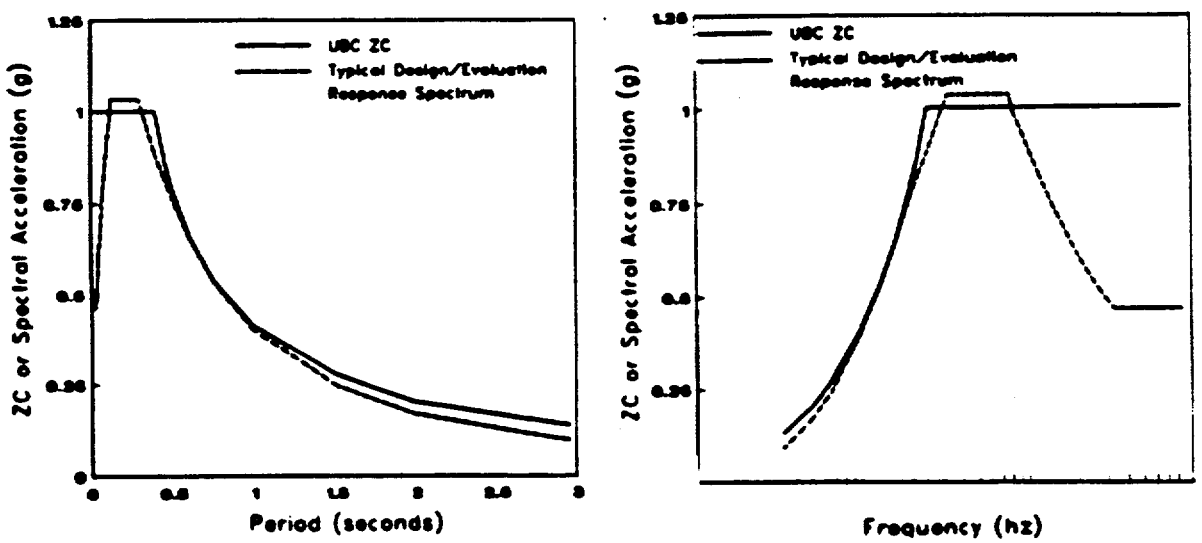


Figure C-10 Comparison of UBC ZC with Typical Design/Evaluation Response Spectrum

It is shown in Figure C-10 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 have a shape similar to that for ZC per the code provisions as shown in Figure C-10. In this manner, the recommended seismic evaluation approach for Performance Category 2 and lower SSCs closely follows the UBC provisions while utilizing seismic hazard data from site-dependent studies.

In the seismic design and evaluation criteria presented in Chapter 2, for Performance Category 3 and higher SSCs, DBE spectra are used for dynamic seismic analysis. However, in accordance with Reference C-5, 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 C-5. This requirement is illustrated in Figure C-1 1. Note that this requirement necessitates that response spectrum dynamic analysis be performed for building response evaluation. Alternately, the actual spectrum may be used for all modes if there is high confidence in the frequency evaluation and F_a 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.

The basis for using the maximum spectral acceleration in the low period range by both the Reference C-2 and C-5 approaches is threefold: (1) to avoid being unconservative when using constant response reduction coefficients, R_w, or inelastic energy absorption factors, F_a; (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 Ridden and Newmark (Ref. C-12), 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 energy absorption factors must be used for low period response if the actual spectra are used (i.e., the inelastic energy absorption factors are frequency dependent). Since constant inelastic energy absorption factors are used herein, increased spec-

tra 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, basemat 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.

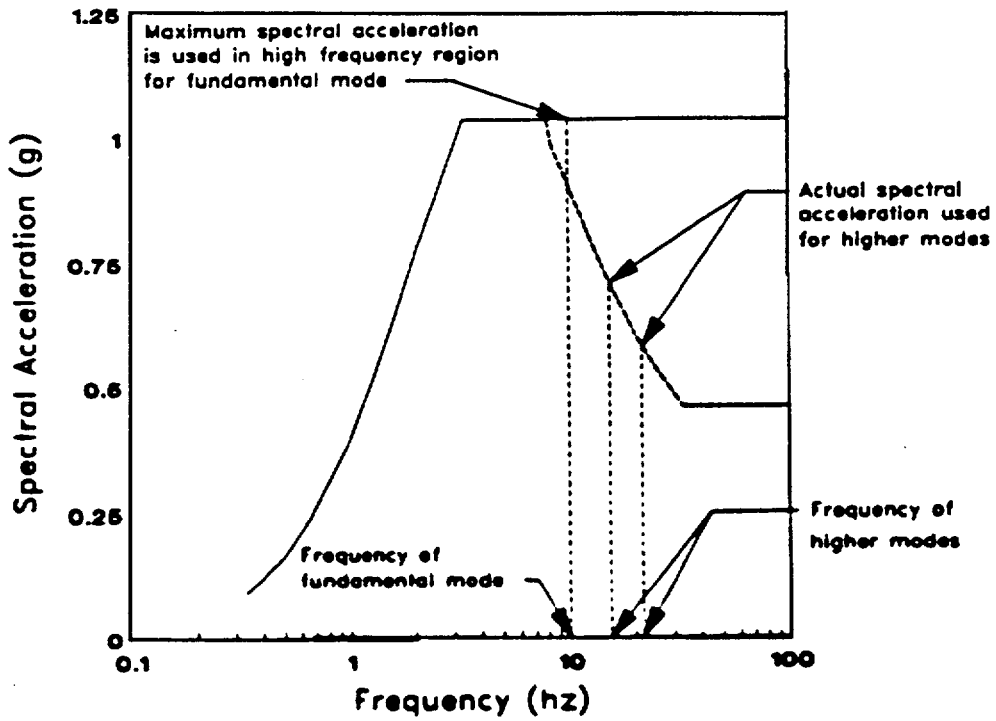


Figure C-11 Spectral Acceleration in the High Frequency Region for an Example Design/Evaluation Ground Response Spectrum

C.3.2.2 DBE Response Spectra Based on Generic Seismic Data

Spectral amplification depends strongly on site conditions. For this reason, it is generally required that response spectra to be used for the design or evaluation of hazardous DOE facilities (Performance Category 3 and higher SSCs) be evaluated from site-specific geotechnical evaluations. Development of response spectra from site-specific evaluations

has been discussed earlier in this section and in Reference C-13. However, for seismic design or evaluation of Performance Category 2 and lower SSCs, site-specific DBE response spectra are not required, if such data are not available.

Where a recent site-specific seismic hazard study is not available, it is permissible to determine DBE response spectra for Performance Category 2 and lower SSCs as the larger of the spectral values from the Uniform Building Code (UBC, Ref. C-2) or from generic seismic hazard evaluations such as UCRL-53582, Rev. 1 (Ref. C-14). UBC seismic input is based on regional seismicity. UBC values of peak ground acceleration, Z and spectral amplification, C are shown in Table C-4. UCRL-53582 spectra were developed from general site conditions and not from a site-specific geotechnical evaluation. It is also permissible to utilize these values for preliminary or final seismic design or evaluation of Performance Category 3 and higher SSCs. However, for final seismic design or evaluation of Performance Category 3 and higher SSCs, it is strongly recommended in DOE-STD-1024-92 (Ref. C-13) that site-specific DBE response spectra be developed and used to determine seismic loadings.

Peak ground accelerations at DOE sites for Performance Categories 1, 2, 3, and 4 are summarized in Table C-5. Table C-5a provides peak ground accelerations for sites away from tectonic plate boundaries. Table C-5b provides peak ground accelerations for California sites governed by tectonic plate boundaries. These ground acceleration values are taken from the most recent seismic hazard evaluation for the site as referenced in the tables. Where a recent seismic hazard evaluation is available, ground motion from that study is presented and ground motion from UCRL-53582 is only shown if a more recent evaluation does not exist. It should be noted that these ground motion values are shown for information only. There are ongoing studies on some of these sites and there probably will be future studies on other sites which could change the ground motions presented herein. Earthquake ground motion for design or evaluation must be established with approvals from the DOE.

If median site-specific spectral amplifications are available for these sites, they may be scaled by the peak ground accelerations in Table C-5 to establish the DBE response spectra. Alternately, a median standardized spectral shape such as the spectral shape defined in NUREG/CR-0098 (Reference C-15) may be used so long as such a shape is either reasonably consistent with or conservative for the site conditions.

Table C-4 DBE Ground Motion (g) from UBC

DOE Site	UBC Zone	Z	Peak ZC @ f_1	ZC(2 hz)
Kansas City	2A	0.15	0.41 @ 3.3	0.30
LANL	2B	0.2	0.55 @ 3.3	0.40
Mound	1	0.075	0.21 @ 3.3	0.15
Pantex Plant (S = 1.2)	1	0.075	0.21 @ 2.5	0.18
Rocky Flats	1	0.075	0.21 @ 3.3	0.15
Sandia, Albuquerque	2B	0.2	0.55 @ 3.3	0.40
Sandia, Livermore (S = 1.2)	4	0.4	1.10 @ 2.5	0.95
Pinellas Plant	0	0.0	0.0	0.0
Argonne-East	0	0.0	0.0	0.0
Argonne-West	2B	0.2	0.55 @ 3.3	0.40
Brookhaven (S = 1.2)	2A	0.15	0.41 @ 2.5	0.36
Princeton (S = 1.2)	2A	0.15	0.41 @ 2.5	0.36
INEL	2B	0.2	0.55 @ 3.3	0.40
Feed Materials	1	0.075	0.21 @ 3.3	0.15
Oak Ridge	2A	0.15	0.41 @ 3.3	0.30
Paducah (S = 1.2)	2A	0.15	0.41 @ 2.5	0.36
Portsmouth	1	0.075	0.21 @ 3.3	0.15
Nevada Test Site (S = 1.2)	3	0.3	0.83 @ 2.5	0.72
Hanford	2B	0.2	0.55 @ 3.3	0.40
LBL	4	0.4	1.10 @ 3.3	0.95
LLNL (S = 1.2)	4	0.4	1.10 @ 2.5	0.79
LLNL, 300-854	4	0.4	1.10 @ 3.3	0.79
LLNL, 300-834 & 836	4	0.4	1.10 @ 3.3	0.79
ETEC	4	0.4	1.10 @ 3.3	0.79
SLAC (S = 1.2)	4	0.4	1.10 @ 2.5	0.95
Savannah River (S = 1.2)	2A	0.15	0.41 @ 2.5	0.36

***** Table C-5a Maximum Horizontal Ground Surface Accelerations (g) as of 1993 at DOE Sites Excluding the Sites Located near Tectonic Plate Boundaries**

DOE Site	PC 1 - $2x10^{-3}$	PC 2 - $1x10^{-3}$	PC 3 - $5x10^{-4}$	PC 4 - $1x10^{-4}$	References
Kansas City Plant	.08	.10	.13	.21	UCRL-53582
Los Alamos (Average)	.12	.16	.21	.37	Woodward-Clyde,1993
Los Alamos (Range)	.10-.14	.14-.19	.17-.25	.31-.43	Woodward-Clyde,1993
Mound Laboratory	.12	.15	.18	.30	UCRL-53582
Pantex Plant	.08	.10	.13	.21	UCRL-53582
Rocky Flats Plants**	.13	.15	.17	.24	UCRL-53582
Sandia, Albuquerque	.17	.22	.28	.47	UCRL-53582
Pinellas Plant, Florida	.04	.05	.06	.11	UCRL-53582
Argonne-East	.09	.12	.15	.26	UCRL-53582
Argonne-West	.12	.14	.17	.24	UCRL-53582
Brookhaven	.12	.15	.18	.30	UCRL-53582
Princeton	.13	.16	.20	.33	UCRL-53582
INEL	.12	.14	.17	.24	UCRL-53582
Feed Materials	.10	.13	.16	.24	UCRL-53582
Oak Ridge, Rock	.08	.13	.19	*	Risk Eng. Inc., 1992
Paducah, Rock	.20	.30	.42	*	DOE Memo
Paducah, Soil	.20	.25	.35	*	DOE Memo
Portsmouth, Rock	.04	.06	.10	*	Risk Eng '92 & WES '93
Portsmouth, Soil	.10	.15	.19	*	Risk Eng '92 & WES '93
Nevada Test Site	.30(UBC)	.30(UBC)	.34	.46	UBC, UCRL-53582
Hanford 200 West Area	.10	.14	.20	.39	Geomatrix, 1993
Hanford 200 East Area	.09	.13	.19	.37	Geomatrix, 1993
Hanford 300 Area	.08	.12	.17	.33	Geomatrix, 1993
Hanford 400 Area	.09	.12	.17	.32	Geomatrix, 1993
Hanford 100 K Area	.10	.14	.20	.40	Geomatrix, 1993
Savannah River Plant	.10	.13	.18	.32	LLNL 1993, Draft

* Value not available and must be determined for Performance Category 4 SSCs at these sites.
** Bedrock slopes at Rocky Flats. This value is surface acceleration at an average soil depth at this site.
*** This table is for illustration only, to show variability among DOE facilities for performance categories. Design values for specific sites and facilities must be established in accordance with requirements in DOE-STD-1023-95.

Table C-5b Maximum Horizontal Ground Surface Accelerations (g) at DOE Sites Located near Tectonic Plate Boundaries

DOE Site	PC 1 $2x10^{-3}$	PC 2 & PC 3 $1x10^{-3}$	PC 4 $2x10^{-4}$	Reference
Sandia, Livermore, Ca	.47	.57	.82	Geomatrix, 1990
Lawrence Berkeley Laboratory	.55	.64	*	UCRL-53582
Lawrence Livermore Nat'l Lab. (LLNL)	.47	.57	.82	Geomatrix, 1990
LLNL, Site 300	.47	.57	.82	Geomatrix, 1990
Energy Technology & Engineering Center	.53	.59	*	UCRL-53582
Stanford Linear Accelerator Center	.45	.59	*	UCRL-53582

* Value not available from UCRL-53582, Rev. 1 and must be determined for Performance Category 4 SSCs at these sites.

C.3.3 Effective Peak Ground Motion

The peak ground accelerations reported in probabilistic seismic hazard assessments typically correspond to acceleration that would be recorded during an earthquake by a motion instrument. This instrumental acceleration may, in some cases, provide an excessively conservative estimate of the damage potential of the earthquake. Instead the effective peak acceleration based on repeatable acceleration levels with frequency content corresponding to that of structures is a better measure of earthquake damage potential. It is acceptable, but often quite conservative, to use the instrumental ground motion as direct input to the dynamic model of the structure. 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 below.

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 C-24 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. Generally, 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 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.

C.4 Evaluation of Seismic Demand (Response)

The earthquake design and evaluation criteria in DOE-STD-1020 generally follow the *Uniform Building Code* (UBC) provisions (Ref. C-2) for Performance Category 2 and lower SSCs and the DOD Tri-service manual for essential buildings (Ref. C-5) for Performance Category 3 and 4 SSCs as indicated in Figure C-12. For Performance Category 2 and lower SSCs, these seismic design and evaluation criteria employ the UBC provisions with the exception that site-specific information is used to define the earthquake input excitation used to establish seismic loadings (see Table C-6). The maximum ground acceleration and ground response spectra determined in the manner illustrated in Figure C-8 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. 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 criteria for Performance Category 2 and lower SSCs. This reduced seismic response is combined with non-seismic concurrent loads and then compared to code allowable response limits (or code ultimate limits combined with code specified load factors). Normally, relative seismic anchor motion (SAM) is not considered explicitly by model building code seismic provisions.

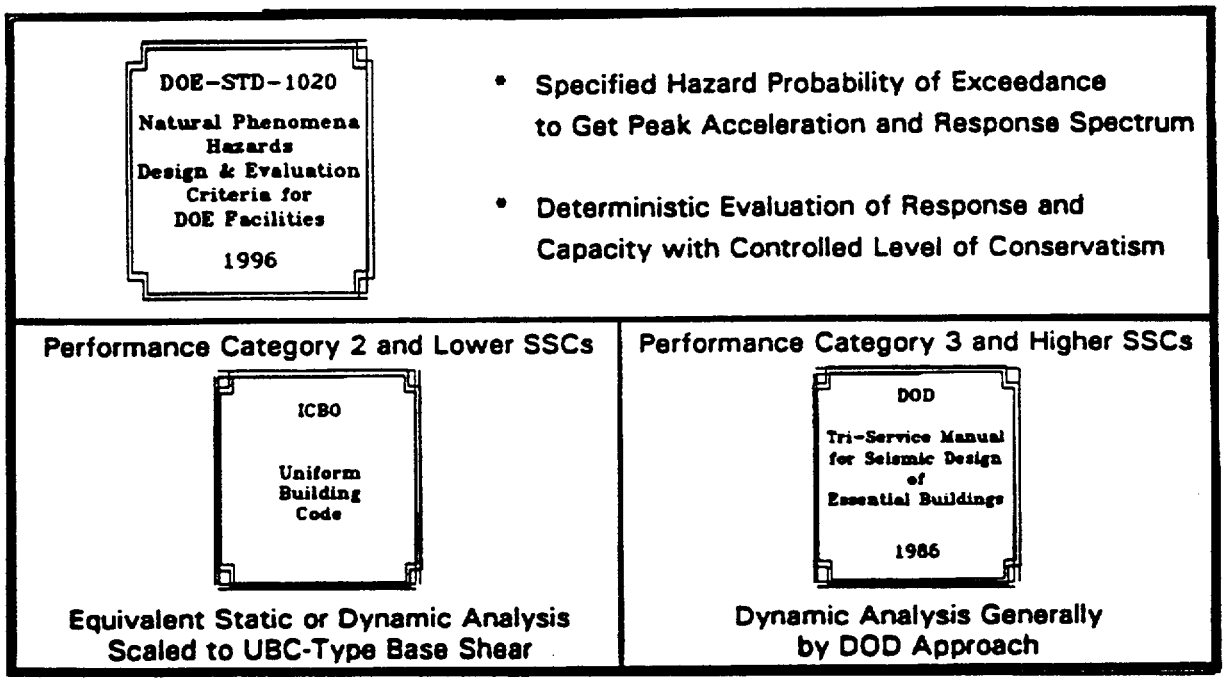


Figure C-12 Earthquake Provisions Basic Approach

The Uniform Building Code (UBC) has been followed for Performance Categories 1 and 2 because it is believed that more engineers are familiar with this code than other model building codes. The Interagency Committee on Seismic Safety in Construction (ICSSC) has concluded that the following seismic provisions are substantially equivalent:

1) 1994 Uniform Building Code (Ref. C-2)
2) 1991 NEHRP Recommended Provisions (Ref. C-3)
3) 1993 BOCA National Building Code (Ref. C-28)
4) 1994 SBCCI Standard Building Code (Ref. C-29)

These other model building codes may be followed provided site-specific ground motion data is incorporated into the development of earthquake loading similar to the manner described in this document for the UBC.

For Performance Category 3 and 4 SSCs, these seismic design and evaluation criteria specify that seismic evaluation be accomplished by dynamic analysis (see Table C-6). The recommended approach is to perform an elastic response spectrum dynamic analysis to evaluate elastic seismic demand on SSCs. However, inelastic energy absorption capability is recognized by permitting limited inelastic behavior. By these provisions, inelastic energy absorption capacity of structures is accounted for by the parameter, F_p. Elastically computed seismic response is reduced by F_p values ranging from 1 to 3 as a means of account-

ing for inelastic energy absorption capability for more hazardous facilities. F_u values are much lower than R_w values increasing the risk reduction ratio, R_R. By these provisions, only the element forces due to earthquake loading are reduced by F_u. This is a departure from the DOD manual (Ref. C-5) in which combined element forces due to all concurrent loadings are reduced. The same F_u values are specified for both Performance Categories of 3 and 4. In order to achieve different risk reduction ratios, R_R, appropriate for the different performance categories, the reduced seismic response is multiplied by a seismic scale factor, SF. Different seismic scale factors SF are specified for Performance Category 3 and 4. The resulting scaled inelastic seismic response is combined with non-seismic concurrent loads and then compared to code ultimate response limits. The design detailing provisions from the UBC, which provide ductility, toughness, and redundancy, are also required such that SSCs can fully realize potential inelastic energy absorption capability. Also, explicit consideration of relative seismic anchor motion (SAM) effects is required for Performance Category 3 and higher.

For Performance Category 3 or higher, the dynamic analysis based deterministic seismic acceptance criteria specified herein are independent of both the desired risk reduction ratio and the performance category specified, other than for the seismic scale factor, SF. Thus, the deterministic acceptance criteria herein may be used over a wide range of applications including special situations where the desired seismic performance goal differs from those specified in Tables C-1 and C-2.

Table C-6 General Description of Earthquake Provisions

Performance Category 2 and Lower SSCs	Performance Category 3 and Higher SSCs
Use 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 considering the mass & stiffness distribution of the structure. • Perform an elastic response spectrum analysis but to permit limited inelastic behavior. Elastic seismic response is reduced by the factor, F_u to obtain inelastic seismic demand. Explicitly account for relative displacement effects, where applicable.

C.4.1 Dynamic Seismic Analysis

As mentioned previously, complex irregular structures cannot be evaluated by the equivalent static force method because the simple formulas for distribution of seismic forces throughout the structure would not be applicable. For such structures, more rig-

orous dynamic analysis approaches are required. In addition, for very important or highly hazardous facilities, such as for Performance Categories 3 and higher, seismic design or evaluation must be based on a dynamic analysis approach. Dynamic analysis approaches lead to a greater understanding of seismic structural behavior; these approaches should generally be utilized for more hazardous facilities. Minimum requirements for dynamic analyses were presented in Chapter 2. It should be noted that the requirement for dynamic analysis does not also require complex dynamic models. For simple structures or components, very simple analyses can be performed as long as: (1) the input is represented by a response spectrum or time history; (2) important SSC frequencies are estimated or the peak of the input spectrum is used; and (3) resulting inertial forces are properly distributed and a load path evaluation is performed. Equivalent static force methods with forces based on the applicable response spectra may be used for equipment and distribution system design and evaluation.

In dynamic seismic 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. This DOE standard endorses ASCE 4 (Ref. C-16) for acceptable methods of dynamic analysis.

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 utilize the time-history of input motion to calculate responses using a time-step by time-step numerical procedure. When the input earthquake excitation is given in terms of response spectra, 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. It should be understood that the strict application of modal analysis assumes elastic response (stiffness remains constant) of the structure.

C.4.2 Static Force Method of Seismic Analysis

Seismic provisions in model building 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 structures, systems, or components 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 C.6.

Key elements of equivalent static force seismic evaluation methods are formulas that provide (1) total base shear; (2) fundamental period of vibration; (3) distribution of seismic forces with height of the structure; and (4) distribution of story forces to individual resisting elements including torsional considerations. 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 UBC provisions (Reference C-2) 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. The UBC 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 are combined with forces due to other loadings using code defined load factors and are compared to code defined strength or stress levels in order to evaluate whether or not the design is adequate for earthquake loads. In addition, in buildings, 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.

C.4.3 Soil-Structure Interaction

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. For very stiff structures (i.e., fixed base fundamental frequency of about 12 hz), the effects of SSI may be significant at shear wave velocities in excess of 3500 fps. In such a case, a fixed base support would not be appropriate.

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 C-37 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. It is always permissible to do the necessary soil-structure interaction studies 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 that the DBE ground motion applies at the foundation level of the structure. However, any frequency shifting due to SSI, when significant, must always be considered. If SSI effects are considered, the seismic analysis should be peer reviewed.

For structures subjected to earthquake excitation, the solution of the dynamic response of the coupled soil-structure system involves the following basic elements:

(i) Characterization of the site including evaluation of local soil/rock stratigraphy, low-strain soil and rock dynamic properties and soil nonlinearities at earthquake-induced strain levels, ground water location, and backfill configuration and dynamic properties.

(ii) Evaluation of free-field input excitation including the effects of local soil conditions. DBE ground motion including the effects of local soil conditions were discussed in Section C.3.

(iii) Development of a model adequately representing the mass, stiffness and damping of the structure.

(iv) Evaluation of foundation input excitation including scattering (modification) of the free-field motions due to the presence of the foundation soil excavation and behavior at the structure-foundation interface. This step is sometimes called the kinematic interaction problem.

(v) Evaluation of foundation stiffness or impedance functions defining the dynamic force-displacement characteristics of the soil.

(vi) Analysis of the coupled soil-structure system by solving the appropriate equations of motion.

Acceptable methods for considering SSI include multi-step impedance function approaches and single step direct methods as described in Sections 3.3.3 and 3.3.4 of ASCE 4 (Ref. C-16). SSI is further addressed in Wolf, 1985, 1988 (Refs. C-31 and C-32). SSI analysis methods and computer programs commonly used include:

(i) Soil spring or lumped parameter methods representing foundation impedances by soil springs and dashpots (see Table C-7) and using a two step solution procedure consisting of impedance analysis and SSI response analysis;

(ii) CLASSI computer program (Ref. C-33) employing 3-D continuum-half space model and multiple step analysis technique consisting of fixed base structure modal extraction analysis, foundation impedance and scattering analysis, and SSI response analysis;

(iii) SASSI computer program (Ref. C-34) employing a 3-D finite element foundation model and multiple step analysis technique consisting foundation impedance analysis and combined scattering and SSI response analysis; and

(iv) FLUSH (2-D) and ALUSH (axisymmetric) computer programs (Refs. C-35 and C-36) using a discretized finite element halfspace foundation model and solving for SSI response in a single step.

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). In lieu of a more sophisticated SSI evaluation, the following reduction factors may be conservatively applied to the input ground response spectra to account for the statistical incoherence of the input wave for a 150-foot plan dimension of the structure foundation (Ref. C-37):

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 struc tures for purposes of computing this reduction. For foundations consisting of individual column or wall footings, consideration of relative displacement between footings may also be required.

Table C-7 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	$k_x = k_1 2(1+\nu)G\beta_x\sqrt{BL}$	$k_x = k_1 \frac{32(1-\nu)GR}{7-8\nu}$	$c_x = c_1 k_x\,(static) R J \sqrt{\rho/G}$
Rocking	$k_\phi = k_2 \frac{G}{1-\nu}\beta_\phi B^2 L$	$k_\phi = k_2 \frac{8GR^3}{3(1-\nu)}$	$c_\phi = c_2 k_\phi (static) R \sqrt{\rho/G}$
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,

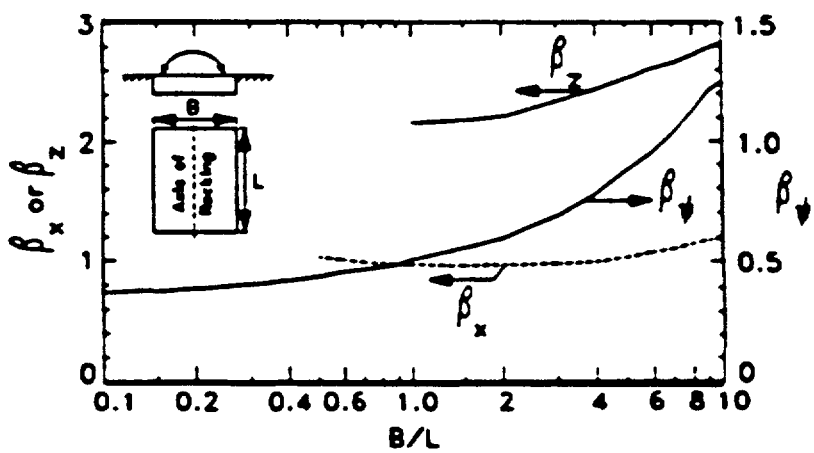
R = radius of the circular base mat,

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

$k_x, k_\phi, k_z, k_\theta$ = frequency dependent coefficients modifying the static stiffness or damping (Refs. C-39, C-40, C-41).
c_1, c_2, c_3, c_4



Constants β_x, β_z, and β_ϕ for a rectangular foundation

Foundation Input Motion. Developing foundation input motion including the variation of ground motion over the width of the foundation necessitates consideration of nonvertically propagating earthquake wave motion. Vertically propagating shear and compressional waves may be assumed for an SSI analysis provided that torsional effects due to nonvertically propagating waves are considered. An accidental eccentricity of 5 percent can be implemented to incorporate possible effects of non-vertically incident waves (Ref. C-37). Variation of amplitude and frequency content with depth may be considered for embedded structures. Input motion to the boundaries of soil models shall be compatible with the design earthquake specified at the finished grade in the free-field. The motions shall be established as a function of the soil properties, the type of waves propagating during the earthquake, and the type of boundary assumed. The analyses to establish boundary motions shall be performed using mathematical models and procedures compatible with those used in the SSI analysis. The design earthquake control motion defined at the free-field surface may be input to the massless rigid foundation in an impedance method SSI analysis. When the control motion is used as the input, rotational input due to embedment or wave passage effects need not be considered. Alternatively, the input motion to the massless rigid foundation may be modified from the control motion at the free-field surface to incorporate embedment or wave passage effects, provided the corresponding computed rotational inputs are also used in the analysis.

Soil Properties and Modeling. Subsurface material properties shall be determined by field and laboratory testing, supplemented as appropriate by experience, empirical relationships, and published data for similar materials (refer to DOE-STD-1022 - Reference C-38). Soil properties needed for conducting equivalent-linear analyses include: shear modulus, damping ratio, Poisson's ratio, and total unit weight. The shear modulus and material damping ratio used to evaluate foundation impedance shall be values compatible with the shear strain level induced in the foundation medium during earthquake excitation. The shear modulus decrease and damping increase with increasing shear strain in soils shall be accounted for when performing an SSI analysis. The behavior of soil, though recognized to be nonlinear with varying soil shear strain, can often be approximated by linear techniques. Nonlinear soil behavior may be accounted for by: (1) using equivalent linear soil material properties typically determined from an iterative linear analysis of the free-field soil deposit; or (2) performing an iterative linear analysis of the coupled soil-structure system. The variation in shear modulus and damping as a function of shear strain for sands, gravelly soils, and saturated clays can be found in References C-42, C-43, and C-44. At very small strains ($\leq 10^{-4}$ percent), the material (hysteretic) damping ratio shall not exceed 2% of critical. In no case should the material damping ratio exceed 15% of critical (Ref. C-30). Poisson's ratio, in combination with shear modulus defines the Young's modulus of the

material in accordance with the theory of elasticity. For saturated soils, the behavior of the water phase shall be considered in evaluating Young's modulus and in selecting values of Poisson's ratio.

Determination of Foundation Impedances. Foundation impedances may be evaluated by mathematical models or by published formulas giving soil spring and dashpot coefficients. Since the foundation medium relative to the structure dimensions is semi-infinite, dynamic modeling of the foundation medium is generally accomplished using a half space model. Such a model permits waves generated at the structural-foundation resulting from the dynamic response of the structure to be dissipated into the far-field of the model. This leads to the phenomenon called radiation damping. The three-dimensional phenomenon of radiation damping and layering effects of foundation soil shall be considered. When significant layering exists in the foundation medium, it should be modeled explicitly or its effects such as significant frequency dependency of the foundation impedance functions and reduction of radiation damping should be considered in the analysis.

When mathematical models are used, either the continuum half space or the discretized halfspace may be employed. The discretized halfspace by finite element or finite difference models requires the use of model-consistent wave transmitting boundaries to accurately simulate radiation damping and to eliminate artificial wave reflections which are not negligible. The lower boundary shall be located far enough from the structure that the seismic response at points of interest is not significantly affected or a transmitting boundary below the model could be used. Soil discretization (elements or zones) shall be established to adequately reproduce static and dynamic effects.

Embedment of the foundation increases the foundation impedances. For structures that are significantly embedded, embedment effects should be included in the SSI analysis. These effects can be incorporated by using available simplified methods for some geometries (Refs. C-39 and C-40). The potential for reduced lateral soil support of the structure due to tensile separation of the soil and foundation should be considered when accounting for embedment effects. One method to comply with this requirement (Section 3.3.1.9 of ASCE 4-86) is to assume no connectivity between structure and lateral soil over the upper half of the embedment or 20 feet, whichever is less. However, full connectivity may be assumed if adjacent structures founded at a higher elevation produce a surcharge equivalent to at least 20 feet of soil. For shallow embedments (depth-to-equivalent-radius ratio less than 0.3), the effect of embedment may be neglected in obtaining the impedance function, provided the soil profile and properties below the basemat elevation are used for the impedance calculations.

Dynamic analysis of the coupled soil-structure system. When the SSI system parameters are assumed to be frequency-independent constants, the equations of motion may be solved by time domain solution procedures, such as either the direct integration or the standard modal time history response analysis methods. Due to relatively large soil radiation damping which can cause relatively large modal coupling, the application of standard modal superposition time history methods requires the determination of "composite" modal damping ratios. The most frequently used are the stiffness-weighted method presented in Section 3.1.5.3 of ASCE 4-86 and the transfer function matching method (Ref. C-45). When the SSI system parameters are frequency-dependent, the equations of motion are generally solved by complex frequency response methods. The computation of the Fourier transform of the input motion should be performed using sufficient time and frequency increments in order to allow for frequency components of motion up to 25 hz to be accurately reproduced unless a lesser limit can be justified.

Uncertainties. There are uncertainties in the soil properties and parameters used for SSI analysis. Therefore, a relatively wide variation of soil properties is recommended such that a conservative structure response calculation may be expected. An acceptable method to account for uncertainties in the SSI analysis, as given in ASCE 4-86 Section 3.3.1,7, is that the soil shear modulus shall be varied between the best estimate value times $(1 + C_v)$, and the best estimate value divided by $(1 + C_v)$, where C_v is a coefficient of variation. In general, a C_v value of about 1.0 covers uncertainties in soil properties. If there are sufficient site soils data, it is permissible to evaluate soil property uncertainty by probabilistic techniques. The minimum value of C_v shall be 0.50.

C.4.4 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.

C.4.4.1 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 which would prevent oscillation in a system excited by an initial perturbation.

Chapter 2 reports typical structural damping values for various materials and construction (Refs. C-5, C-15, C-16, and C-17) for three different response levels. Response Level 3 values correspond to strains beyond yielding of the material, and, they are recommended for usage along with other provisions of this document for seismic response analyses of existing Performance Category 3 and higher SSCs. Post yield damping values are judged to be appropriate because DOE-STD-1020 acceptance criteria permit post yield behavior (i.e., F_a greater than unity). It is judged that Response Level 3 damping will be reached if seismic response levels approach the criteria limits. If seismic response is less than the criteria limits, the level of damping used is not important to the design or evaluation. Similar post yield acceptance criteria and damping are used in Reference C-5. For design of new facilities, it is recommended that Response Level 2 damping be used. Response Level 2 damping introduces a small amount of conservatism compared to Response Level 3. It is judged that the use of lower damping for design will not result in significant additional cost and that it is desirable to have slightly increased seismic forces for design of new facilities. For Performance Category 2 and lower SSCs, the criteria 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 C-2. In this case, it is recommended that the 5 percent damped spectra be used for all Performance Category 2 and lower SSCs to be consistent with building code evaluation methods. The spectral amplification factor in model building codes is based upon 5 percent damped spectral amplification.

The Response Level 1 and 2 damping values given in Chapter 2 are to be used to evaluate in-structure response spectra or displacements to be used in seismic interaction evaluations. In these cases, it is important to use damping which is consistent with stress levels reached in the majority of the lateral force resisting system so as to not be unconservative in the evaluation of input to structure-supported components or input for interaction

considerations. Even though seismic design is performed in accordance with these criteria which permit limited inelastic behavior, actual stress levels in structures may be relatively low due to unintentional conservatism introduced during the design process or because the design may be governed by loads other than earthquake loads.

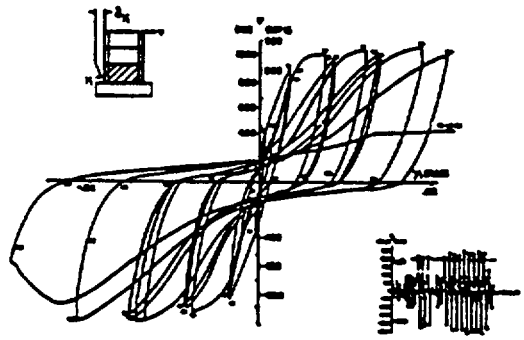
C.4.4.2 Inelastic Behavior

Energy absorption in the inelastic range of response of structures and equipment to earthquake motions can be very significant. Figure C-13 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. Generally, an accurate determination of inelastic behavior necessitates dynamic nonlinear analyses performed on a time-history time step integration 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 energy absorption factors. Spectral reduction factors and inelastic energy absorption factors 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 energy absorption factors are briefly described below.

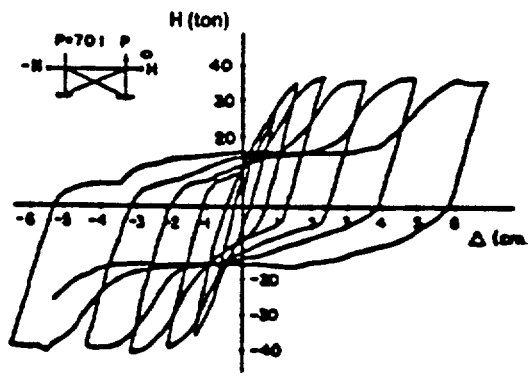
1. *Spectral reduction factors* - Structural response is determined from a response spectrum dynamic analysis. The spectral reduction factors are used to deamplify the elastic acceleration response spectrum producing an inelastic acceleration response spectrum which is used in the analysis. The resulting member forces are combined with concurrent non-seismic member forces and compared to ultimate/limit-state level stresses to determine structural adequacy.
2. *Inelastic energy absorption factors* - 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 elastically computed member forces are reduced by member specified inelastic energy absorption factors to give the inelastic demand. The inelastic demand is combined with concurrent non-seismic demand and the resulting total demand is compared to the capacity determined from member forces at ultimate/limit-state stress level to determine structural adequacy.

The spectral reduction factors and inelastic energy absorption factors 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 inelastic energy absorption factors are used. For less ductile shear walls or braced frames,

lower reduction values or inelastic energy absorption factors are employed. For more hazardous facilities, lower reduction factors or inelastic energy absorption factors 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.



a. Shear force-distortion for concrete wall test (Ref. C-18)



b. Lateral force-displacement for steel braced frame (Ref. C-19)

Figure C-1 3 Cyclic Load-Deflection Behavior of Concrete Shear Walls and Steel Braced Frames

The inelastic energy absorption factor approach is employed for design or evaluation of Performance Category 3 and higher SSCs by these criteria. This approach is recommended in the DOD manual for seismic design of essential buildings (Ref. C-5). Inelastic energy absorption factors are called F_a in these criteria. Inelastic energy absorption factors have the advantage over spectral reduction factors in that different values may be speci-

fied 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 energy absorption factor 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 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 Performance Category 2 and lower SSCs by these criteria.

Reduction coefficients, R_w, to be used for evaluation of Performance Category 2 and lower SSCs and recommended inelastic energy absorption factors, F_μ, for Performance Category 3 and higher SSCs are presented in Chapter 2 for various structural systems. R_w factors used by this standard are those directly from Reference C-2. The F_μ factors presented in Chapter 2 were established to approximately meet the performance goals for structural behavior of the SSCs as defined in Chapter 2 and as discussed in Section C.2. These factors are based both on values given in Reference C-5 and on values calculated from code reduction coefficients in a manner based on the performance goals. Reference C-20 describes the detailed method for establishing the values of F_μ.

The code reduction coefficients, R_w, by the UBC approach and inelastic energy absorption factors, F_μ, by the DOD approach differ in the procedures that define permissible inelastic response under extreme earthquake loading. By the 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 energy absorption factor, 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 criteria 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 UBC and DOD provisions. By the 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 energy absorption factor (as shown in Figures 3-2 and 3-3 of Reference C-5). By the DOD interaction formulas, the inelastic energy absorption factor is applied only to the bending moment, and axial forces are unaffected. In addition, the inelastic energy absorption factors are low compared to ratios for other types of members such as beams. The DOD approach for columns is followed by these guidelines for Performance Category 3 and higher SSCs.

Performance Category 3 and higher SSCs can be evaluated by elastic dynamic analyses. However, limited inelastic behavior is permitted by utilizing inelastic energy absorption factors, F_μ. Recommended F_μ values for various structural elements, lateral force resisting systems, and materials of construction are presented in Chapter 2. The inelastic energy absorption factor, F_μ, is related to the amount of inelastic deformation that is permissible for each type of structural element. 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 Performance Categories 3 and higher as compared to the larger R_w factors for Performance Categories 2 and lower, the margin of safety for that category is effectively increased (i.e., the risk reduction ratio, R_n, is increased), and the probability of damage is reduced in accordance with the performance goals. Note that the F_μ values are employed with acceptance criteria based on ultimate stress limits with unity load factors while the R_w values are employed with acceptance criteria based on either ultimate stress limits compared with response including load factors or allowable stress limits.

The inelastic energy absorption factor is defined as the amount that the elastic-computed seismic demand may exceed the capacity of a component without impairing the performance of the component. Thus, the elastic-computed seismic demand D_e may be divided by an inelastic energy absorption factor F_μ to obtain an inelastic seismic demand D_{in}. This inelastic energy absorption factor F_μ should be defined by:

$$F_\mu = F_{\mu 5\%} \qquad \text{(C-7)}$$

where $F_{\mu 5\%}$ is the estimated inelastic energy absorption factor associated with a permissible level of inelastic distortions specified at about the 5% failure probability level.

If practical, it would be preferable to perform nonlinear analysis on the structure or component being evaluated in order to estimate $F_{\mu 5\%}$ for use in Equation C-7 to define F_μ. Some guidance on estimating $F_{\mu 5\%}$ is given in Section C.4.4.3. However, such analyses

are often expensive and controversial. Therefore, a set of standard F_μ values is provided in Chapter 2 for common elements. These F_μ values may be used in lieu of performing non-linear analyses, so long as the following cautions are observed.

A significant difference between the R_w factors from the UBC and the F_μ factors to be used for Performance Category 3 and higher SSCs 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 Chapter 2, 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 steel special moment resisting frame (SMRF) or (2) in the case of a steel concentric braced frame, F_μ of 1.75 may be used.

In the evaluation of existing facilities, it is necessary to evaluate an appropriate value of F_μ to be used. The F_μ values in Chapter 2 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 current Uniform Building Code (UBC, Ref. C-2). It is highly unlikely that existing facilities will satisfy the seismic detailing requirements of the current UBC if they were 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 this criteria to compensate for non-conforming structural details. As a result, F_μ values must be reduced from the values given in Chapter 2. 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 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. C-46) and ATC-22, "A Handbook for Seismic Evaluation of Existing Buildings" (Ref. C-47) be reviewed for guidance on this subject.

The use of F_μ values of 1.5 and greater for concrete walls is conditional on wall cracking occurring as described in Table C-2, with stable wall behavior constituting acceptable wall performance. If a lesser amount of wall cracking is required, then F_μ should be 1.0.

The F_μ values for ductile failure modes (i.e., greater than 1.0) assume that steel reinforcing bars, structural steels, metal tank shells, and anchorage will remain ductile during

the component's entire service life. It is assumed that the metal will retain at least a 6% uniaxial elongation strain capability including the effects of welding. If this metal can become embrittled at some time during the service life, F_μ should be 1.0.

In some cases, reinforcement details in older facilities do not satisfy the development length requirement of current codes (Refs. C-48 and C-49). In these instances, the potential exists for a ductile failure mode associated with yielding of the reinforcement to become a less ductile mode associated with bond failure. Data exists (Ref. C-50), however, indicating that bond failure modes retain a reasonable amount of ductility provided that the reinforcement is suitably confined within the region of the potential bond failure. The confinement may be provided by a cover of at least 2.5 bar diameters or by ties (stirrups) spaced no further than 5 bar diameters apart. If this confinement is provided, a strength of the reinforcement equal to the yield strength of the steel times the ratio of actual to required development length may be used in the capacity evaluations. In these cases, the factor F_μ should be limited to 1.0. If the confinement is not provided, the reinforcement should be omitted in the capacity evaluations.

For low-ductility failure modes such as axial compression or shear in concrete walls or columns and wall-to-diaphragm, wall-to-column, or column-to-base connections, the F_μ values are 1.0. In most cases, such stringent limits can be relaxed somewhat, as described below, because most components also have a ductile failure mode which when reached is likely to limit the demand in the low-ductility failure modes. Unless the component has a seismic capacity in the ductile failure mode significantly in excess of its required capacity, inelastic distortions in this ductile failure mode will likely limit the scaled inelastic seismic demand D_μ in the low-ductility failure modes. Since greater conservatism exists in code capacities C_c for low-ductility failure modes than for ductile failure modes, the failure will be controlled by the ductile failure mode so long as the low-ductility failure mode code capacity is at least equal to the ductile failure mode capacity. Thus, for low-ductility failure modes, the factored seismic demand D_μ can be limited to the lesser of the following:

1) D_μ given by dividing elastic demand by F_μ, or
2) $D_\mu = C_c \cdot D_m$ computed for the ductile failure mode, where C_c is the ductile failure mode capacity.

Therefore, for example, connections do not have to be designed to have code capacities C_c greater than the code capacities C_c of the members being connected, or the total factored demand D_m, whichever is less. Similarly, the code shear capacity of a wall does not have to exceed the total shear load which can be supported by the wall at the code flexural capacity of the wall. Finally, the horizontal seismic-induced axial force in a

moment frame column can be limited to the axial force which can be transmitted to the column when a full plastic hinge mechanism develops in the frame where the plastic hinge capacities are defined by the code ultimate flexural capacities.

Several other factors may be noted about the inelastic energy absorption factors, F_μ:

1. Chapter 2 F_μ values assume that good seismic design detailing practice (Reference C-21) 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 energy absorption factors should be used instead of those presented in Chapter 2.
2. Chapter 2 F_μ 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 energy absorption factors should be used than those presented herein.
3. Inelastic energy absorption factors are provided in Chapter 2 for the structural systems described in References C-2 and C-5. For other structural systems, engineers must interpolate or extrapolate from the values given based on their own judgement in order to evaluate inelastic energy absorption factors that are consistent with the intent of these criteria.

C.4.4.3 Guidance on Estimating the Inelastic Energy Absorption Factor F_μ

Introduction - It is recognized that the inherent seismic resistance of a well-designed and constructed structure is usually much greater than that expected based on elastic analysis. This occurs largely because nonlinear behavior is mobilized to limit the imposed forces.

Two general methods currently exist for treating the nonlinear behavior of a structure. One approach is to perform a time history nonlinear analysis and compare the maximum element demand ductility to a conservative estimate of its ductility capacity.

An alternate means of accounting for the inelastic energy dissipation of civil structures and equipment at response levels above yield is the use of an elastic energy absorption factor F_μ based on a ductility modified inelastic response spectrum approach (References C-12 and C-22 through C-24).

In general, the analyst would first make an estimate of a permissible inelastic distortion corresponding to about a 5% failure probability level. For example, for a low-rise concrete shear wall or concentric braced frame structure, a permissible total story distortion of 0.4% of the story height for in-plane drift would provide an adequately low probability of severe structural distress, and thus would result in an adequately conservative distortion

criterion for overall structural failure (Reference C-25). However, such a distortion would result in severe cracking of a low rise concrete shear wall structure such that if there were safety related equipment mounted off the wall, the anchorage on this equipment might fail. To protect such anchorage, permissible total story distortions would more appropriately be limited to the range of 0.2% to 0.25% of the story height for low rise concrete shear walls. Once a permissible distortion has been selected, the inelastic energy absorption factor F_μ may be determined from nonlinear analysis of an appropriate model of the structure using multiple realistic input time histories. First, the input time history is scaled to a level at which the elastic computed elastic demand is equal to the yield (or ultimate) capacity. Then, the input is further scaled until the distortion resulting from a time history nonlinear analysis reaches a permissible value. The inelastic energy absorption factor F_μ is equal to this additional scaling factor.

Alternately, a simplified nonlinear analysis procedure may be used at least for cantilever type structures. First, the analyst must estimate the nonlinear deformed shape of the structure corresponding to the maximum permissible distortion being reached in the story with the highest value of the ratio of the demand to the capacity. Then the system ductility μ is estimated from:

$$\mu = \frac{\sum W_i \delta_{T_i}}{\sum W_i \delta_{e_i}} \qquad \text{(C-8)}$$

where W_i is the inertial weight applied at each story of the structure, δ_{T_i} is the total displacement relative to the base of each story corresponding to the permissible total distortion occurring in the critical story, and δ_{e_i} is the elastic displacement relative to the base corresponding to a unit value of the ratio of the elastic demand to the capacity for the critical story. For a single story, Equation C-8 simplifies down to a story ductility, μ_s of:

(C-9)

where δ_T is the total permissible story displacement and δ_y is the yield displacement (Demand/Capacity equals unity). However, for multistory structures, μ from Equation C-8 is always less than μ_s from Equation C-9 except when the nonlinear distortions are spread throughout the structure which is very unlikely. The following equation can be used to relate μ to μ_s:

$$\mu = 1 + F_k(\mu_s - 1) \qquad \text{(c-10)}$$

where F_k is a reduction factor to convert a story ductility estimate to a system ductility estimate. For a well designed structure in which the ratio of the demand to the capacity does not differ by more than a factor of about 1.3 over the structure height, F_k will typically range from 0.5 to 0.75. In these cases, Equation C-10 may be used with a conservatively estimated F_k of 0.5 in lieu of Equation C-8 or nonlinear time history analyses.

Once the permissible system ductility μ has been established, many approaches can be used for estimating F_μ. For broad, smooth input spectra and moment-frame structures with essentially full elasto-plastic nonlinear hysteretic loops, either the Newmark-Hall (Reference C-15) or Riddell-Newmark (Reference C-12) approach is commonly used. However, for concrete shear wall structures or braced frames which have severely pinched hysteretic loops, Kennedy, et.al. (Reference C-24) has shown that these approaches are likely to be slightly nonconservative. For such structures, the approach of Reference C-24 is preferred.

The following provides an example application of this simplified nonlinear analysis procedure for estimating F_μ for a concrete shear wall structure.

For purposes of this illustration, a three-story structure with the properties shown in Figure C-14 will be used. This figure shows the weights, W, at each story, the elastic stiffnesses, K, and the ultimate capacities, V_u, for the walls between story levels. This structure has a fundamental frequency f of 8.25 Hz.

Assuming a damping parameter β of 7%, for a reference 1.0 g NUREG/CR-0098 median spectrum (Reference C-15), at f equal to 8.25 Hz the elastic spectral acceleration is:

$$S_A(f, \beta) = 1.86g \qquad \text{(C-11)}$$

and the elastic response of this structure is given in Table C-8. For this reference spectrum response, the ratio of the elastic demand to the capacity ranges from 1.02 for the first story wall to 1.32 for the second story. Thus yielding will initially occur in the first story wall, and the elastic displaced shape at the onset of yielding is given by δ_e in Table C-8. Note in Table C-8 that the minimum value of V/V_u (i.e., 1.02) is used to calculate δ_e since this corresponds to the first element which reaches yield (i.e., level 1).

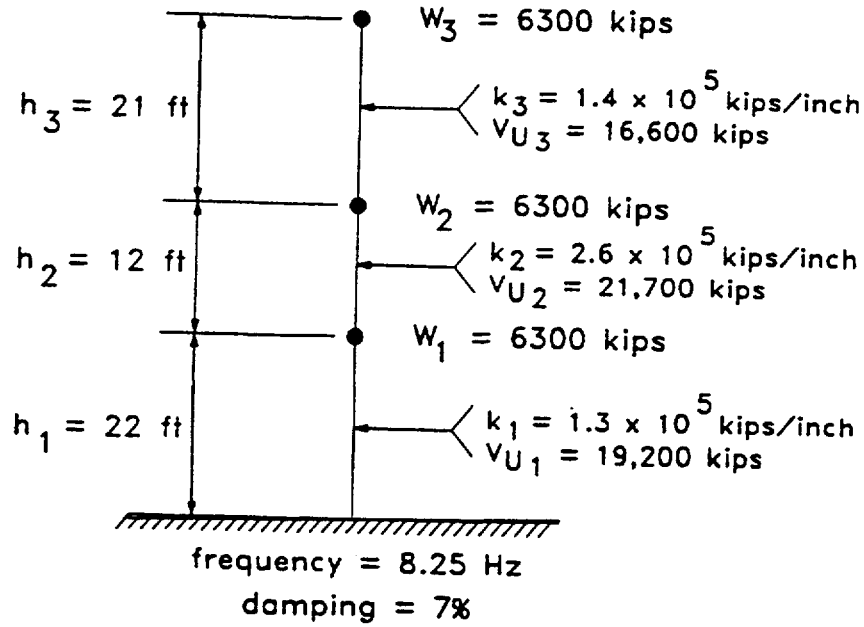


Figure C-14 Three Story Shear Wall Structure

Table C-8 Elastic Response to Reference 1. 0g NUREG/CR-0098 Spectrum (7% damping)

Level	Demand		Capacity/Demand V_y/V_e (inch)	Yield Displacement $\delta_y = \delta_e(V_y/V_e)_{max}$ (inch)
	Displacement δ_e (inch)	Shear V_e (kips)		
3	0.304	13,400	1.24	0.309
2	0.208	16,400	1.32	0.211
1	0.145	18,900	1.02	0.147

Computation of System Ductility - In accordance with the recommendations given above, a permissible total story distortion of 0.4% of the total story height will be selected for the critical first story. Thus:

$$\delta_{T_1} = 0.004(22\ ft)(12\ inch/ft) = 1.06\ inch \qquad \text{(C-12)}$$

and the story ductility μ_s from Equation C-9 is:

$$\mu_s = \frac{1.06}{0.147} = 7.2 \tag{C-13}$$

From Equation C-10, the system ductility μ is expected to lie within the range of:

$$\mu = 4.1 \, to \, 5.6 \tag{C-14}$$

However, a more accurate estimate of μ may be obtained from Equation C-8 after estimating the inelastic deformed shape. A slightly conservative estimate of the inelastic deformed shape may be obtained by assuming that all of the nonlinear drift occurs in the story with the lowest ratio of the capability to the demand (the first story for this example). The other stories retain the same differential drifts as given by δ_e in Table C-8. Thus:

δ_{T_1} = 1.06 inch

δ_{T_2} = 1.12 inch

δ_{T_3} = 1.22 inch

and from Equation C-8:

$$\mu = \frac{6300(1.22) + 2100(1.12) + 2500(1.06)}{6300(0.309) + 2100(0.211) + 2500(0.147)} = 4.6 \tag{C-15}$$

Computation of Inelastic Energy Absorption Factor - For concrete shear wall structures, it is recommended that the inelastic energy absorption factor F_e be computed by the effective frequency/effective damping approach of Reference C-24, as summarized herein. For this example, it will be assumed that the force-deflection relationship on initial loading is elasto-perfectly plastic with an ultimate capacity V_u. Thus, the ratio of secant to elastic frequency is given by:

$$(f_s/f) = \sqrt{1/\mu} = \sqrt{1/4.6} = 0.466 \tag{C-16}$$

Then, the effective frequency is given by:

$$(f_e/f) = (1 - A) + A(f_s/f) \tag{C-17}$$

where:

$$A = C_f[1-(f_s/f)] \leq 0.85$$

For ground motions with strong durations greater than one second, C_f may be approximated as 2.3. Thus, for (f_s/f) less than or equal to 0.63 one may take A equal to 0.85. For this example:

$$(f_e/f) = 0.15 + 0.85(0.466) = 0.546 \tag{C-18}$$

$$f_e = 0.546(8.25Hz) = 4.5Hz$$

The effective damping β_e may be estimated from:

$$\beta_e = (f_s/f_e)^2[\beta + \beta_H] \tag{C-19}$$

where β is the elastic damping (7% in this case) and β_H is the pinched hysteretic damping which can be approximated by:

$$\beta_H = 11\%[1-(f_s/f)] = 11\%(.534) = 5.9\% \tag{C-20}$$

for strong durations greater than one second. Thus, for this example:

$$\beta_e = \left(\frac{.466}{.546}\right)^2[12.9\%] = 9.4\% \tag{C-21}$$

For the reference 1.0g NUREG/CR-0098 spectrum, at f_e equal to 4.5 Hz and β_e equal tc 9.4%, the spectral acceleration is given by:

$$S_A(f_e, \beta_e) = 1.68g \tag{C-22}$$

and the inelastic energy absorption factor is given by:

$$F_\mu = \left[\frac{f_e}{f_s}\right]^2 \frac{S_A(f,\beta)}{S_A(f_e,\beta_e)} = \left[\frac{.546}{.466}\right]^2\left[\frac{1.86}{1.68}\right] = 1.52 \tag{C-23}$$

C.5 Capacities

C.5.1 Capacity Approach

For existing components, material strength properties should be established at the 95% exceedance actual strength levels associated with the time during the service life at which such strengths are a minimum. If strengths are expected to increase during the service life, then the strength of an existing component should be its value at the time the evaluation is performed. If strengths are expected to degrade during the service life, then strengths to be used in the evaluation should be based on estimated 95% exceedance strengths at the end of the service life. Whenever possible, material strengths should be based on 95% exceedance values estimated from tests of the actual materials used at the facility. However, when such test data is unavailable, then code minimum material strengths may be used. If degradation is anticipated during the service life, then those code minimum strengths should be further reduced to account for such degradation (for example, long term thermal effects on concrete).

For new designs, material strength properties should be established at the specified minimum value defined by the applicable code or material standard. If degradation is anticipated during the service life, then those code minimum strengths should be further reduced to account for such degradation. The use of code specified minimum strength values or 95% exceedance strength levels (i.e., 95% of measured strengths exceed the design/evaluation strength level) is one location in the design/evaluation process where intentional conservatism is introduced.

In general, for load combinations which include the DBE loading, capacities C_c to be used should be based upon code specified minimum ultimate or limit-state (e.g., yield or buckling) capacity approaches coupled with material strength properties as specified above. For concrete, the ultimate strength approach with the appropriate capacity reduction factor, ϕ, included as specified in Chapter 26 of the UBC (Ref. C-2) are used. Also, ACI-318 (Ref. C-49) or ACI-349 (Ref. C-48) provide useful information. For structural steel, UBC Chapter 27 and referenced UBC Standards are used. The LRFD (applicable UBC Standard and Ref. C-5 1) limit-state strength approach with the appropriate capacity reduction factor, ϕ, included is preferred. However, the Plastic Design (applicable UBC Standard, Part 2, Ref. C-52 or Chapter N, Ref. C-53) maximum strength approach may be used so long as the specified criteria are met. The plastic design strengths can be taken as 1.4 times the allowable shear stress for members and bolts and 1.7 times other allowable stresses specified in Refs. C-52 or C-53 unless another factor is defined in the specified

code. For ASME Section III Division 1 components, ASME Service Level D (Ref. C-54) capacities should be used. In some cases, functional failure modes may require lesser limits to be defined (e.g., ASME Mechanical Equipment Performance Standard, Ref. C-55).

For existing facilities, in most cases, the capacity evaluation equations should be based on the most current edition of the appropriate code, particularly when the current edition is more conservative than earlier editions. However, in some cases (particularly with the ACI and ASME codes), current code capacities may be more liberal than those specified at the time the component was designed and fabricated, because fabrication and material specification requirements have become more stringent. In these later cases, current code capacities will have to be reduced to account for the more relaxed fabrication and material specifications that existed at the time of fabrication. In all cases, when material strength properties are based on code minimum material strengths, the code edition enforced at the time the component was fabricated should be used to define these code minimum material strengths.

C.5.2 Seismic Design and Detailing

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 and redundant 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 (to provide redundancy and to reduce potential for differential displacement); adequate equipment anchorage; non-uniform, non-symmetrical configuration of 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 pro-

duce transient, limited energy loading on facilities. Because of these earthquake characteristics, well-designed and well-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 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 has been prepared as part of this project (UCRL-CR-106554, Ref. C-21). 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 distribution systems include: (1) configuration; (2) continuous and redundant load paths; (3) detailing for ductile behavior; (4) tying systems together; (5) influence of nonstructural components; (6) function 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, distribution systems, 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. These also include imbalance in strength and failure mechanisms even if elastic stiffnesses and masses are balanced in plan. Vertical irregularities can produce large local forces during an earthquake, These include large differences or eccentricities 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 Ser-

vices 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 distribution systems) 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.

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.

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 low ductility 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 low ductility failure 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 Elements 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 building primary structural systems 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 out-of-plane 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 and individual footings must be adequately tied together when the foundation media is capable of significant differential motion.

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, large bore piping, 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 distribution systems, 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.

Survival of Emergency Systems - In addition to preventing damage to structures, equipment, distribution systems, nonstructural elements, etc., it is necessary for emergency systems and lifelines to perform their safety-related functions following the earthquake. Means of ingress and egress (such as stairways, elevator systems, doorways) must remain functional for personnel safety and for control of hazardous operations. Fire protection systems should remain operational after an earthquake if there is a significant potential for seismic-induced fire. Normal off-site power and water supplies have been vulnerable during past earthquakes. Emergency on-site power and water supplies may be required 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 functioning of systems that prevent unacceptable facility damage or destruction due to fires or explosions.

C.6 Special Considerations for Systems and Components

C.6.1 General

The seismic adequacy of equipment and distribution systems is often 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. C-56). 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 distribution systems include:

1. Equipment or distribution systems supported within a structure respond to the motion of the supporting 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 distribution systems supported at two or more locations within a structure may be stressed due to both inertial effects and relative support displacements.
3. Equipment or distribution systems 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 needs to 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 including mass and stiffness properties 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, does not require additional seismic analysis or testing if it could be shown to be adequately anchored and representative of the experience data.
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. These deficiencies can also threaten adjacent safety related items or personnel through spatial interaction.

Engineered anchorage is one of the most important factors affecting seismic performance of systems or components and is required for all performance categories. It is intended that anchorage have both adequate strength and sufficient stiffness to perform its function. Types of anchorage include: (1) cast-in-place bolts or headed studs; (2) expansion or epoxy grouted anchor bolts; and (3) welds to embedded steel plates or channels. The most reliable anchorage will be achieved by properly installed cast-in-place bolts or headed studs, undercut type expansion anchors, or welding. Other expansion anchors are less desirable than cast-in-place, undercut, or welded anchorage for vibratory environments (i.e., support of rotating machinery), for very heavy equipment, or for sustained tension supports. Epoxy grouted anchorage is considered to be the least reliable of the anchorage alternatives in elevated temperature or radiation environments.

Evaluation of facilities following past earthquakes has demonstrated that ductile structures with systems and components which are properly anchored have performed very well. As a result, properly engineered anchorage of systems and components is a very important part of the seismic design criteria. Wherever possible, these criteria encourage the use of larger and deeper embedment than minimum calculated anchorage as well as the use of cast-in-place and undercut-type expansion anchors. It is recommended that minimum anchor bolt size be 1/2 inch in diameter regardless of calculated anchorage requirements. Furthermore, it is recommended that anchorage embedment be longer than needed, wherever practical.

For new design of systems and components, seismic qualification will generally be performed by analysis or testing as discussed in the previous sections. However, for existing systems and components, it is anticipated that many items will be judged adequate for seismic loadings on the basis of seismic experience data without analysis or testing. Seismic experience data has been developed in a usable format by ongoing research programs sponsored by the nuclear power industry. The references for this work are the Senior Seismic Review and Advisory Panel (SSRAP) report (Reference C-60) and the Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment (Reference C-61). Note that there are numerous restrictions ("caveats") on the use of this data as described in the SSRAP report and the GIP. It is necessary to conduct either seismic analyses or shake table testing to demonstrate sufficient seismic capacity for those items that cannot be verified by seismic experience data or for items that are not obviously inherently rugged for seismic effects. There is an ongoing DOE program on the application of experience data for the evaluation of existing systems and components at DOE facilities.

In early 1982, the Seismic Qualification Utility Group (SQUG) was formed for the purpose of collecting seismic experience data as a cost effective means of verifying the seismic adequacy of equipment in existing nuclear power plants. Sources of experience data include: (1) the numerous non-nuclear power plants and industrial facilities with equipment similar to that in nuclear plants which have experienced major earthquakes and (2) shake table tests which had been performed to qualify safety-related equipment for licensing of nuclear plants. This information was collected and organized and guidelines and criteria for its use were developed. The GIP is the generic means for applying this experience data to verify the seismic adequacy of mechanical and electrical equipment which must be used in a nuclear power plant during and following the occurrence of a design level earthquake.

In order to utilize earthquake experience data to demonstrate seismic adequacy of equipment, four conditions are required to be met:

1. The seismic motion at the equipment location must be enveloped by the Experience Data Bounding Spectrum or the Generic Equipment Ruggedness Spectrum (GERS).
2. The equipment must fall within the bounding criteria for a given class of similar equipment which have survived strong earthquake shaking or past qualification tests.
3. The anchorage of the equipment must be shown to be adequate to survive design level seismic loads.
4. The equipment must meet the inclusion or exclusion rules, also called caveats, to determine whether the equipment has important characteristics and features necessary to be able to verify its seismic adequacy by this approach.

The use of earthquake experience data to verify the seismic adequacy of equipment requires considerable engineering judgement. As a result, the use of these procedures should be given special attention in the peer review process.

For Performance Category 2 and lower SSCs, seismic evaluation of equipment or nonstructural elements supported by a structure can be based on the total lateral seismic force as given in the UBC. For Performance Category 3 and higher SSCs, the seismic evaluation of equipment and distribution systems can necessitate the development of floor response spectra representing the input excitation. Once seismic loading is established, seismic capacity can be determined by analysis, testing, or, if available, the use of seismic experience data. It is recommended that seismic evaluation of existing equipment and distributions systems be based on experience data whenever possible.

C.6.2 Seismic Interaction

During the occurrence of an earthquake, it is possible for the seismic response of one structure, system, or component to affect the performance of other structures, systems,

and components. This sequence of events is called seismic interaction. Seismic interactions which could have an adverse effect on SSCs shall be considered in seismic design and evaluation of DOE facilities. Cases of seismic interaction which must be considered include:

1. Structural failing and falling.
2. Proximity.
3. Flexibility of attached lines and cables.
4. Flooding or exposure to fluids from ruptured vessels and piping systems.
5. Effects of seismically induced fires.

Structural failing and falling is generally prevented by single-failure seismic design criteria as described in other portions of this document. An interaction problem arises where a higher category (such as Performance Category 4) SSC (target) is in danger of being damaged due to the failure of overhead or adjacent lower category (such as Performance Category 1, 2, or 3) SSCs (source) which have been designed for lesser seismic loads than the higher category SSC (target). Lower category items interacting with higher category items or barriers protecting the target items need to be designed to prevent adverse seismic interaction. If there is potential interaction, the source does not move to the performance category of the target but remains in its own category based on its own characteristics. However, the source is subject to additional seismic design requirements above those for its own performance category. These requirements are that the source (or barrier) shall be designed to maintain structural integrity when subjected to the earthquake ground motion associated with the performance category of the target.

Impact between structures, systems, or components in close proximity to each other due to relative motion during earthquake response is another form of interaction which must be considered for design and evaluation of DOE SSCs. If such an impact could cause damage or failure, there should be a combined design approach of sufficient separation distance to prevent impact, and adequate anchorage, bracing, or other means to prevent large deflections. Note that even if there is impact between adjacent structures or equipment, there may not be potential for any significant damage such that seismic interaction would not result in design measures being implemented. An example of this is that a 1 inch diameter pipe cannot damage a 12 inch diameter pipe irregardless of the separation distance. The designer/evaluator shall justify and document these cases.

Design measures for preventing adverse performance from structural falling and falling and proximity seismic interaction modes include: (1) strength and stiffness; (2) separation distance; and (3) barriers. Sources may be designed to be strong enough to prevent

falling or stiff enough to prevent large displacements such that adverse interaction does not occur. To accomplish this, the source item shall be designed to the structural integrity design criteria of the target item. Maintaining function of the source item under this increased seismic design requirement is not necessary. Source and targets can be physically separated sufficient distance such that, under seismic response displacements expected for target design criteria earthquake excitation, adverse interaction will not occur. Barriers can be designed to protect the target from source falling or source motions. Barriers shall also be designed to the structural integrity design criteria of the target item. In addition, barriers shall be designed to withstand impact of the source item without endangering the target.

Another form of seismic interaction occurs where distribution lines such as piping, tubing, conduit, or cables connected to an item important to safety or production have insufficient flexibility to accommodate relative movement between the important item and adjacent structures or equipment to which the distribution line is anchored. For DOE SSCs, sufficient flexibility of such lines shall be provided from the important item to the first support on nearby structures or equipment.

Other forms of seismic interaction result if vessels or piping systems rupture due to earthquake excitation and cause fires or flooding which could affect performance of nearby important or critical SSCs. In this case, such vessels or piping systems must continue to perform their function of containing fluids or combustibles such that they shall be elevated in category to the level of the targets that would be endangered by their failure.

C.7 Special Considerations for Existing Facilities

It is anticipated that these criteria 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. C-63), a DOD manual (Ref. C-64), and in ATC-14, "Evaluating the Seismic Resistance of Existing Buildings" (Ref. C-46) and ATC-22, "A Handbook for Seismic Evaluation of Existing Buildings" (Ref. C-47). In addition, guidelines for upgrading and strengthening equipment are presented in Reference C-56. Also, guidance for evaluation of existing equipment by experience data is provided in Reference C-61. 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 DBE 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-is 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 including the effects of erosion and corrosion as appropriate. 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, erosive and 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. 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-is design detailing rather than using the inelastic energy absorption factors presented in Chapter 2. An existing facility may not have seismic detailing to the desired level and upon which the inelastic energy absorption factors 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 equipment and distribution systems, and other 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 design and detailing considerations, 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 might be readily upgraded.

Once the as-is condition of a facility has been verified and deficiencies or weak links have been identified, detailed seismic evaluation and/or upgrading of the facility as necessary 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-is configuration is evaluated instead of several configurations in an iterative manner (as is often 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 criteria presented in this standard and good seismic design practice had been employed, 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 criteria of this chapter, several alternatives can be considered:

1. If an existing SSC is close to meeting the criteria, a slight increase in the annual risk to natural phenomena hazards can be allowed within the tolerance of meeting the target performance goals. Note that reduced criteria for seismic evaluation of existing SSCs is supported in Reference C-63. As a result, some relief in the criteria can be allowed by performing the evaluation using hazard exceedance probability of twice the value specified for new design for the performance category of the SSC being considered.
2. The SSC may be strengthened such that its seismic resistance capacity is sufficiently increased to meet these seismic criteria. When upgrading is required, it should be accomplished in compliance with unreduced criteria (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 performance 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 SSC may be shown to be adequate. Alternatively, a probabilistic assessment might be undertaken in order to demonstrate that the performance goals can be met.

C.8 Quality Assurance and Peer Review

Earthquake design or evaluation considerations discussed thus far address recommended engineering practice that maximizes earthquake resistance of structures, systems, and components. It is further recommended that designers or earthquake consultants employ special quality assurance procedures and that their work be subjected to independent peer review. Additional earthquake design or evaluation considerations include:

a. Is the SSC constructed of known quality materials that meet design plans and specifications for strength and stiffness?
b. Have the design detailing measures, as described above, been implemented in the construction of the SSC?

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

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

a. Understand the seismic response of the SSC.

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 low ductility response.

d. Provide material testing and construction inspection which assures construction-/fabrication as intended by the designer.

All DOE structures, systems, and components must be designed or evaluated utilizing an earthquake engineering quality assurance plan as required by 10 CFR 830.120.6C (Ref. C-65) and similar to that recommended by *Recommended Lateral Force Requirements and Commentary,* Seismology Committee, Structural Engineers Association of California (Ref. C-66). The level of rigor in such a plan should be consistent with the performance category and a graded approach. In general, the earthquake engineering quality assurance plan should include:

Performance Categories 1, 2, 3, and 4

* 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.
* 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. The accuracy of the computer program and the analysis results must be verified.
* 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.

Performance Categories 2, 3, and 4

* All aspects of the seismic design or evaluation must include independent peer review. 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. The peer review is to be

performed by independent, qualified personnel. The peer reviewer must not have been involved in the original design or evaluation. If the peer reviewer is from the same company/organization as the designer/evaluator, he must not be part of the same program where he would be influenced by cost and schedule considerations. Individuals performing peer reviews must be degreed civil/mechanical engineers with 5 or more years of experience in seismic evaluations. Note that it can be very beneficial to have the peer reviewer participating early in the project such that rework can be minimized.

C.9 Alternate Seismic Mitigation Measures

Seismic Base Isolation - An innovative technology for mitigating the effects of earthquakes on structures is seismic base isolation. With this technology, a flexible isolation system is placed between the structure and the ground to decouple the structure from the potentially damaging motion of an earthquake. Ideally, an isolation system shifts the natural period of an isolated structure above the predominant period range of an earthquake. In addition to the period shift, isolation changes the dynamic response of the structure due to nonlinear hysteretic behavior of the isolation system and the flexibility of the isolation system compared to that of the structure. An isolation system essentially transforms the large accelerations from earthquake motion in to large displacements of the isolation system. A main attribute of seismic base isolation is that it substantially limits damage in a structure by significantly reducing the forces and interstory drift that are generated during an earthquake. For a design-level earthquake, the displacements in a structure are essentially limited to a rigid body displacement with negligible interstory drift. Additionally, the seismic demand is limited to the base shear or base acceleration transferred through the isolation system. By reducing forces and interstory drift generated in a structure, seismic base isolation provides protection for the structure and its contents so that a structure can remain operational during and immediately following an earthquake, Seismic base isolation may be an earthquake resistant design option that provides increase structural performance as compared with conventional seismic design. In contrast to traditional design techniques of strengthening and anchoring, a base isolation system dissipates seismic energy so that a new or existing SSC can be designed for lower seismic forces.

The UBC (Ref. C-2), beginning with the 1991 edition, contains regulations for the design of seismic isolated structures. Efforts are currently underway to determine how these regulations can be adopted within the DOE. Without specific guidance or criteria for the use of seismic base isolation in the DOE, it is recommended that the regulations in the UBC be carefully followed with the same considerations as outlined in Chapter 2. As discussed in Chapter 2, the UBC provisions are appropriate for Performance Category 2 and

lower SSCs. It is not recommended that seismic base isolation be considered for Performance Category 3 or 4 SSCs until there is specific guidance or criteria for its use in the DOE.

There are important technical and economic issues that should be considered before applying seismic base isolation to a SSC. Technical issues include the local site conditions, the structural configuration of the SSC, additional structural considerations for the SSC in order to properly accommodate seismic base isolation, and the interaction of the isolated SSC and adjacent SSCs. Economic issues include additional design and analysis efforts beyond that which is typically required for traditional design techniques, special provisions to meet regulatory requirements, and the techniques for properly evaluating the cost/benefit ratios of a base isolated SSC. While these technical and economic issues are not all-inclusive, they provide an indication of the care and precautions that should be exercised when considering seismic base isolation. Adequate evaluations are needed to determine, on a case-by-case basis, if seismic base isolation is a viable design option or if it is not an appropriate design solution.

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Appendix D
Commentary on Wind Design and Evaluation Criteria

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

D.1 Wind Design Criteria

Design goals are established for SSCs in Performance Categories 1 through 4. Design or evaluation of SSCs requires that the performance goals be met by selecting an appropriate hazard exceedance probability and utilizing sufficient conservatism in the methodologies and assumptions to assure the performance goals are met or exceeded.

A consensus standard, ANSI/ANS 2.3-1983 (Ref. D-1), which provides guidelines for estimating tornado and straight wind characteristics at nuclear power plant sites is an acceptable alternative approach to wind hazard assessment and design. However, the standard, which establishes tornado hazard probabilities at the 10^{-5}, 10^{-6}, and 10^{-7} levels on a regional basis, was not adopted by the Natural Phenomena Hazards Panel for the following reasons:

(a) The document is intended for siting of commercial nuclear power plants. Criteria are not necessarily appropriate for DOE SSCs.

(b) Site-specific hazard assessments were performed for each DOE site; it is not necessary nor desirable to use regional criteria

(c) Although published in 1983, the ANSI/ANS Standard is based on 15 year old technology

(d) Although ANSI/ANS Standard is a consensus document, it has not been approved by the U.S. Nuclear Regulatory Commission.

Instead of the ANSI/ANS Standard, a uniform approach to wind design is proposed herein, which is based on procedures of ASCE 7. The ASCE 7 document is widely accepted as the most technologically sound consensus wind load standard in the U.S.

The uniform approach to design for wind loads treats the types of windstorms (straight, hurricane and tornado) the same. Since ASCE 7 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 ASCE 7 is applied for determining wind pressures on

structures or net forces on systems and components. The additional effects of atmospheric pressure change (APC) and missile impact produced by tornadoes must also be considered at some sites.

The following argument is presented to justify the uniform approach to wind design. ASCE 7 addresses the physical characteristics of wind, including variation of wind speed with height and terrain roughness, effects of turbulence, and the variation of wind pressure over the surface of a building. Wind effects addressed in ASCE 7 can be detected and measured on wind tunnel models and on full-sized structures. Furthermore, evidence of the physical effects of wind found in wind tunnel and full-size 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 SSCs 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 D-1 lists specific examples where the appearance of damage from the three types of windstorms is identical. Many other examples could be given. The conclusion reached is that the proposed uniform approach is reasonable for estimating wind loads produced by straight winds, hurricanes and tornadoes.

D.2 Tornado Hazard Assessment

The traditional approach for establishing tornado criteria is to select extremely low exceedance probabilities. The precedence was established in specifying tornado criteria for the design of commercial nuclear power plants. An annual exceedance probability of 1×10^{-7} was adopted circa 1960 when very little was known about tornado effects from an engineering perspective. Much has been learned since 1960, which suggests that larger exceedance probabilities could be adopted. Some increase over the 1×10^{-7} is justified, especially for facilities that pose substantially smaller risks than commercial nuclear plants. However, two factors make it possible and desirable to use relatively low tornado hazard probabilities: 1) straight and hurricane winds control the criteria for probabilities down to about 1×10^{-4} and 2) additional construction costs to achieve low tornado probabilities are relatively small, when compared to earthquake design costs. For these reasons, the tornado hazard probabilities are set lower than straight winds and hurricanes. They also are set lower than earthquake and flood hazard probabilities.

Table D-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, Irvine, California 1977	Residence, Hurricane Frederick 1979	Apartment building, Omaha, Nebraska 1975
Wall corners	Metal building, Irvine, 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

A somewhat arbitrary, but quantitative approach is used to determine if a particular DOE site should be designed for tornadoes. Hazard assessments for both straight winds and tornadoes for each DOE site are presented in Reference D-2. The intersection of the straight wind and tornado hazard curves determines if tornadoes should be included in the design and evaluation criteria. If the exceedance probability at the intersection is greater than or equal to $2x10^{-5}$, tornadoes are a viable threat at the site. If the exceedance probability is less than $2x10^{-5}$, straight winds control the design or evaluation criteria. The concept is illustrated in Figure D-1. Straight wind and tornado hazard curves are shown for Oak Ridge National Laboratory (ORNL) and Stanford Linear Accelerator Center (SLAC). The SLAC curves intersect at an exceedance probability of approximately $2x10^{-7}$, indicating that tornadoes are not a viable threat at the California site. On the other hand, the intersection of the ORNL curves is at $6x10^{-5}$ suggesting that tornadoes should be included in the design and evaluation criteria. Design wind speeds for the 25 DOE project sites were selected on this basis.

D.3 Load Combinations

The ratios of hazard probabilities to performance goal probabilities (risk reduction factor) for the Performance Categories in Table D-2 are an approximate measure of the conservatism required in the design to achieve the performance goal. The ratio is largest for SSC

Performance Categories 1 and 2, and is progressively smaller for Performance Categories 3 and 4 for winds and tornadoes. The trend is just the opposite from earthquake design. The reason for the decreasing trend in wind is because we use smaller hazard probabilities and thus need a lessor degree of conservatism in Performance Categories 3 and 4.

Conservatism can be achieved in design by specifying factors of safety for Allowable Stress Design (ASD) and load factors for Strength Design (SD). These factors for straight wind should be obtained from applicable material design standards. Consistent with the ratios in Table D-2, the loading combinations recommended for tornado design and evaluation of DOE SSCs are given in Table D-3.

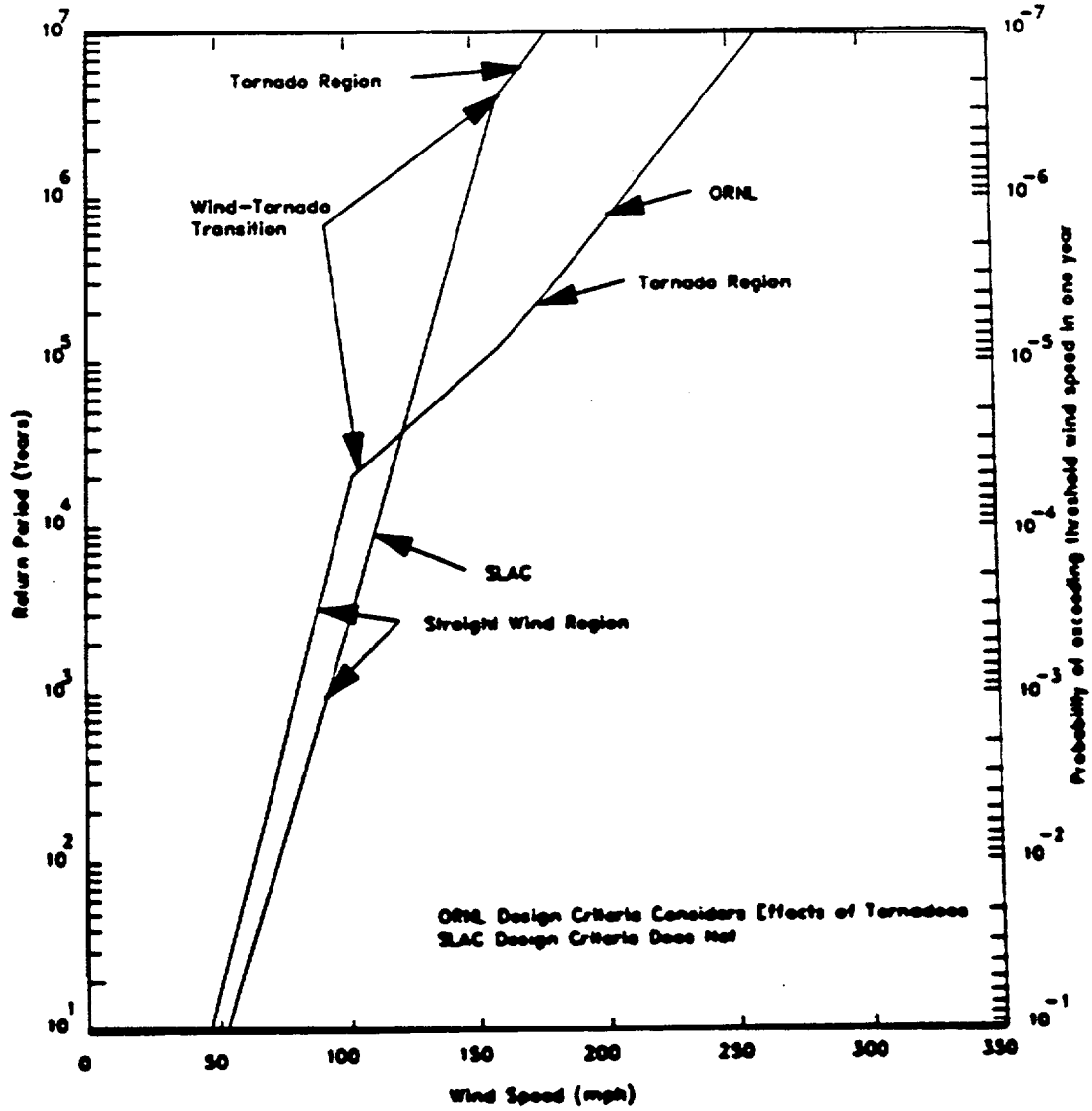

Figure D-1 Straight Wind and Tornado Regions of Wind Hazard Curves

Table D-2 Ratio of Hazard Probabilities to Performance Goal Probabilities

Performance Category	Performance Goals	Hazard Probability	Ratio of Hazard to Performance Probability
Straight Winds			
1	10^{-3}	$2x10^{-2}$	20
2	$5x10^{-4}$	10^{-2}(1)	20
3	10^{-4}	10^{-3}	10
4	10^{-5}	10^{-4}	10
Tornadoes			
3	10^{-4}	$2x10^{-5}$	1/5
4	10^{-5}	$2x10^{-6}$	1/5

(1) 2×10^{-2} with $1 = 1.07 \approx 10^{-2}$

Table D-3 Recommended Tornado Load Combinations for Performance Categories 3 and 4

ASD	$\frac{1.0}{1.6}[D + W_t]$ $\frac{1.33}{1.6}[0.75(D + L + L_r + W_t)]$ $\frac{1.5}{1.6}[0.66(D + L + L_r + W_t + T)]$
SD	$D + W_t$ $D + L + L_r + W_t$ $D + L + L_r + W_t + T$

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

SD = Strength Design
Use ϕ factors appropriate for building material

D = Dead load

L = Live load

L_r = Roof live load

W = Straight wind load

W_t = Tornado load, including APC if appropriate

T = Temperature load

The 1.6 denominator represents the factor of safety for material allowable stress, effectively removing this unneeded conservatism. The 1.33 and 1.5 factors negate the 0.75 and 0.66 factors permitted in ASD.

ASD is typically used for the design of steel, timber and masonry construction. Allowable stresses for the material and the type of loading (axial, shear, bending, etc.) are determined from applicable codes and specifications. The specified load combinations for ASD for Performance Categories 1 and 2 should be taken from the applicable material design

standard (e.g. ACI or AISC) for straight winds. Load combinations for Performance Categories 3 and 4 can be less conservative than for Performance Categories 1 and 2. Because the ratio of hazard to performance probability is smaller by a factor of two, it is judged that the load combinations can be reduced by 10 percent. The load combinations for Performance Categories 3 and 4 for straight winds should reflect this reduction. The hazard to performance probabilities for tornadoes is more than satisfied by the hazard probability, as indicated by the ratio 1/5. The tornado load combinations for ASD Performance Categories 3 and 4 were somewhat arbitrarily chosen, based on engineering judgment.

Strength Design (SD) has been used for the design of reinforced concrete structures since about 1977 (Ref. D-4). Recently a strength design approach was introduced for steel construction which is called Load and Resistance Factor Design (LRFD) (Ref. D-5). Strength design concepts are currently being developed for use with timber and masonry construction. With SD, the nominal strength of the material is reduced to account for uncertainties in material and workmanship. The reduced material strengths must be greater than or equal to the factored loads in order to satisfy a postulated limit state. The required conservatism is reflected in the load factors for loads involving straight winds. In this case, the load factors for Performance Categories 3 and 4 are increased by ten percent. Load factors for Performance Categories 1 and 2 are recommended in References D-3, D-4 and D-5. Since the performance goals are satisfied by the tornado hazard probabilities, unit value of load factors can be used for SD. Unit values are justified in this case, because the material reduction factors account for uncertainties associated with materials. The load factors for tornadoes are consistent with recommendations for commercial nuclear power plants as given in ACI 349 (Ref. D-6) for concrete and ANSI/AISC N690-1984 (Ref. D-7) for steel.

D.4 Windborne Missiles

Windborne missile criteria specified herein are based on windstorm damage documentation and computer simulation of missiles observed in the field. Reference D-8 documents the occurrence of classes of missiles that are picked up and transported by straight winds and tornadoes. Computer simulation of tornado missiles is accomplished using a methodology developed at Texas Tech University. The method is similar to one published in Reference D-9.

The timber plank missile is typical of a class of missiles that are frequently found in the windstorm debris. The 2x4 timber plank weighing 15 lbs is typical of the debris from damaged

or destroyed residences, office trailers and storage shacks. It can be carried to heights up to 200 ft in strong tornadoes. The 3-in. diameter standard steel pipe is typical of a class of missiles, which includes small diameter pipes, posts, light-weight rolled steel sections and bar joists. These objects are not likely to be carried to heights above 100 ft because of their larger weight to surface area ratio. Automobiles, storage tanks, trash dumpsters are rolled and tumbled by high winds and can cause collapse of walls, columns and frames. These heavy missiles are not picked up by winds consistent with the design criteria, they simply roll and tumble along the ground.

The missile wall and roof barriers recommended herein were all tested at the Tornado Missile Impact Facility at Texas Tech University. The impact tests are documented in Reference D-8. Structural response tests are not available for automobile impacts. Theoretical treatment of structural response calculations are given in References D-10, D-11, and D-12. Barriers that have not been tested such as grills, doors, wall cladding, tanks, mechanical ducts, etc should be tested in order to certify their performance.

Several empirical equations have been proposed for estimating the impact resistance if concrete and steel barriers. The equations were developed for use in the design of commercial nuclear power plants, and may not be applicable to the missile criteria specified herein. See Reference D-8 for a discussion of empirical missile impact equations.

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Appendix E
Effects of Natural Phenomena Hazards

E.1 Effects of Earthquakes

For most facilities, the primary seismic hazard is earthquake ground shaking. These criteria specifically cover the design and evaluation of buildings, equipment, distribution systems, and other structures for earthquake ground 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 compacted 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 natural 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.

The response of structures to earthquake ground shaking depends on the characteristics of the supporting soil. The amplitude and frequency of the response of massive, stiff structures founded or embedded in a soil media can be significantly affected by

soil-structure interaction (SSI), including spatial variation of the ground motion. For structures founded on rock media, these effects are much less pronounced. The foundation media is, in effect, another structural element of the structural system and changes the natural frequencies and mode shapes. That is, the structure plus an additional foundation element may have free vibration characteristics that differ from those of the same structure on a rigid foundation and without the additional foundation element. A significant affect of soil-structure interaction is radiation of energy from the structure into the ground (radiation damping). As a result, this foundation element must represent both the stiffness and damping of the foundation media. Spatial variation of earthquake ground motion result in reduced motion at the base of a structure from that recorded by an instrument on a small pad. These reductions are due to vertical spatial variation of the ground motion (reduced motion with depth), horizontal spatial variation of the ground motion (basemat averaging effects), and wave scattering effects (modification of earthquake waves striking a rigid structure foundation).

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. Even in symmetrical structures, propagating earthquake ground waves can give rise to torsion. Hence, a minimum torsional loading should be considered in design or evaluation.

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 rein-

forcement includes adequate ties to prevent buckling or spalling. With proper design details, structures can be designed to withstand 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-is condition of the structure.

Potential damage and failure of structures, systems, and components (SSCs) due to both direct earthquake ground shaking and seismic response of adjacent SSCs must be considered. The interaction of SSCs during earthquake occurrence can produce additional damage/failure modes to be addressed during seismic design or evaluation. Examples of interaction include: (1) seismic-induced failure of a relatively unimportant SSC which falls on a SSC which is important to safety or to the mission; (2) displacements of adjacent SSCs during seismic response resulting in the adjacent SSCs pounding together; (3) displacements of adjacent SSCs during seismic response resulting in failure of connecting pipes or cables; (4) flooding and exposure to fluids from vessels or piping systems ruptured as a result of earthquake motion; and (5) effects of seismic-induced fires.

The occurrence of an earthquake affects many or all SSCs in a facility. Hence, it is possible to have multiple seismic-induced failures of SSCs. These common cause effects must be considered in design or evaluation. For example, multiple failures in a tank farm can result in loss of contents greater than that held in any single tank which in turn could overflow a retention berm and/or flood adjacent SSCs. The effects of this large quantity of tank contents on SSCs must be considered.

Earthquake ground shaking also affects building contents and nonstructural features such as windows, facades, and hanging lights. It is common for the structure to survive an earthquake without serious structural damage but to have significant and expensive damage of contents. 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 specially attached in order to accommodate

the relative seismic movement of the structure without damage. Building contents can usually be adequately 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, glove boxes, tanks, or silos 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 canisters, glove boxes, tanks or silos and adequate support of ventilation ducting, filters, and fans to prevent their loss of function during an earthquake. Long-term 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 distribution systems can also cause injuries, loss of function, or loss of confinement. Many of these items can survive strong earthquake ground shaking with adequate anchorage or restraint. 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.

E.2 Effects of Wind

In this document high winds capable of damaging SSCs are classified as 1) straight winds, 2) hurricane winds or 3) tornado winds. Straight winds generally refer to winds in thunderstorm gust fronts or mesocyclones. Winds circulating around high or low pressure systems (mesocyclones) are rotational in a global sense, but are considered straight winds in the context of this document. Tornadoes and hurricanes both have rotating winds. The diameter of the rotating winds in a small hurricane is considerably larger than the diameter of a very large tornado. However, most tornado wind diameters are large compared to the dimensions of typical buildings or structures.

Although the three types of wind are produced by distinctly different meteorological events, research has shown that their effects on SSCs are essentially the same. Wind effects from straight winds are studied in boundary layer wind tunnels. The results of wind tunnel studies are considered reliable because they have been verified by selected full-scale measurements (Reference E-1). Investigations of damage produced by straight winds also tend to support wind tunnel findings. Although the rotating nature of hurricane and tornado winds cannot be precisely duplicated in the wind tunnel, wind damage investigations suggest that the magnitudes and distribution of wind pressures on SSCs produced by hurricane and tornado winds are essentially identical to those produced by straight winds, if the relative wind direction is taken into account. Thus, the approach for determining wind pressures on SSCs proposed in this document is considered to be independent of the type of windstorm.

Measurements of hurricane and straight wind speeds are obtained from anemometer readings. Wind speeds must be cited within a consistent frame of reference. In this document the frame of reference is "fastest-mile" wind speed (average speed of one mile of air passing an anemometer) at 33 ft (10 meters) above ground in flat open terrain. Wind speeds measured relative to one frame of reference can be converted to another frame of reference through the use of wind speed profiles and relationships between averaging times.

Tornado wind speeds cannot be measured easily by conventional anemometers. Instead tornado wind speeds are estimated from appearance of damage in the storm path The Fujita Scale (F-Scale) classification is generally accepted as the standard for estimating tornado wind speeds (Reference E-2). Table E-1 lists the wind speed ranges and describes the damage associated with each category. The wind speeds associated with the Fujita Scale are considered to be peak gusts (2-3 second averaging time). The tornado hazard assessments used in this document are based on F-Scale wind speeds at 33 ft (10 meters)

above ground in flat open country. The relationship developed by Durst (Reference E-3) between wind speeds averaged over time t and mean hourly wind speed are used to convert peak gust tornado wind speeds to fastest-mile wind speeds. For wind speeds greater than 60 mph, the equivalent fastest-mile wind speed V_{fm} is given by:

$$V_{fm} = 0.958\ V_t - 11.34 \qquad \text{(E-1)}$$

where V_t is the peak gust tornado wind speed.

Table E-1 F-Scale Classification of Tornadoes Based on Damage (Ref. E-2)

(F0)	LIGHT DAMAGE 40-72 mph (peak gust wind speed) Some damage to chimneys or TV antennae; breaks branches off trees; pushes over shallow rooted trees; old trees with hollow insides break or fall; sign boards damaged.
(F1)	MODERATE DAMAGE 73-112 mph (peak gust wind speed) 73 mph is the beginning of hurricane wind speed. Peels surface off roofs; windows broken; trailer houses pushed or overturned; trees on soft ground uprooted; some trees snapped; moving autos pushed off the road.
(F2)	CONSIDERABLE DAMAGE 113-157 mph (peak gust wind speed) Roof torn off frame houses leaving strong upright wall standing; weak structure or outbuildings demolished; trailer houses demolished; railroad boxcars pushed over; large trees snapped or uprooted; light-object missiles generated; cars blow off highway; block structures and walls badly damaged.
(F3)	SEVERE DAMAGE 158-206 mph (peak gust wind speed) Roofs and some walls torn off well-constructed frame houses; some rural buildings completely demolished or flattened; trains overturned; steel framed hanger-warehouse type structures torn; cars lifted off the ground and may roll some distance; most trees in a forest uprooted, snapped, or leveled; block structures often leveled.
(F4)	DEVASTATING DAMAGE 207-260 mph (peak gust wind speed) Well-constructed frame houses leveled, leaving piles of debris, structure with weak foundation lifted, torn, and blown off some distance; trees debarked by small flying debris, sand soil eroded and gravels fly in high winds; cars thrown some distances or rolled considerable distance finally to disintegrate, large missiles generated.
(F5)	INCREDIBLE DAMAGE 261-318 mph (peak gust wind speed) Strong frame houses lifted clear off foundation and carried considerable distance to disintegrate, steel-reinforced concrete structures badly damaged, automobile-sized missiles carried a distance of 100 yards or more; trees debarked completely, incredible phenomena can occur

E.2.1 Wind Pressures

Wind pressures on structures (buildings) can be classified as external or internal. External pressures develop as air flows over and around enclosed structures. The air par-

ticles change speed and direction, which produces a variation of pressure on the external surfaces of the structure. 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 structure, except on windward walls and on steep windward roofs. 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 structure through existing openings or openings created by airborne missiles. In some cases natural porosity of the structure also allows air to flow into or out of the building. Internal pressure acts either inward or outward, depending on the location of the opening and the wind direction. If air flows into the structure 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. Openings in any wall or roof area where the external pressures are outward acting allows air to flow from inside the structure: pressure inside the structure 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 structure's surface.

With systems and components, interest focuses on net overturning or sliding forces, rather than the wind pressure distribution. The magnitude of these forces is determined by wind tunnel or full-scale tests. Also, in special cases associated with aerodynamically sensitive SSCs, vortex shedding or flutter may need to be considered in design. Typical wind sensitive SSCs include stacks, poles, cooling towers, utility bridges, and relatively light-weight structures with large smooth surfaces.

Gusts of wind produce dynamic pressures on SSCs. Gust effects depend on the gust size relative to SSC size and gust frequency relative to the natural frequency of the SSC. 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 are not negligible. The size (spatial extent) of a gust relative to the size of the SSC contributes to the magnitude of the dynamic pressure. A large gust that engulfs the entire SSC has a greater dynamic effect on the SSC than a small gust that only partially covers the SSC. In any event, wind loads may be treated as quasi-static loads by including an appropriate gust response factor in calculating the magnitude of the wind

pressure. Straight wind, hurricane or tornado gusts are not exactly the same, but errors owing to the difference in gust characteristics are believed to be relatively small for those SSCs that are not wind sensitive.

The roughness of terrain surrounding SSCs 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.

E.2.2 Additional Adverse Effects of Tornadoes

In addition to wind effects, tornadoes produce atmospheric pressure change effects and missile impacts from windborne debris (tornado-generated missiles).

Atmospheric pressure change (APC) only affects sealed structures. Natural porosity, openings or breach of the structure envelope permit the inside and outside pressures of an unsealed structure 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 (Reference E-1). SSCs that are purposely sealed will experience the net pressure difference caused by APC. APC, when present, acts outwardly 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 do not occur at the same location within the tornado vortex. The lowest APC occurs at the center of the tornado vortex, whereas the maximum wind pressure occurs at the radius of maximum wind, which ranges from 150 to 500 ft from the tornado center. The APC is approximately one-half its maximum value at the radius of maximum wind speed. With APC acting on a sealed building, internal pressure need not be considered. 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. Treatment of these effects is beyond the scope of this document.

High winds and tornadoes pick up and transport various pieces of debris, including roof gravel, pieces of sheet metal, timber planks, plastic pipes and other objects that have high surface area to weight ratios. These objects can be carried to heights up to 200 feet in strong tornadoes. Steel pipes, posts, light-weight beam sections and open web steel joists having smaller area-to-weight ratios are transported by tornado winds, but occur less frequently and normally do not reach heights above 100 ft. Automobiles, storage tanks,

and railroad cars may be rolled and tumbled by severe tornado winds. In extremely rare instances, large-diameter pipes, steel wide flange sections and utility poles are transported by very intense tornado winds. These latter missiles are so rare that practicality precludes concern except for SSCs having lower probabilistic performance goals than Performance Category 4, which are comparable to SSCs found in commercial nuclear power plants.

E.2.3 Effects on Structures, Systems, and Components

A structure as used herein is an element or collection of elements that provide support or enclosure of space, e.g. a building. The walls and roof make up the envelope of a structure. Wind pressures develop on the surfaces of a building envelope and produce loads on the support structure, which, in turn, transmits the loads to the foundation. The support structure also must carry dead, live and other environmental loads.

Element failure is quite rare. More frequently the element connections are the source of failure. A properly conceived wind-force resisting structure should not fail as a result of the failure of a single element or element connection. A multiple degree of redundancy should be provided in a ductile structure that allows redistribution of load when one element or connection of the structure is overloaded. Ductility allows the structure to undergo large deformations without sudden and catastrophic collapse. The structure also must have sufficient strength and stiffness to resist the applied loads without unacceptable deflection or collapse.

Cladding forms the surface of the structure envelope. Cladding includes the materials that cover the walls and roof of a structure. Cladding failure results in a breach of the structure envelope. A breach develops because the cladding itself fails (excessive yielding or fracture); the connections or anchorages are inadequate; or the cladding is perforated by windborne missiles. Cladding is sometimes relied upon to provide lateral support for purlins, girts or columns. Cladding may be an integral part of shear wall construction. 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 the 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 larger than overall external pressures.

Breach of structure 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 combine with the external pressures, both overall and local, to produce a worse loading condition. If the structure 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 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 unconfined toxic or radioactive materials to the environment. A breach can also lead to water damage due to rain.

Systems, consisting primarily of piping, utilities, and distribution configurations, are more susceptible to wind damage when located outdoors. Electrical lines, transformers, overhead pipe bridges, steam lines, storage tanks are examples cf wind vulnerable systems. Net wind forces are calculated for each element of the system. Channeling and shielding may be a factor in complex facilities. Windborne missiles also pose a threat to systems. Collapse or failure of less vulnerable SSCs could cause damage to more critical ones.

Components, consisting primarily of equipment such as fans, pumps, switch gear, are less vulnerable to wind than earthquake forces, but can be damaged if exposed to flying debris.

E.3 Effects of Flooding

E.3.1 Causes and Sources of Flooding and Flood Hazards

There are a number of meteorologic and hydrologic phenomena that can cause flooding at 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, the damage potential of a flood is increased if there are impact or dynamic forces, hydrostatic forces, water-borne debris, etc. Table E-2 lists the various sources or causes of flooding that can occur and the particular hazards they pose. 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.

In most cases, flood hazards are characterized in terms of the depth of flooding that occurs on site. Depth of inundation is the single most relevant measure of flood severity. However, the degree of damage that is caused by flooding depends on the nature of the hazard. For example, coastal sites experience 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 add substantially to the potential for 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.

Table E-2 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

E.3.2 Flooding Damage

In many ways, flood hazards differ significantly from other natural phenomena. As an example, it is often relatively easy to eliminate flood hazards as a potential contributor to damage at a site through 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 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 critical components required to maintain safety and 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 can 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 flooding increases as 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 hydrostatic loads. Structural failure of roof systems can occur when drains become clogged or are inadequate, and 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 structures that are not properly designed. 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 E-3 summarizes the damage to buildings and flood-protection devices that various flood hazards can cause.

Table E-3 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 confinement 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.

E.4 References

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CONCLUDING MATERIAL

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