Chapter 2
Earthquake Design and Evaluation Criteria

2.1 Introduction

This chapter describes requirements for the design or evaluation of all classes (i.e. safety class, safety significant) 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 all SSCs per the International Building Code 2000 (IBC 2000) and other codes with seismic provisions comparable to NEHRP provisions. 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 listed in Appendices B and C 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 (as applicable) 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 applicable drift limits, (acceptance criteria)
4. Ductile detailing requirements for buildings

It is important to note that the above four elements taken together comprise the earthquake 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.

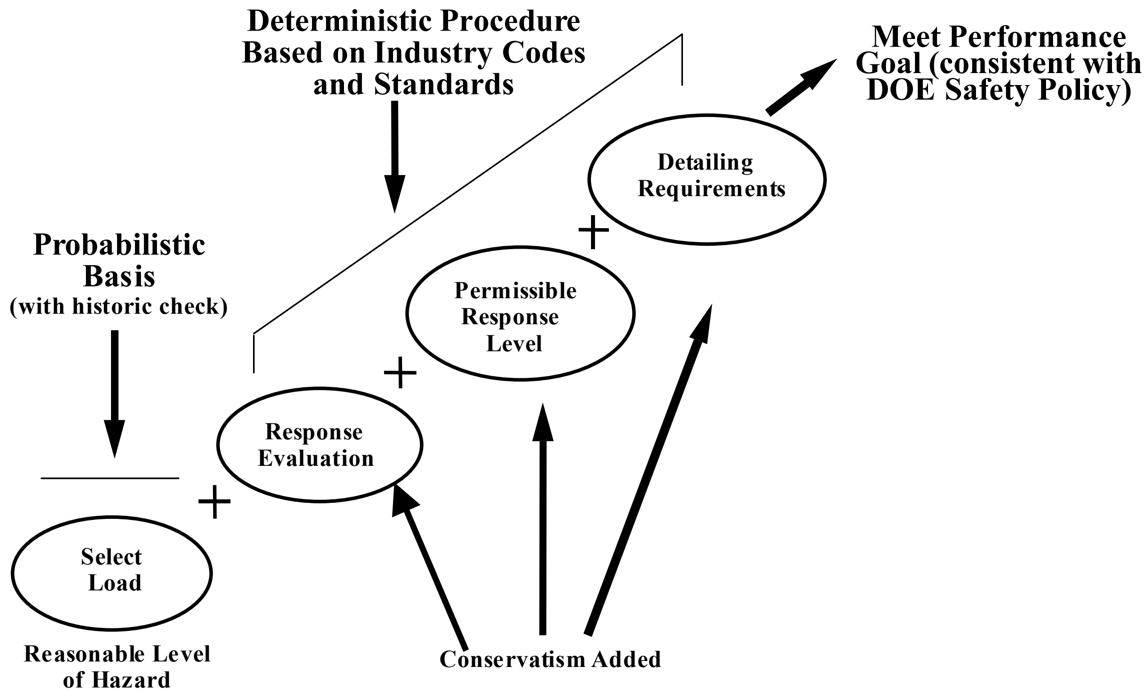


Figure 2-1. DOE-STD-1020 Combines Probabilistic and Deterministic Methods to Achieve Performance Goals

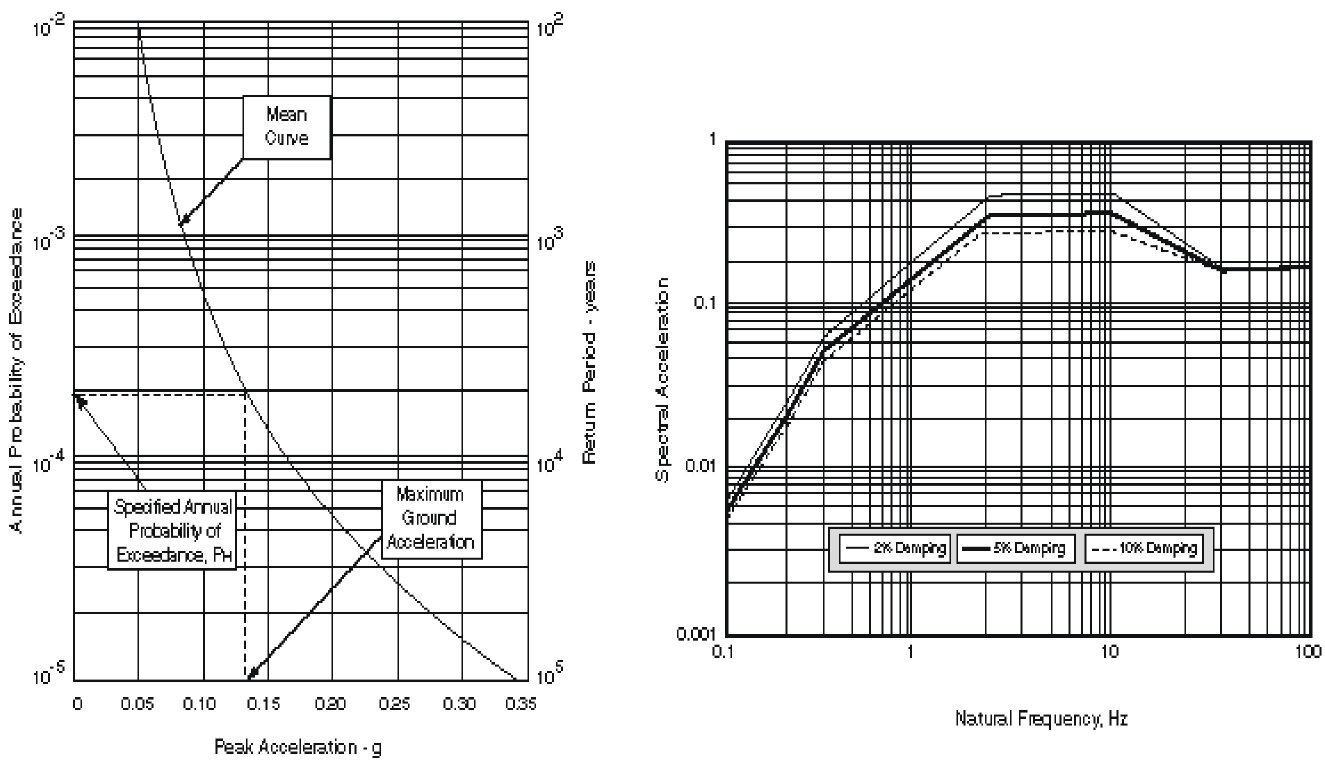
Criteria are provided for each of the four Performance Categories 1 to 4 as defined in DOE O 420.1, the accompanying Guide DOE G 420.1-2 (Ref. 1-2) and DOE-STD-1021 (Ref. 1-10). The criteria for Performance Categories 1 and 2 are similar to those from model building codes. Criteria for PC-3 are similar 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-specific design response spectrum (the Design/Evaluation Basis Earthquake, [DBE]). Either a site-specific design response spectrum 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). Such spectra are determined in accordance with DOE-STD-1023 (Ref. 2-19).

For PC-1 and PC-2 facilities IBC/USGS maps should be used for ground motion unless there are special reasons to conduct site-specific studies. However, use of any site specific results shall conform to the limits established in the IBC 2000.

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 boradened, 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	Remarks
0	No Requirements	
1	Follow IBC 2000 in its Entirety*	Use IBC 2000 Seismic Use Group I Criteria-2/3 MCE Ground Motion
2	Follow IBC 2000 in its Entirety*	Use IBC 2000 Seismic Use Group III Criteria 2/3 MCE Ground Motion with Importance Factor of 1.5
3	4×10^{-4} (1×10^{-3}) [1]	Establish DBE Per DOE-STD-1023 Analysis Per DOE-Std. 1020
4	1×10^{-4} (2×10^{-4}) [1]	Establish DBE Per DOE-STD-1023 Analysis Per DOE-Std. 1020

* Based on Maximum Considered Earthquake (MCE) Ground Motion – generally 2% Exceedance Probability in 50 years from the seismic hazard maps, modified to account for site effects. $P_H = 4 \times 10^{-4}$

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

Performance Category 2 and lower SSCs may be designed or evaluated using the approaches specified in IBC 2000 seismic provisions. Common cause effects and interaction effects per DOE- STD-1021 should be taken into account. 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.

A "dynamic analysis approach" does not 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 are used in the appropriate terms of the IBC code equation for base shear. The maximum ground acceleration is also used in the IBC code equation for seismic forces on equipment and non-structural components. Use of modern site-specific earthquake ground motion data and the IBC 2000 requirements based on NEHRP provisions (Reference 2-6) maps are considered to be preferable to the general seismic zonation maps from the previous codes and should be applied according to the guidance provided in DOE-STD-1023 (Ref. 2-22). For structures, the IBC code 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. Elastically computed seismic response is reduced by R values ranging from 1¼ to 8 as a means of accounting for inelastic energy absorption capability in the IBC code 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). For concrete structures, the design detailing provisions from the IBC 2000 (for PC 1& 2) and ACI-349 (for PC-3 & 4) which provide ductility, toughness, and redundancy, are also required such that SSCs can fully achieve potential inelastic energy absorption capability. For structures constructed of other materials follow the relevant codes and standards specified in Chapter 1. Normally, relative seismic anchor motion (SAM) is not considered explicitly by model building code seismic provisions. However, SAM should be considered for PC-3 and PC-4 SSCs.

Executive Order 12699 (Ref. 1-7) establishes the minimum seismic requirements for new Federal buildings. NEHRP updates the provisions required to meet these requirements every 3 years. The Interagency Committee on Seismic Safety in Construction (ICSSC) compares model building codes with the NEHRP provisions. Designers must consider the NEHRP provisions and ICSSC comparisons to ensure the use of the proper model building code in their design and evaluation. Currently the IBC 2000 and ASCE 7-98 meet the requirements of the NEHRP provisions. While using the IBC 2000 or successor documents, designers must consider the Seismic Use Group and Seismic Design Category.

The seismic provisions in the IBC 2000 have been specified for PC-1 and PC-2 because it is the only current model code meeting NEHRP provisions. The Interagency Committee on Seismic Safety in Construction has concluded that the following seismic provisions are equivalent for a given DBE the and latest NEHRP provisions:

1. International Building Code 2000
2. ASCE 7-98

The seismic provisions in the Uniform Building Code (UBC) 1997 have not been found to be equivalent to the 1997 NEHRP provisions. However, this code may be used on case by case basis as long as the intent of the seismic provisions in this standard are met (Based on IBC 2000/1997 NEHRP provisions). The seismic design maps associated with 1997 UBC are generally out of date (over 20 years old) and at a minimum the MCE ground motion maps in IBC 2000 should be consulted to ensure that DBE ground motion are adequate and conservative. Other model building codes may be followed provided site-specific ground motion data is incorporated into the development of the earthquake loading in a manner consistent with DOE-STD-1023, and the NEHRP provisions.

For PC-3 and PC-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 the elastic seismic demand on SSCs. Inelastic energy absorption capability is allowed by permitting limited inelastic behavior. By these provisions, the inelastic energy absorption capacity of structures is accounted for by the parameter, $F\mu$. However, strength and ductile detailing for the entire load path should be assured. Elastically computed seismic response is reduced by $F\mu$ values ranging from 1 to 3 as a means of accounting for inelastic energy absorption capability. The same $F\mu$ 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. Alternatively for PC-3 and PC-4 SSCs, non-linear static (push-over analysis) may be adequate, and in extreme cases a non-linear dynamic analysis may be used, if justified. $F\mu$ factors should not be used when performing non linear analysis. For concrete structures, the design detailing provisions from the ACI 349 which provide ductility, toughness, and redundancy, are also required such that SSCs can fully achieve potential inelastic energy absorption capability (for other materials, follow relevant codes listed in Chapter 1). Also, explicit consideration of relative seismic anchor motion (SAM) effects is required for PC-3 and PC-4 SSCs.

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.

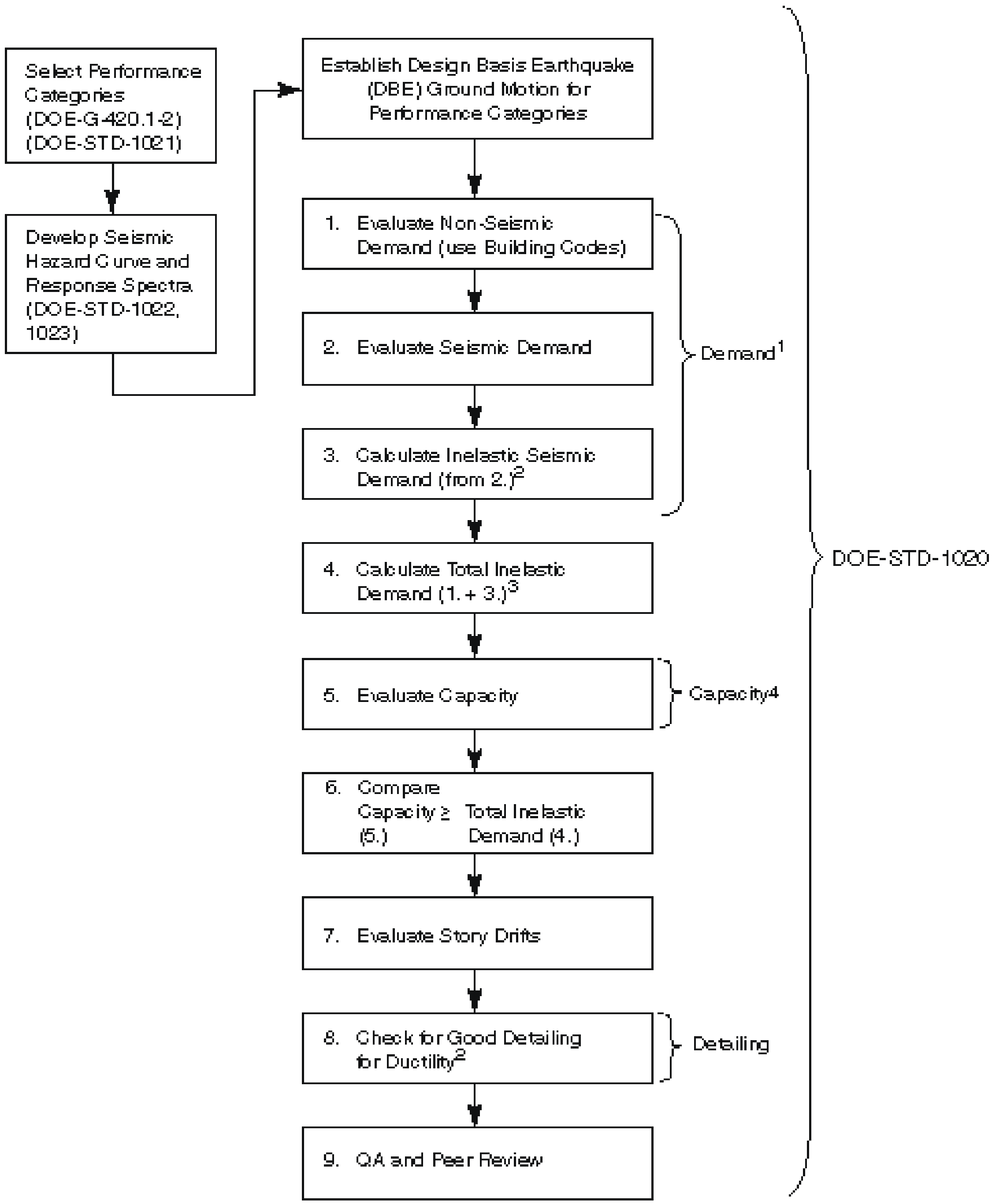
2.3 Seismic Design and Evaluation of Structures, Systems, and Components

- Select Performance Categories of structure, system, or component based on DOE G 420.1-2 (Ref. 1-2) and DOE-STD-1021 (Ref. 1-10).

- For sites with PC-3 or PC-4 SSCs, obtain or develop a seismic hazard curve and design response spectra in accordance with DOE-STD-1023 (Ref. 2-19) for all performance categories based on site characterization discussed in DOE-STD-1022 (Ref. 1-14).

- 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 and PC-2 SSCs, and no site-specific seismic hazard curve, obtain seismic coefficients from model building codes which are based on national seismic hazard maps prepared by the United States Geological Survey. If available, site specific data can be used for these categories but with limitations imposed in the IBC 2000.



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. For load combinations, use IBC 2000 for PC-1 & PC-2 SSCs.
4. See Section C.5 for further discussion.

Figure 2-3. DOE Seismic Design and Evaluation Procedure

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

Seismic design or evaluation of PC-1 and PC-2 SSCs is based on model building code seismic provisions. In these criteria, the current version of the International Building Code shall be followed. Alternatively, the other equivalent model building codes may be used as discussed in Section 2.2. All of the IBC 2000 seismic provisions shall be followed for Performance Category 1 and PC-2 SSCs Load combinations to be used for PC-1 and PC-2 will be based on the provisions in the IBC 2000. Use of site specific data will be limited per provisions of IBC 2000. Post-Northridge earthquake SAC recommendations should be taken into consideration for steel structures.

The steps in the procedure for PC-1 and PC-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 IBC 2000.

  b. In either case, the total base shear is given in the IBC 2000 where the parameters are evaluated as follows:

     - Use Seismic Use Group I for design of PC-1 SSCs
     - Use Seismic Use Group III for design of PC-2 SSCs which essentially results in a multiplier of 1.5 to forces for PC-1

The seismic design categories per IBC 2000 must also be taken into consideration.

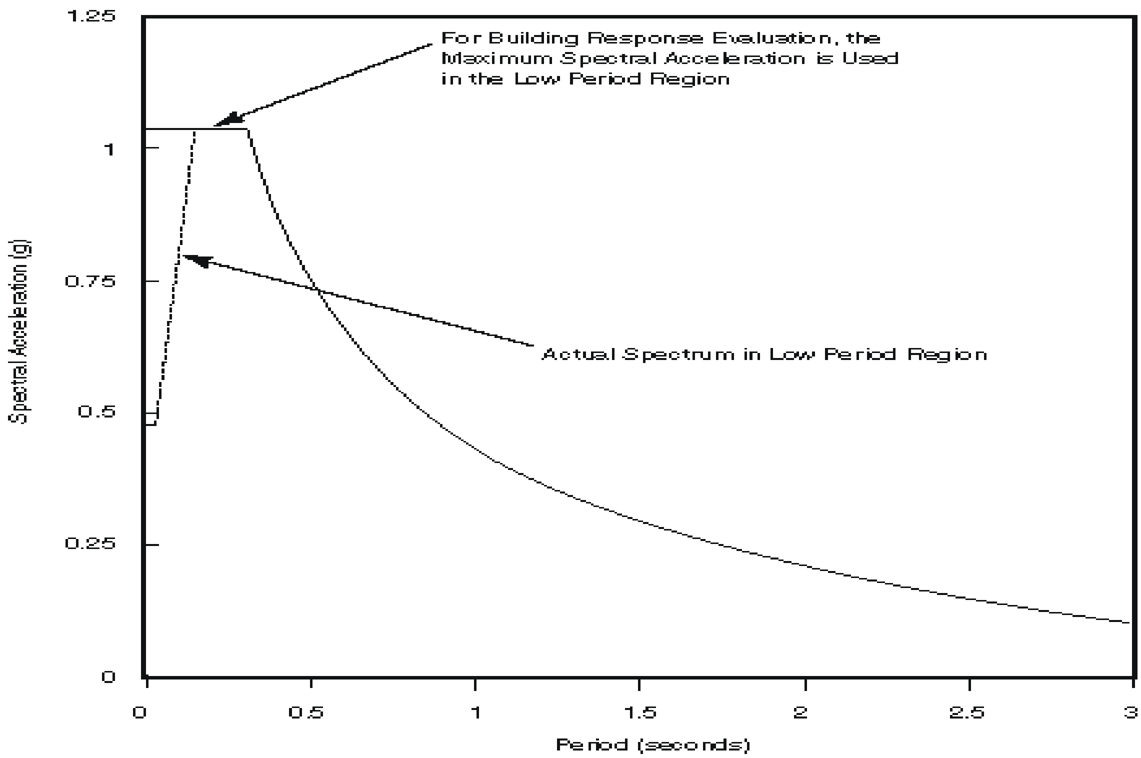


Figure 2-4. Example Design/Evaluation Earthquake Ground Motion Response Spectrum

For systems and components, seismic design forces are accounted as per the IBC 2000 provisions.

If a recent site-specific seismic hazard assessment is available, it can be used subject to limitations imposed in the IBC 2000. For evaluation of SSCs using site specific hazard analysis, the design shall be based on 5% critical damping as recommended by the IBC 2000. Final earthquake loads are subject to approval by DOE.

For structures, response modification coefficients, R, and for systems and components R_P are given in IBC 2000

- 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 applicable load factors for allowable stress design).

- Evaluate capacities of SSCs, C_C, from code ultimate values when strength design is used (e.g., IBC 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 population values statistically adjusted for sample size, 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.

- 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 the limitations in IBC 2000.

- Elements of the facility shall be checked to assure that all detailing requirements IBC 2000 provisions are met keeping into consideration the seismic design category of the building.

- 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 PC-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 2-2. 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 $F\mu > 1$, the maximum spectral acceleration should be used for fundamental periods lower than the period at which the maximum spectral amplification occurs (See Figure 2-4). For higher modes, the actual spectral accelerations should be used.

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

$$D_{SI} = SF \frac{D_S}{F_\mu} \tag{2-1}$$

where: $F\mu$ = Inelastic energy absorption factor from Table 2-3 for the appropriate structural system and elements having adequate ductile detailing

SF = Scale factor related to Performance Category
= 1.25 for PC-4
= 0.9 for PC-3

Variable scale factors, based on the slope of site-specific hazard curves are discussed in Appendix C, to result in improved achievement of performance goals. Site specific scale factors for low seismicity sites should be quantified to ensure that use of
S.F = 0.9 is adequately conservative. 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 2-3 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).

$$D_{TI} = D_{NS} + D_{SI} \tag{2-2}$$

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

Reinforced Concrete

Use IBC 2000, ACI 318 & ACI-349

Steel

Use IBC 2000 and AISC
- 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.:

$$C_C > D_{TI} \qquad (2\text{-}3)$$

- 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 $F\mu$ 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 flexure. 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 $F\mu$ given in Table 2-3 are upper limit values. For concrete structures, good design detailing practice and consistency with recent ACI 349 provisions should be followed. (For other materials, use relevant codes per Chapter 1). Existing facilities may not be consistent with the recent ACI 349 provisions, and, if not, must be assigned a reduced value of $F\mu$.

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

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

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

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 2-2 at three different response levels as a function of D_T/C_C.

D_T is the elastically computed total demand,

$$D_T = D_{NS} + D_S \qquad (2\text{-}4)$$

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 (See 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 2-2 Specified Damping Values

Type of Component	Damping (% of critical) 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 2-3 Inelastic Energy Absorption Factors, F_{μ}

Structural System	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 , ACI 349 and other relevant codes along with reasonably uniform inelastic behavior. Otherwise, lower values should be used.
2. $F\mu$ 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 $F\mu$ 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 IBC 2000 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 $F\mu$ of unity, whichever is less.
5. $F\mu$ for chevron, V, and K bracing is 1.5. K bracing requires special consideration for any building if acceleration is 0.25g or more.

2.4 Additional Requirements

2.4.1 Equipment and Distribution Systems

For PC-1 and PC-2 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, given by the IBC provisions (Ref. 2-7). For PC-2 equipment expected to remain functional during or after earthquake, testing or experience based data for such equipment shall be an additional qualification requirement. For PC-3 and PC-4 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 IBC provisions for PC-1 and PC- 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. All the provisions of the IBC () shall be followed for PC-1 & PC-2 SSCs.

The lateral force determined using IBC 2000 shall be distributed in proportion to the mass distribution of the element or component. Forces determined 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.

For PC-3 and PC-4 subsystems and components, support excitation shall be calculated 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 multiple-supported components with different seismic inputs, support displacements can be obtained either from the structural response calculations of the supporting structure or from spectral displacements 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:

RRS ≥ (1.1)(In-Structure Spectra) for PC-1 and PC-2 (2-5)

RRS ≥ (1.4SF)(In-Structure Spectra) for PC-3 and PC-4 (2-6)

where SF is the seismic scale factor from Equation 2- 1.

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. Currently, use of experience data is permitted for existing facilities and for the items specified in the three references, (Ref 2-5), (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 IBC Chapter 19 provisions (Ref. 2-7) for PC-1 and
PC-2 SSCs and on ACl 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 References 2-5 and 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 References 2-5 and 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 References 2-5 and 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). In addition, guidelines for upgrading and strengthening equipment are presented in Ref. 2-20. Also, guidance for evaluation of existing equipment by experience data is provided in References 2-5 and 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 current design criteria.

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. 1-8), as discussed in the DOE Guide 420.1-2 are to be implemented. The requirements of ICSSC RP6 are minimum requirements to be met for existing buildings, especially the mitigation requirements triggered by section 2 of this standard. The line organization may define “Exceptionally High Risk buildings” to meet their safety and mission needs. Provisions in FEMA 310 and FEMA 356 should be taken into account while evaluating and upgrading existing buildings. Specific provisions may have to be modified to meet criteria for PC-3 and PC-4 in this standard.

DOE O 420.1 requires that the sites should prepare upgrade plans for buildings that are deemed deficient to meet NPH equirements. Some of these deficiencies may have been discovered during the facility safety reviews and or during implementation of Executive Order 12941 (Reference 1-8). One of the prioritization schemes to upgrade such deficient building is given in Table 2-4, although sites may choose their own schemes.

Table 2-4 Suggested Priortization Scheme*

Description	Priority Group	Function	Model Building Groups	Seismicity	Occupancy Groups
Buildings which pose the greatest risk to life or loss of essential function	1	All	Extremely Poor Buildings	Very High Hazard	High Occupancy
	2	All	Extremely Poor Buildings	Very High Hazard	M/L Occupancy
	3	All	Extremely Poor Buildings	High Hazard	High Occupancy
	4	All	Very Poor Buildings	High Hazard	High Occupancy
	5	All	Very Poor Buildings	High Hazard	High Occupancy
	6	All	Poor Buildings	Very High Hazard	High Occupancy
	7	All	Poor Buildings	High Hazard	High Occupancy
	8	Essential	Very Poor Buildings	Very High Hazard	M/L Occupancy
	9	Essential	Poor Buildings	Very High Hazard	M/L Occupancy
	10	Essential	All Building	Moderate Hazard	High Occupancy
	11	Non-essential	Extremely Poor Buildings	Moderate Hazard	High Occupancy
	12	Essential	All Buildings	High Hazard	M/L Occupancy
Decreasing Risk	13	Non-essential	Extremely Poor Buildings	High Hazard	M/L Occupancy
	14	Non-essential	Very Poor Buildings	Moderate Hazard	High Occupancy
	15	Non-essential	Poor Buildings	Moderate Hazard	High Occupancy
	16	Essential	All Buildings	Moderate Hazard	M/L Occupancy
	17	Non-essential	Extremely Poor Buildings	Moderate Hazard	M/L Occupancy
	18	Non-essential	Very Poor Buildings	Very High Hazard	M/L Occupancy
	19	Non-essential	Very Poor Buildings	High Hazard	M/L Occupancy
	20	Non-essential	Poor Buildings	Very High Hazard	M/L Occupancy
	21	Non-essential	Poor Buildings	High Hazard	M/L Occupancy
	22	Non-essential	Very Poor Buildings	Moderate Hazard	M/L Occupancy
Buildings which pose the least risk to life or loss of essential function	23	Non-essential	Poor Buildings	Moderate Hazard	M/L Occupancy

* SOURCE: DRAFT FEMA REPORT TO CONGRESS ON E.O.12941

Definitions:
Seismicity
Very High: Area where earthquakes could happen in the near future, say the next 30 years.
High: Area where damaging earthquakes could happen within the life of a typical building say 100 years.
Moderate: Area where earthquakes could happen.
Low: Area where earthquakes are not expected to happen at all.

Occupancy Groups

High: Greater than 200 People
Moderate/Low: Up to 200 people

Definitions above are based on 24 hours average occupancies.

Model Building Groups

The model building types listed in Table 2-4 are defined in FEMA 178, (Ref. 2-22) NEHRP Handbook for the Seismic Evaluation of Existing Buildings.

Extremely Poor:	Concrete Moment Frame, Precast/Tilt-up Concrete Walls with Lightweight Flexible Diaphragms, Precast Concrete Frames with Concrete Shear Walls, Unreinforced Masonry Buildings.
Very Poor:	Steel Braced Frame, Steel Frame with Infill Shear Walls, Concrete Shear Walls, Concrete Frame with infill Shear Walls, Reinforced Masonry Bearing Walls with Precast Concrete Diaphragms. Other type Unknown type.
Poor:	Wood, Light Frame, Wood, Commercial and Industrial, Steel Moment Frame, Steel Light Frame, Steel Frame with Concrete Shear Walls, Reinforced Masonry Bearing Walls with Wood or Metal Deck Diaphragms.
Essential Buildings:	Buildings that, in the judgement of the owning agency require a level of seismic resistance higher than life safety in order to support earthquake response, critical functions, hazardous materials or extremely valuable contents.

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:

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

where SF is the appropriate seismic scale factor from Equation 2-1.

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 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 1.

Table 2-5 Summary of Earthquake Evaluation Provisions

	Performance Category (PC)			
	1	2	3	4
Hazard Exceedance Probability, P_H	(MCE) G.M.[2]	(MCE) G. M.[2]	$4x10^{-4}$ ($1x10^{-3}$)[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	IBC Force equation for equipment and non-structural elements (or more rigorous approach)		Dynamic analysis using in-structure response spectra (Damping from Table 2-2)	
Seismic Use group	Seismic Use Group I	Seismic Use Group III	Not used	
Load Factors	Code specified load factors appropriate for structural material		Load factors of unity	
Scale Factors	Not Used		SF = 0.9	SF = 1.25
Inelastic Energy Absorption Ratios For Structures	Accounted for by R in IBC 2000		$F\mu$ from Table 2-3 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 the IBC 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 IBC requirements from PC-2 to nuclear power plant requirements for PC-4)		

[1]For sites such as LLNL, SNL-Livermore, SLAC, LBNL, & ETEC which are near tectonic plate boundaries.
[2]MCE GM = Maximum considered earthquake ground motion (generally, $P_H = 4x10^{-4}$) - for Seismic Use Group I (PC-1) use 2/3 MCE, for Seismic Group III (PC-2) use 2/3 MCE with importance Factor of 1.5.

2.6 References

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2-20 Federal Emergency Management Agency, *Pre-standard and Commentary for the Seismic Rehabilitation of Buildings*, FEMA 356, November 2000

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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 to assure safe and reliable performance of DOE facilities during future potential earthquakes. It is to be noted that these ar merely target performance goals which need not be proven mathematically or by probabilistic risk assessments. Design of structures, systems, and components to withst and 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 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 G 420.1-2 (Ref. C-67) and DOE-STD-1021-93 (Ref. C-26) in that hierarchial order. For Performance Category 3, 4 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) 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	Onset of SSC(1) damage to the extent that occupants are endangered
2.	Occupant Safety, Continued Operation with Minimum Interruption	SSC damage to the extent that the component cannot perform its function
3.	Occupant Safety, Continued Operation, Hazard Confinement	10^{-4} of SSC damage to the extent that the component cannot perform its function
4.	Occupant Safety, Continued Operation, Confidence of Hazard Confinement	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.

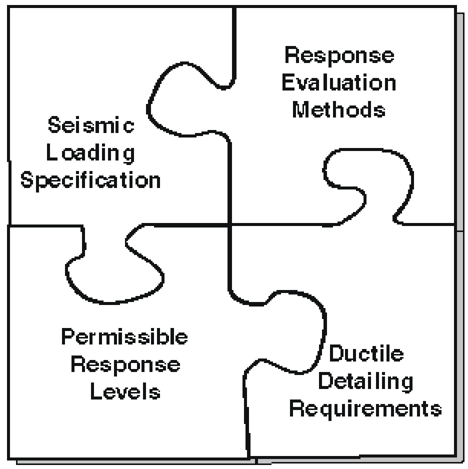

Figure C-1 Consistent Specification of All Seismic Design/Evaluation Criteria Elements

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

C.2.1 Overall Approach for DOE Seismic Criteria

Historical Perspective Since Early Development Using Uniform Building Code 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 PC-1) and for nuclear power plants (similar to DOE PC- 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.

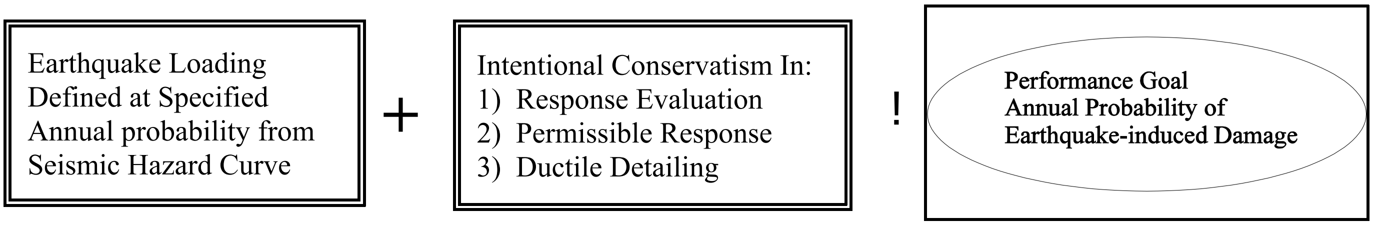

Figure C-2 Performance Goal Achievement

The performance goals for PC-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 PC-3 SSC's are consistent with DOE essential facilities and Pu handling facilities. The performance 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 conceptual development, 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:

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

Current Status

This concept enunciated above has been carried forward after issuance of IBC 2000. The performance goals achieved with IBC 2000 criteria are better than indicated Appendix B and for intent of this standard are deemed to meet the target performance goals.

In any case, the performance goals given in Appendix B and C for Performance Categories 1 and 2 are for historical purposes from the days of the 1994 Uniform Building

Code and for all intent and purposes the exact numerical values have no practical significance. The numerical values for PC 1 and PC 2 are no longer exact for the seismic provisions of the IBC 2000 which primarily intends to provide uniform margin of collapse for PC 1 SSCs throughout the United States.

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) Annual seismic hazard exceedance of 4×10^{-4} (generally) based on IBC2000 for PC-1 and PC-2, and PC-3 but 1×10^{-4} for PC-4.

2) gradual reduction in hazard annual exceedance probability of other natural phenomena hazards.

3) 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	**	*	
2	**	*	
3	$1x10^{-4}$	$4x10^{-4}$* $(1x10^{-3})^1$	4 $(10)^1$
4	$1x10^{-5}$	$1x10^{-4}$ $(2x10^{-4})^1$	10 $(20)^1$

* The seismic exceedence probability is based on USGS maps generated in 1997 (and included in IBC 2000) for 2% exceedence probabiity in 50 years. $P_H = 4x10^{-4}$ (Generally). Supplement by deterministic ground motions near very active faults.

** The design methodology of the IBC 2000 for Seismic Use Groups I and III achieves approximately performance goals of PC-1 & PC-2 respectively though it does not meet the relationship shown in equation C-1 for the seismic provisions.

1 For sites such as LLNL, SNL-Livermore, SLAC, LBNL, 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 implementation document that invokes these criteria. As shown in Table C-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/multipliers.
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 factors have been retained for the evaluation of PC-1 and PC-2 SSCs because the IBC approach (which includes these factors) is followed for these categories. These 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 PC-3 and PC-4. Material strengths and structural capacities specified for Performance Category 3 and higher SSCs correspond to ultimate strength code-type provisions (i.e., ACI 318-99 for reinforced concrete, LRFD, or AISC Chapter N for steel). Material strengths and structural capacities specified for PC-1 and PC-2 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 achieved if considerations of effective peak ground motion, soil-structure interaction are introduced, 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 design forces are 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:

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

where SF is the appropriate seismic scale factor (SF is 0.9 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_R, 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 are the basis of the seismic design and evaluation criteria presented in this standard. For PC 3 and PC 4, target performance goals, P_F, of 1×10^{-4} and 1×10^{-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.

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

$$\boxed{H(a) = Ka^{-k_H}} \qquad \text{(C-3)}$$

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. A_R is related to k_H by:

$$\boxed{k_H = \frac{1}{\log(A_R)}} \qquad \text{(C-4)}$$

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^{-3} to 10^{-5}. 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:

$$\boxed{R_R = \left(C_{50} / DBE\right)^{k_H} \; e^{-\frac{1}{2}\left(k_H\beta\right)^2}} \qquad \text{(C-5)}$$

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:

$$\boxed{R_R = \left(1.5SF\right)^{k_H} \; e^{\left[1.282\,k_H\beta - \frac{1}{2}\left(k_H\beta\right)^2\right]}} \qquad \text{(C-6)}$$

Equation (C-6) demonstrates the risk reduction ratio achieved by DOE seismic criteria as a function of hazard curve slope, uncertainty, and seismic scale factor, SF. Note from Table C-3 that for PC-4 (not near tectonic plate boundaries), the hazard probability is $1\text{x}10^{-4}$ and the performance goal is $1\text{x}10^{-5}$ such that the target risk reduction ratio, R_R is 10 and for PC-3, the hazard probability is $4\text{x}10^{-4}$ and the performance goal is $1\text{x}10^{-4}$ such that the target risk reduction ratio, R_R is 4. The actual risk reduction ratios from Equation (C-6) versus slope coefficient A_R are plotted in Reference C-20 for Performance Categories 3 and 4, respectively. In these figures, SF of 0.9 is used for PC 3 and SF of 1.25 is used for PC 4 and the range of   from 0.3 to 0.6 has been considered. For the hazard curves considered by DOE-STD-1023-92 (Ref. C-13), A_R 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_R 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.

Figures in Reference C-20 demonstrate that for SF= 0.9 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 $2\text{x}10^{-4}$ to $5\text{x}10^{-5}$. 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 4 for PC 3 and sites not near tectonic plate boundaries. Figures in Reference C-20 demonstrate 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 $5x10^{-6}$. 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 Reference C-20 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 Figures in Reference C-20 combined with Equation C-6 improves the R_R values compared to target values as shown in Figures in Reference C-20 for PC 3 (R_R = 4) and PC 4 (R_R = 10), respectively. Figures in Reference C-20 demonstrate that when the variable scale factors are used, risk reduction factors achieved are within about 10% of the target values of 4 and 10, respectively. As a result, target performance goals would be met within about the same 10%.

It is to be noted that the information in Ref. C-20 may need to be adjusted to new P_H value of $4x10^{-4}$ for PC-3 SSCs, with $R_R = 4$. The variable scale Factor is altered from that in Ref. C-20 and becomes, SF = maxium (0.9, $.6A_R{}^{0.4}$). If the variable scale factor is significantly larger, it should be used instead of 0.9 and 1.25 for PC-3 and PC-4 respectively. This is particularly significant at low seismicity sites.

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 in Reference C-20 demonstrate that larger risk reduction ratios are achieved than the target levels of 4 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.

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-3. 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-3 anchored to this maximum ground acceleration. Note that the three spectra presented in Figure C-3 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-3 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 Guide (Ref. C-27 and C-67), it is not required that a site-specific probabilistic seismic hazard assessment be conducted if the site includes only PC-1 and PC-2 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 those determined in accordance with the IBC (Ref. C-28). Design/evaluation earthquake ground motion determined from a recent site-specific probabilistic seismic hazard assessment is considered to be preferable to the IBC 2000 but cannot be lower than limitations in the IBC 2000.

For design or evaluation of SSCs in Performance Category 3 and higher, a modern site-specific seismic hazard assessment shall be performed to provide the basis for DBE ground motion levels and response spectra (See DOE-STD-1023). DOE Order 420.1 and the associated NPH 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.

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.

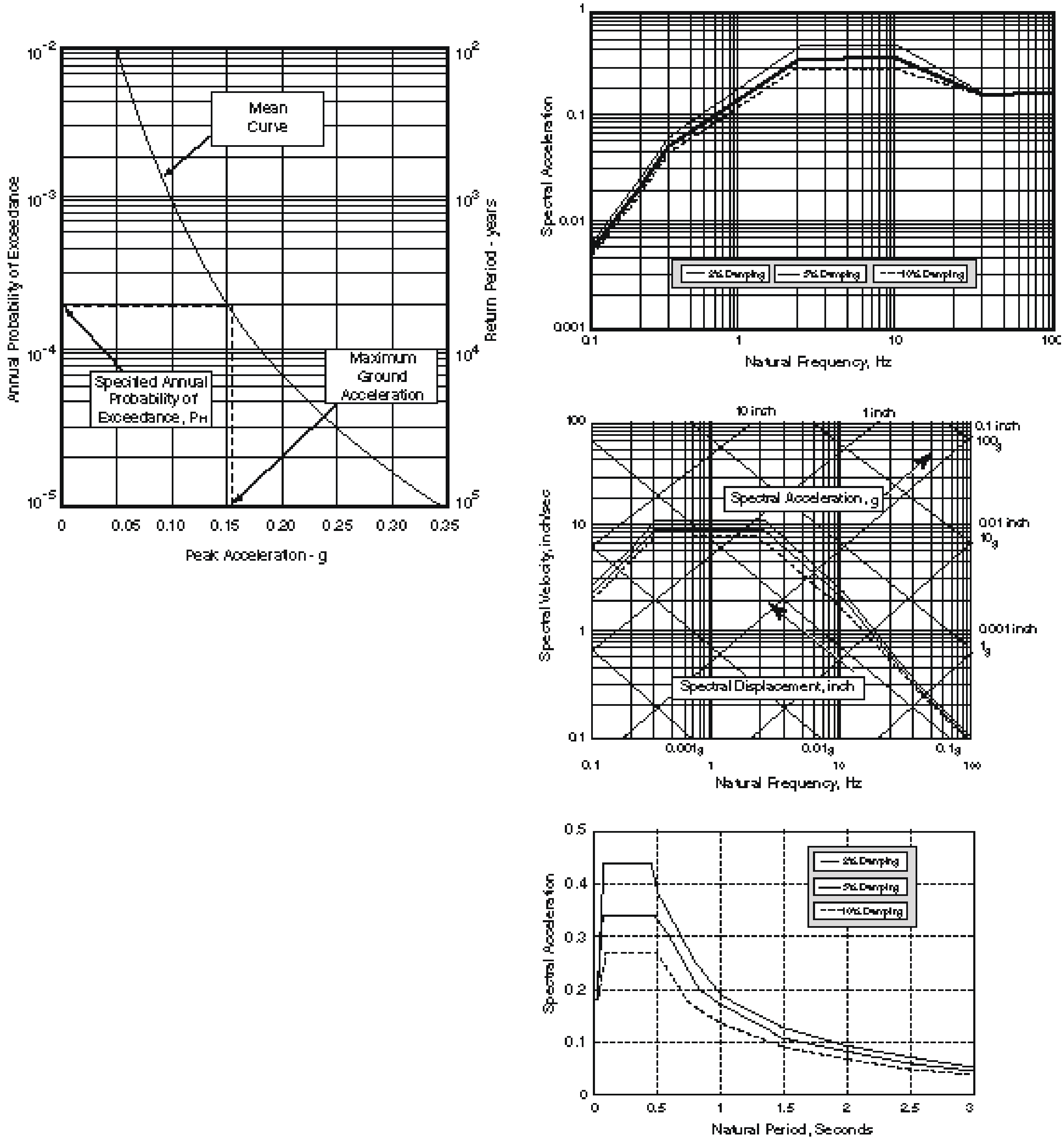

Figure C-3 Earthquake Input Excitation is Defined by Maximum Ground Acceleration Anchoring Site-Specific Response Spectra

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 20 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^{-3}. 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 $1x10^{-2}$ as the annual exceedance level for their design seismic hazard definition. The Department of Defense (DOD) tri-services seismic design provisions for essential buildings (Ref. 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 $1x10^{-3}$ annual exceedance frequency. Recently the IBC 2000 has adopted use of USGS maps for 2% exceedence probability in 50 years based on NEHRP 1997 provisions. These are being incorporated in this standard for PC-1 and PC-2 facilities.

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- 4. 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-4 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. (See also U.S. NRC Regulatory Guide 1.165)

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 PC-1 to PC-4 SSCs. The conservatism associated with Performance Categories 1 and 2 should be consistent with that contained in the IBC (Ref. C-28), or NEHRP (Ref. C-68) 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. In general for majority of DOE sites, the criteria contained herein follow the philosophy of a gradual increase in the conservatism of the evaluation procedures and acceptance criteria as one goes from Performance Category 1 to Performance Category 4.

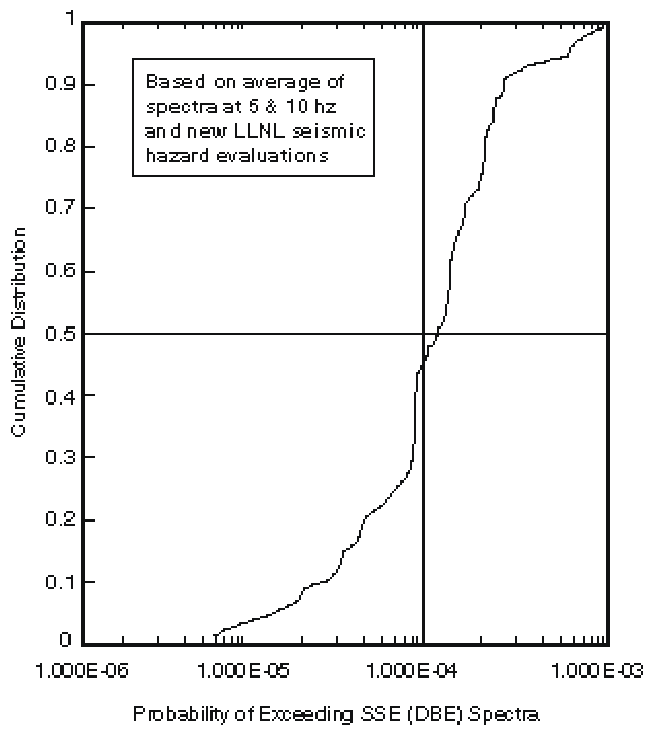

Figure C-4 Probability of Exceeding SSE Response Spectra

C.3.2 Earthquake Ground Motion Response Spectra

Design/evaluation Basis Earthquake (DBE) response spectra generally have the shape shown in Figure C-3. 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_H, 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. (See DOE-STD-1023)

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 IBC 2000 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

For PC-1and PC-2 SSCs, earthquake loading is evaluated from the base shear equation in accordance with IBC 2000 seismic provisions. IBC 2000 includes a typical design/evaluation spectra, which is plotted showing spectral acceleration as a function of natural period.

In the seismic design and evaluation criteria presented in Chapter 2, for PC-3 and PC-4 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-5. 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 $F\mu$ 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 or inelastic energy absorption factors, $F\mu$; (2) to account for the fact that stiff structures may not be as stiff as idealized in dynamic models; and (3) earthquakes in the eastern U.S. may have amplification extending to lower periods or higher frequencies than standard median design response spectra. Constant factors permit the elastically computed demand to exceed the capacity the same amount at all periods. Studies of inelastic response spectra such as those by Riddell and Newmark (Ref. 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 spectra 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.

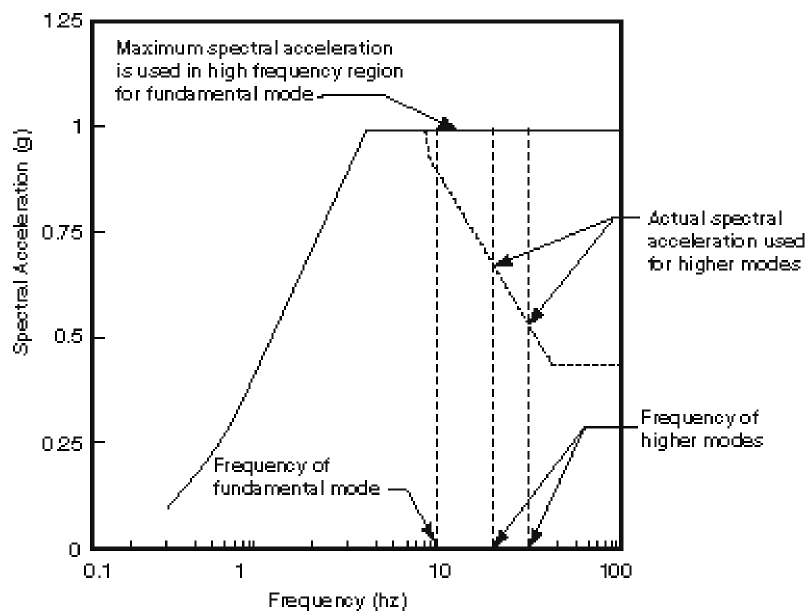

Figure C-5 Spectral Acceleration in the High Frequency Region for an Example Design/Evaluation Ground Response Spectrum

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 International Building Code, IBC 2000 provisions (Ref. C-28) 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-6. For Performance Category 2 and lower SSCs, these seismic design and evaluation criteria employ the IBC 2000 seismic provisions with the

exception that site-specific information may be used if appropriate 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 defined in IBC 2000 are used in the appropriate terms of the IBC 2000 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 IBC 2000 with limitations imposed in the IBC 2000 Code. IBC 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. Elastically computed seismic response is reduced by R values ranging from 1¼ to 8 as a means of accounting for inelastic energy absorption capability in the IBC 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.

DOE-STD-1020 Natural Phenomena Hazards Design & Evaluation Criteria for DOE Facilities 2002	• Specified Hazard Probability of Exceedance to Get Peak Acceleration and Response Spectrum • Deterministic Evaluation of Response and Capacity with Controlled Level of Conservatism

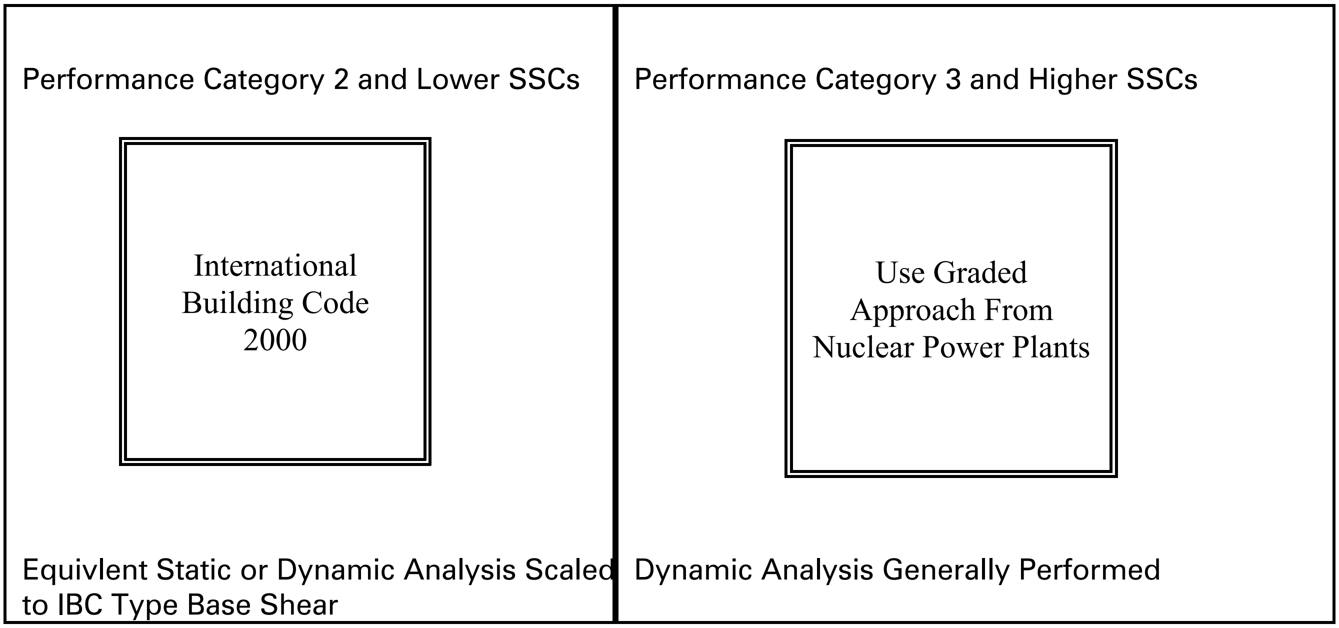

Figure C-6 Earthquake Provision Basic Approach

The International Building Code (IBC 2000) has been followed for Performance Categories 1 and 2 because it is only current model building code meeting NEHRP provisions . The Interagency Committee on Seismic Safety in Construction (ICSSC) has concluded that the following seismic provisions are substantially equivalent:

1) International Building Code, IBC 2000 (Ref. C-28)
2) 1997 NEHRP Recommended Provisions (Ref. C-68)
3) ASCE 7-98

The 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 IBC 2000.

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-7). 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, $F\mu$. Elastically computed seismic response is reduced by $F\mu$ values ranging from 1 to 3 as a means of accounting for inelastic energy absorption capability for more hazardous facilities. $F\mu$ values are much lower than R values increasing the risk reduction ratio, R_R. By these provisions, only the element forces due to earthquake loading are reduced by $F\mu$. 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 $F\mu$ 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 ACI 349, 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- 4 General Description of Earthquake Provisions

Performance Category 2 and Lower SSCs	Performance Category 3 and Higher SSCs
Use IBC provisions	- 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, $F\mu$ 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 rigorous 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 IBC provisions (Reference C-28) 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 IBC defines the distribution of lateral forces of various floor levels. 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 Ref. C-16) 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-halfspace 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
10.00	0.90
25.00	0.80

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

foundations consisting of individual column or wall footings, consideration of relative displacement between footings may also be required.

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 (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 halfspace 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 halfspace 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 4-98) 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 Sections 3.1.5.3 and 3.1.5.4 of ASCE 4-98 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 4-98 Section 3.3.1.7, is that the soil shear modulus shall be varied between the best estimate value times , and the best estimate value divided by $(1+C_v)$, where C_v is a coefficient of variation. In general, a $(1+C_v)$ 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 design or evaluation seismic response analyses of Performance Category 3 and higher SSCs. Post-yielding damping values are judged

to be appropriate because facilities designed by these criteria are intended to reach strains beyond yield level if subjected to the design/evaluation level earthquake ground motion, and such damping levels are consistent with other seismic analysis provisions based on Reference C-5.

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-7 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.

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 $F\mu$ in these criteria. Inelastic energy absorption factors have the advantage over spectral reduction factors in that different values may be specified for individual elements of the facility instead of a single spectral reduction factor for the entire lateral force resisting system. As a result, critical elements such as columns or connections can be easily designed for larger forces by specifying a smaller inelastic 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 by the IBC provisions. R is more like a spectral reduction factor in that it is applied to the entire lateral force resisting system. There are special IBC 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. The IBC provisions are followed for Performance Category 2 and lower SSCs by these criteria.

Reduction coefficients, R, to be used for evaluation of PC-1 and PC-2 are as per IBC 2000. Recommended inelastic energy absorption factors, $F\mu$, for PC-3 and PC-4 SSCs are presented in Chapter 2 for various structural systems. The $F\mu$ 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 $F\mu$.

The response modification factor coefficients, R, by the IBC approach and inelastic energy absorption factors, $F\mu$, by the DOD approach differ in the procedures that define permissible inelastic response under extreme earthquake loading. By the IBC approach, only the element forces due to earthquake loads are reduced by the reduction coefficient, R, 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, 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 $F\mu$ in a manner more similar to the IBC 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 IBC and DOD provisions. By the IBC approach, seismic axial forces and moments are both reduced by R, 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 PC-3 and PC-4 SSCs.

PC-3 and PC-4 SSCs can be evaluated by elastic dynamic analyses. However, limited inelastic behavior is permitted by utilizing inelastic energy absorption factors, $F\mu$. The inelastic energy absorption factor, $F\mu$ 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 PC-3 and PC-4 as compared to the larger R factors for PC-1 and PC-2, the margin of safety for that category is effectively increased (i.e., the risk reduction ratio, R_R, is increased), and the probability of damage is reduced in accordance with the performance goals. Note that the $F\mu$ values are employed with acceptance criteria based on ultimate stress limits with unity load factors while the R 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_S may be divided by an inelastic energy absorption factor $F\mu$ to obtain an inelastic seismic demand D_{SI}. This inelastic energy absorption factor $F\mu$ should be defined by:

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

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 $F\mu$.

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 $F\mu$ values is provided in Chapter 2 for common elements. These $F\mu$ values may be used in lieu of performing nonlinear analyses, so long as the following cautions are observed.

A significant difference between the R factors from the IBC and the $F\mu$ factors to be used for PC-3 and PC-4 SSCs is that R applies to the entire lateral force resisting system and $F\mu$ applies to individual elements of the lateral force resisting system. For elements for which $F\mu$ is not specified in Chapter 2, it is permissible to use the $F\mu$ value which applies to the overall structural system. For example, to evaluate diaphragm elements, footings, pile foundations, etc.: (1) $F\mu$ 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, $F\mu$ of 1.75 may be used.

In the evaluation of existing facilities, it is necessary to evaluate an appropriate value of $F\mu$ to be used. The $F\mu$ 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 International Building Code (IBC, Ref. C-28) or ACI 349 (Reference C-48). It is highly unlikely that existing facilities will satisfy the seismic detailing requirements of the current IBC if they were designed and constructed many years ago. If structures have less ductility than the IBC 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, $F\mu$ 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., $F\mu$ of unity). To arrive at reduced $F\mu$ 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 $F\mu$ 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 $F\mu$ should be 1.0.

The 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, $F\mu$ 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 $F\mu$ 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 $F\mu$ 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_{SI} 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_{SI} can be limited to the lesser of the following:

1) D_{SI} given by dividing elastic demand by $F\mu$, or

2) $D_{SI} = C_C - D_{NS}$ 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_{TI}, 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, $F\mu$:

1. Chapter 2 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 $F\mu$ 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 several common structural systems. 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. For PC-1 and PC-2, R Factors are to be used per IBC 2000.

C.4.4.3 Guidance on Estimating the Inelastic Energy Absorption Factor, *Fμ*

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 $F\mu$ 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 $F\mu$ may be determined from nonlinear analysis of an appropriate model of the structure using multiple realistic input time histories. First, the input tine 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 $F\mu$ 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:

$$\mu = \frac{\sum W_i \delta_{Ti}}{\sum W_i \delta_{ei}} \tag{C-8}$$

where W_i is the inertial weight applied at each story of the structure, δ_{Ti} is the total displacement relative to the base of each story corresponding to the permissible total distortion occurring in the critical story, and δ_{ei} 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:

$$\mu_s = \frac{\delta_T}{\delta_y} \tag{C-9}$$

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:

$$\mu = 1 + F_K(\mu_s - 1) \tag{C-10}$$

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 l.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 $F\mu$. 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 $F\mu$ for a concrete shear wall structure.

For purposes of this illustration, a three-story structure with the properties shown in Figure C-7 will be used. This figure shows the weights, W, at each story, the elastic stiffnesses, k, and the ultimate capacities, $V\mu$, 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 l.0 g NUREG/CR-0098 median spectrum (Reference C-15), at f equal to 8.25 Hz the elastic spectral acceleration is:

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

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 l.02 for the first story wall to l.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 in Table C-8. Note in Table C-8 that the minimum value of $V\mu/V_R$ (i.e., l.02) is used to calculate δ since this corresponds to the first element which reaches yield (i.e., level l).

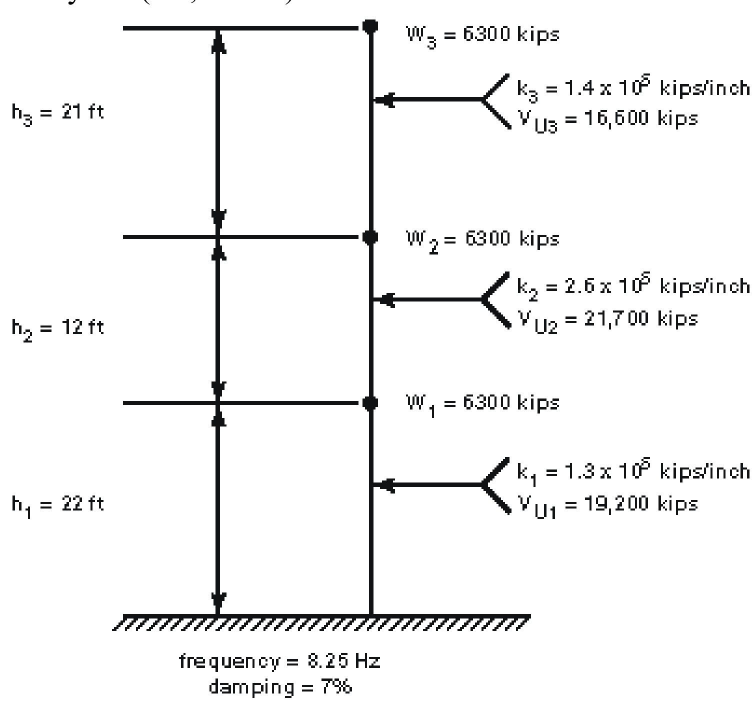

Figure C-7 Three Story Shear Wall Structure

Table C-5 Elastic Response to Reference 1.0g NUREG/CR-0098 Spectrum (7% damping)

Level	Demand			
	Displacement δ_R (inch)	Shear V_R (kips)	Capacity/Demand V_u / V_R (inch)	Yield Displacement $\delta_e=\delta_R(V_u/V_R)_{min}$ (inch)
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:

$$\delta_{TI} = 0.004(22 \quad ft)(12 \quad inch/ft) = 1.06 \quad inch \tag{C-12}$$

and the story ductility μ 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:

$\mu = 4.1 \, to \, 5.6$	(C-14)

However, a more accurate estimate of 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 in Table C-8. Thus:

δ_{T1} = 1.06 inch

δ_{T2} = 1.12 inch

δ_{T3} = 1.22 inch

and from Equation C-8:

$\mu = \dfrac{6300\,(1.22) + 2100\,(1.12) + 2500\,(1.06)}{6300\,(.309) + 2100\,(0.211) + 2500\,(0.147)} = 4.6$	(C-15)

Computation of Inelastic Energy Absorption Factor - For concrete shear wall structures, it is recommended that the inelastic energy absorption factor $F\mu$ 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:

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

Then, the effective frequency is given by:

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

where:

$$A = C_f\left[1-\left(f_s/f\right)\right] \le 0.85$$

For ground motions with strong durations greater than one second, Cf 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:

$$\left(f_e/f\right) = 0.15 + 0.85(0.466) = 0.546 \qquad \text{(C-18)}$$

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

The effective damping β0 may be estimated from:

$$\beta_e = \left(f_s/f_e\right)^2\left[\beta + \beta_H\right] \qquad (C-19)$$

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

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

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

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

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

$S_A\left(F_e, \beta_e\right) = 1.68g$	(C-22)

and the inelastic energy absorption factor is given by:

$F_\mu = \left[\frac{f_e}{f_s}\right]^2 \frac{S_A(f,\beta)}{S_A(f_e,\beta_e)} = \left[\frac{.546}{.466}\right]^2 \left[\frac{1.86}{1.68}\right] = 1.52$	(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, `, included as specified in ACI-318 or ACI-349 are used. For structural steel, the appropriate AISC specifications are used. The LRFD Ref. C-51) limit-state strength approach with the appropriate capacity reduction factor, `, included is preferred. However, the Plastic Design (applicable IBC 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 produce 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 Services Building, Imperial Valley, CA earthquake, 1979). In addition, adjacent structures should be separated sufficiently so that they do not hammer one another during seismic response.

Continuous and Redundant Load Paths - Earthquake excitation induces forces at all points within structures or equipment of significant mass. These forces can be vertical or along any horizontal (lateral) direction. Structures are most vulnerable to damage from lateral seismic-induced forces, and prevention of damage requires a continuous load path (or paths) from regions of significant mass to the foundation or location of support. The designer/evaluator must follow seismic-induced forces through the structure (or equipment or 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. SAC document recommendations of AISC following Northridge earthquake must be complied with.

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. Based on DOE program on the application of experience data for the evaluation of existing systems and components at DOE facilities, a document has been prepared specifically tailoring it to DOE type facilities (Reference C-29).

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.

PC-1 and PC-2 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 IBC. For PC-3 and PC-4 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 failing and failing 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 and C-29. 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.

The requirements of Executive Order 12941, ICSSC Standard RP6, (Ref. C-71) FEMA 310 and FEMA 356 (Ref. C-70) must be taken into consideration while evaluating and upgrading existing facilities.

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 DOE Order 5700.6C (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 IBC 2000 be

carefully followed with the same considerations as outlined in Chapter 2. As discussed in Chapter 2, the IBC provisions are appropriate for PC-1 and PC-2 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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