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BASIS FOR SEISMIC PROVISIONS OF DOE-STD-1020

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1. Introduction

DOE-STD-1020 (Ref. 1) provides for a graded approach for the seismic design and evaluation of DOE structures, systems, and components (SSC). By this graded approach, each SSC is assigned to a Performance Category (PC) with a performance description and an approximate annual probability of seismic-induced unacceptable performance, P_F . Table 1-1 presents the seismic annual probability performance goals for PC 1 through 4 for which specific seismic design and evaluation criteria are presented in DOE-STD-1020. In addition, DOE-STD-1020 also provides a seismic design and evaluation procedure applicable to achieve any seismic performance goal annual probability of unacceptable performance specified by the user.

The desired seismic performance goal is achieved by defining the seismic hazard in terms of a site-specified design/evaluation response spectrum (called herein, the Design/Evaluation Basis Earthquake, DBE). Probabilistic seismic hazard estimates are used to establish the DBE. The resulting seismic 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. Once the DBE is defined, the SSC is designed or evaluated for this DBE using 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 , i.e.:

$$R_R = \frac{P_H}{P_F} \quad (1)$$

The required degree of conservatism in the deterministic acceptance criteria is a function of the specified risk reduction ratio.

The seismic design or evaluation criteria for a specific SSC is established in three steps:

Step 1: Establish an acceptable approximate seismic performance goal for the components being designed or evaluated.

- Step 2:** Establish a set of conservative seismic acceptance criteria which introduce a significant reduction in the risk of unacceptable seismic performance below the annual frequency of exceedance of the DBE.
- Step 3:** Establish the DBE at an annual frequency of exceedance, P_H , equal to R_R (from Step 2) times P_F (from Step 1), i.e.:

$$P_H = R_R(P_F) \quad (2)$$

Table 1-2 provides a set of seismic hazard exceedance probabilities P_H , and risk reduction ratios R_R for Performance Categories 1 thru 4 required to achieve the seismic performance goals specified in Table 1-1. Sufficient conservatism must be embedded into the specified seismic acceptance criteria for each performance category to achieve the desired risk reduction ratio, R_R . The next section defines the required degree of conservatism to achieve risk reduction ratios of 2, 5, 10 and 20. Subsequent sections then define conservative deterministic seismic evaluation and design criteria which are aimed toward achieving this required degree of conservatism.

Table 1-1

Structure, System, or Component (SSC) Seismic
Performance Goals for Various Performance Categories

Performance Category	Performance Goal Description	Seismic Performance Goal Annual Probability of Unacceptable Performance
1	Maintain Occupant Safety	$\approx 10^{-3}$ of the Onset of SSC ⁽¹⁾ Damage to the Extent that Occupants Are Endangered
2	Occupant Safety, Continued Operation with Minimal Interruption	$\approx 5 \times 10^{-4}$ of SSC Damage to the Extent that the Component Cannot Perform its Function
3	Occupant Safety, Continued Function, Hazard Confinement	$\approx 10^{-4}$ of SSC Damage to the Extent that the Component Cannot Perform its Function
4	Occupant Safety, Continued Function, Hazard Confinement	$\approx 10^{-5}$ of SSC Damage to the Extent that the Component Cannot Perform its Function.

(1) SSC Refers to Structure, Distribution System or Components (Equipment)

Table 1-2

Seismic Performance Goals & Recommended
Seismic Hazard Probabilities

Performance Category	Seismic Performance Goal, P_F	Seismic Hazard Exceedance Probability, P_H	Ratio of Hazard to Performance Probability, R_R
1	1×10^{-3}	2×10^{-3}	2
2	5×10^{-4}	1×10^{-3}	2
3	1×10^{-4}	5×10^{-4} 1×10^{-3} (1)	5 10(1)
4	1×10^{-5}	1×10^{-4} 2×10^{-4} (1)	10 20(1)

(1) Hazard exceedance probability, P_H , and risk reduction ratio, R_R , are doubled at sites near tectonic plate boundaries (see Chapter 2).

2. Required Level of Seismic Design Conservatism to Achieve a Specified Seismic Risk Reduction Ratio

2.1 Derivation

Figure 2-1 presents two representative probabilistic seismic hazard curves expressed in terms of mean annual probability of exceedance versus peak ground acceleration. Curve A represents a hazard estimate for a higher seismicity site. Curve B represents a typical hazard estimate for a lower seismicity site.

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

$$H_{(a)} = K_1 a^{-K_H} \quad (3)$$

where $H_{(a)}$ is the annual frequency of exceedance of ground motion level "a," K_1 is an appropriate constant, and K_H is a slope parameter defined by:

$$K_H = \frac{1}{\log(A_R)} \quad (4)$$

in which A_R is the ratio of ground motions corresponding to a ten-fold reduction in exceedance probability. The use of Equation (3) is suggested in Ref. 3. From Figure 2-1 comes the ground motion ratios and hazard slope parameters shown in Table 2-1. These results are representative. Appendix A 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 ranges from 2 to 4 although values of as much as 6 have been estimated. For California and other high seismic sites with seismicity dominated by close active faults with high recurrence rates associated with tectonic plate boundaries, A_R typically ranges from 1.5 to 2.25. For other western sites with seismicity not dominated by tectonic plate boundaries 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. Furthermore,

A_R is not constant over probability ranges that differ by an order of magnitude, with A_R always reducing as the exceedance probability is lowered.

In order to compute the risk reduction ratio, R_R , corresponding to any specified seismic design/evaluation criteria, one must also define a mean seismic fragility curve for a component resulting from the usage of these seismic criteria. This mean seismic fragility curve describes the probability of an unacceptable performance versus the ground motion level. This fragility curve is defined as being lognormally distributed and is expressed in terms of two parameters: a median capacity level and a composite logarithmic standard deviation β (see Ref. 2 for further amplification). To estimate this composite logarithmic standard deviation, it is sufficient to estimate the 50% failure probability capacity C_{50} and the capacity associated with any one of the following failure probabilities: 1%, 2%, 5% or 10%. Then, the composite logarithmic standard deviation can be computed from the ratio of these two capacity estimates. The logarithmic standard deviation β will generally lie within the range of 0.3 to 0.5 for structures and equipment mounted at ground level. For equipment mounted high in structures, β will generally lie within the range of 0.4 to 0.6.

The probability, P_F , of unacceptable performances is obtained by a convolution of the seismic hazard and fragility curves. This convolution can be expressed by either:

$$P_F = \int_0^{+\infty} \left(\frac{dH(a)}{da} \right) P_{F/a} da \quad (5a)$$

or

$$P_F = \int_0^{+\infty} H(a) \left(\frac{dP_{F/a}}{da} \right) da \quad (5b)$$

where $P_{F/a}$ is the conditional probability of failure given the ground motion level "a" which is defined by the SSC fragility curve. Assuming a lognormally distributed fragility curve with median capacity, C_{50} , and logarithmic standard deviation β , and defining the hazard exceedance probability $H(a)$ by Equation (3), from Equation (5b) one obtains:

$$P_F = \int_0^{+\infty} \{K_1 a^{-K_H}\} \left[\frac{1}{a\beta\sqrt{2\pi}} e^{-\left\{\frac{(\ln a - M)^2}{2\beta^2}\right\}} \right] da \quad (5c)$$

$$M = \ln C_{50}$$

Defining $X = \ln a$, Equation (5c) becomes:

$$P_F = \frac{K_1}{\beta\sqrt{2\pi}} \int_{-\infty}^{+\infty} \left[e^{K_H x} \right] \left[e^{-\left\{\frac{(x-M)^2}{2\beta^2}\right\}} \right] dx \quad (5d)$$

Then, solving this definite integral by the approach shown in Appendix A of Reference 4 or other probability textbook, one obtains:

$$P_F = K_1 e^{-K_H M} e^{1/2(K_H \beta)^2} \quad (5e)$$

or from the previous definitions of M:

$$P_F = K_1 C_{50}^{-K_H} e^{1/2(K_H \beta)^2} \quad (5f)$$

Defining H_D as the annual frequency of exceedance of the DBE ground motion level, from Equation (3):

$$K_1 = H_D [DBE]^{K_H} \quad (5g)$$

from which:

$$P_F = \frac{H_D e^{1/2(K_H \beta)^2}}{(C_{50} / DBE)^{K_H}} \quad (5)$$

Equation (5) is exact so long as the fragility is lognormally distributed and the hazard curve is defined by Equation (3), (i.e., is linear on a log-log plot). Equation (5) will be used to derive the required level of seismic design conservatism to achieve any specified seismic risk reduction ratio R_R .

With the DBE defined by:

$$DBE = a_{p_H} \quad (6)$$

where a_{p_H} is the mean ground motions at the seismic hazard probability, P_H , then:

$$H_D = P_H \quad (7)$$

and thus the risk reduction ratio, R_R , between the annual frequency of exceedance of the DBE and the probability of an unacceptable performance is given by:

$$R_R = \frac{H_D}{P_F} = (C_{50} / DBE)^{K_H} e^{-1/2(K_H \beta)^2} \quad (8)$$

Next, the minimum required capacity, C_p , at any failure probability "P" to achieve a risk reduction, R_R , can be defined by:

$$C_p = F_{p_R} (DBE) \quad (9)$$

where F_{p_R} is the required safety factor which is a function of both the probability P and the risk ratio, R_R . Then, the required median capacity is:

$$C_{50\%} = C_p e^{X_p \beta} \quad (10)$$

where X_p is the factor associated with the failure probability "P" for the standard normal distribution, i.e.:

P	X _p	P	X _p
1%	2.326	15%	1.037
5%	1.645	20%	0.842
10%	1.282	50%	0

Combining Equation (8) with Equations (9) and (10):

$$R_R = (F_{PR})^{K_H} e^{[X_P K_H \beta - 1/2(K_H \beta)^2]} \quad (11)$$

Since the seismic fragility logarithmic standard deviation β is unknown but is expected to lie in the range of 0.3 to 0.6, it is desirable to select "P" and X_P to minimize the variation of R_R with β over this range. Thus:

$$f = X_P K_H \beta - 1/2 (K_H \beta)^2 \quad (12)$$

should have minimum variation with variation of β . Table 2-2 presents the range of maximum to minimum values of the factor f over the range of β from 0.3 to 0.6 for various K_H and X_P values. Over the range of A_R from 2 to 4, the range of variation of f is minimum for X_P of 1.282, that is, a failure capacity "P" of 10%. This range is not significantly increased when 5% or 15% failure capacities are used. However, when either the median failure capacity $C_{50\%}$ or the very low 1% failure capacity $C_{1\%}$ is used, the scatter range on f becomes much larger which indicates that it is much less desirable to define the required factor of conservatism F_{PR} in terms of these failure probabilities instead of the 10% failure probability capacity.

It will be shown in Section 4 that the basic deterministic evaluation criteria given in DOE-STD-1020 is aimed at achieving a 1.5 factor of safety on the 10% failure capacity $C_{10\%}$. If a different 10% failure capacity factor of safety F_R is desired, this can be achieved by scaling the elastic computed seismic demand for the DBE by a scale factor SF. Then:

$$C_{10\%} \geq F_R (\text{DBE}) \quad (13a)$$

$$F_R \geq 1.5 \text{ SF} \quad (13b)$$

and Equation (11) may be rewritten as:

$$R_R = (1.5 SF)^{K_H} e^{[1.282 K_H \beta - 1/2 (K_H \beta)^2]} \quad (14)$$

Then, Equation (14) may be used to determine the risk reduction ratio R_R which is achieved for any given scale factor SF .

2.2 Establishing Required Scale Factor for a Given Risk Reduction Ratio

Table 2-3 shows the risk reduction ratio R_R for scale factors of 0.67, 1.0, 1.25, and 1.6, respectively, for A_R ratios from 1.5 to 4 and β from 0.3 to 0.6 based on Equation (14). These results are illustrated in Figure 2-2 extending out to an A_R of 6. Table 2-4 presents the recommended constant scale factors to achieve risk reduction ratios of 2, 5, 10, and 20 based on the results shown in Table 2-3 and Figure 2-2. Over a slope ratio A_R range of 2 to 4, the resulting risk reduction ratio to target ratio range is also shown in Table 2-4. This is as good as one can do with the use of a constant scale factor over the range of A_R and β values.

Table 2-3 and Figure 2-2 demonstrate that the risk reduction ratio, R_R is about double the target R_R value when A_R is about 2. Hence, DOE-STD-1020 permits doubling the hazard exceedance probability for tectonic plate boundary sites where low A_R are estimated. By such an approach, the same target performance goals may be met for all sites in an acceptable manner.

The ability to achieve the desired risk reduction ratio can be greatly improved by introducing the complexity of making the scale factor SF a function of the ground motion A_R ratio (as defined by Equation 15c). Except at low A_R values (less than 2.0), excellent fitting to the target risk reduction ratio is obtained by defining the scale factor by:

$$SF = \text{larger of } \begin{cases} SF_1 \\ SF_2 \end{cases} \quad (15a)$$

$$SF_2 = 0.6 (A_R)^\alpha \quad (15b)$$

where SF_1 and α are defined in Table 2-5 and the resulting variable scale factors are plotted in Figure 2-3 for R_R of 5, 10, and 20. This refinement is unnecessary for $R_R = 2$ and, for this case, the constant SF of 0.67 is retained.

Table 2-6 presents the achieved risk reduction ratios for various A_R and β values when the scale factor SF is defined by Equations (15a) and (15b). These results are plotted in Figure 2-4. Over the range of A_R from 2 to 4 and β from 0.3 to 0.6, the ratio of achieved to target R_R values range from 78% to 117% with the extremes occurring for either $\beta = 0.3$ or $\beta = 0.6$ which are both unlikely extremes on β . However, since the criteria has been defined in terms of the 10% failure capacity in order to minimize the influence of β for A_R values from 2 to 4 (see Table 2-2), at A_R values less than 2, the achieved risk reduction ratio becomes quite sensitive to β , so the agreement between the achieved and target R_R values becomes poor at low A_R values and either high or low β . Even so, the agreement is good for the most typical β values in the range of 0.4 to 0.5. Greater refinement would require the estimation of the fragility logarithmic standard deviation β and to make the scale factor a function of both A_R and β , and thus refinement is not practical. Note that doubling of hazard exceedance probabilities at tectonic plate boundary sites is not permitted when the variable scale factor is used.

2.3 Recommended Scale Factors

If the seismic hazard curve slope is either unknown or questionable for annual probabilities less than the seismic hazard exceedance probability, P_H , use the constant scale factors given in Table 2-4. However, if the hazard curve slope ratio A_R can be reasonably estimated between the hazard exceedance probability P_H and the performance goal probability P_F , a substantially improved estimate of the appropriate scale factor can be obtained from Equations (15a) and (15b) with the parameters from Table 2-5 or Figure 2-3. Over the portion of the hazard curve from the hazard exceedance probability, P_H , to the performance goal probability, P_F , A_R may be estimated from:

$$A_R = \left(\frac{a_{PF}}{a_{PH}} \right)^{1/\log R_R} \quad (15c)$$

where a_{PH} and a_{PF} are the ground motions at the hazard exceedance probability, P_H , and the performance goal probability, P_F , respectively.

2.4 Validation

Actual hazard curves are not perfectly linear when plotted on a log-log scale (for example, see Figure 2-1). Therefore, Equation (3) which was used to derive Equation (5), and thus the results of the previous subsections, is only an approximation of an actual hazard curve. Essentially all hazard curves have reduction in A_R (i.e., increasing K_H values) as the exceedance probability is lowered, which shows up as a slightly concave downward hazard curve in Figure 2-1. The combined effect of using Equation (3) to define the hazard curve and the use of the required factor of safety F_R values defined by Equation (13b) and the scale factor from Figure 2-3 will be shown in this section using both hazard curves A and B from Figure 2-1.

For any shape hazard curve, the probability P_F of unacceptable performance may be obtained by numerically integrating either Equation (5a) or (5b). From Equation (5a):

$$P_F = \sum_{i=1}^{\infty} [H(a_i) - H(a_{i+1})] P_{F|a_{cg_i}} \quad (16)$$

where a_{cg_i} is the center-of-gravity ground motion level between a_i and a_{i+1} defined by:

$$a_{cg_i} = \frac{\int_{a_i}^{a_{i+1}} H(a) a da}{\int_{a_i}^{a_{i+1}} H(a) da} \quad (16a)$$

and $P_{F|a_{cg_i}}$ is the conditional failure probability at ground motion level a_{cg_i} . Assuming a piecewise linear hazard curve defined locally by Equation (3) between a_i and a_{i+1} and defining the local slope parameter K_{H_i} by:

$$K_{Hi} = \frac{\log(H_{(a_i)} / H_{(a_{i+1})})}{\log((a_{i+1}) / (a_i))} \quad (16b)$$

then Equation (16a) gives:

$$a_{CGi} = \frac{(1 - K_{Hi}) \left[a_{i+1}^{(2 - K_{Hi})} - a_i^{(2 - K_{Hi})} \right]}{(2 - K_{Hi}) \left[a_{i+1}^{(1 - K_{Hi})} - a_i^{(1 - K_{Hi})} \right]} \quad (16c)$$

The use of Equation (16c) to define a_{CGi} improves the accuracy of Equation (16) over that obtained using the midpoint acceleration and thus permits larger acceleration steps to be used.

Hazard exceedance values versus peak ground acceleration from Figure 2-1 are tabulated in Table 2-7 for seismic hazard curves A and B. Table 2-8a presents for seismic hazard curve A (Figure 2-1) and target performance goals ranging from 1×10^{-5} to 5×10^{-4} , the required DBE level (from Equation (6)), the scale factor from Figure 2-3, the required 10% probability of failure capacity $C_{10\%}$ (from Equations (13a) and (13b)), the resultant 50% failure probability capacity C_{50} for several logarithmic standard deviations (from Equation 10), and the resulting unacceptable performance probability P_F obtained from Equation (16) using these capacities. Table 2-8b presents similar results for seismic hazard curve B (Figure 2-1). Table 2-9 presents an example solution of Equation (16) for the case of seismic hazard curve B, target performance goal of 1×10^{-4} , and logarithmic standard deviation, β , of 0.4 to illustrate how the resulting P_F results given in Tables 2-8a and 2-8b were obtained.

For the 18 cases presented in Tables 2-8a and 2-8b, the basic criteria of the previous subsection leads to probabilities of unacceptable performance which range from 72% to 119% of the desired performance goal probabilities. This excellent prediction of failure probabilities validates the basic criteria of the previous section.

2.5 Conclusion

The performance goals are accurately achieved over a wide range of hazard curves and fragility curves when:

1. The DBE is defined from probabilistic hazard curves by Equation (6).
2. The required 10% probability of failure seismic capacity $C_{10\%}$ is defined by Equations (13a) and (13b) where the scale factor SF is defined as a function of R_R .

These two steps represent the basic seismic acceptance criteria. The most general approach to demonstrate compliance with the above basic seismic criteria is as follows:

1. Establish the DBE ground motion.
2. Define a Scaled Design Basis Earthquake (SDBE) ground motion by increasing the DBE ground motion by the required safety factor F_R , i.e.:

$$\text{SDBE} = F_R (\text{DBE}) = (1.5 \text{ SF}) (\text{DBE}) \quad (17)$$

3. Perform sufficient linear analyses, nonlinear analyses, testing, etc., to reasonably determine that for the combination of the SDBE with the best-estimate of the concurrent non-seismic loads there is less than about a 10% probability of unacceptable performance.

Any seismic evaluation approach which is consistent with the above three steps is an acceptable approach to approximate the desired performance goal and is thus permitted in DOE-STD-1020 (Ref. 1). However, most seismic design and evaluation engineers prefer to work with a deterministic evaluation procedure based on elastic analysis and code capacities in lieu of the quasi-probabilistic approach defined above. Therefore, DOE-STD-1020 provides deterministic seismic evaluation procedures which are aimed toward achieving the basic seismic acceptance criteria defined above. The basis for these deterministic seismic evaluation procedures is presented in Section 3 for Performance Categories 1 and 2, ($R_R = 2$) and in Section 4 for Performance Categories 3 ($R_R = 5$) and 4 ($R_R = 10$).

Figure 2-1 Typical Probabilistic Seismic Hazard Curves

Figure 2-2 Risk Reduction Ratios R_R for Various Constant Scale Factors

Figure 2-3 Variable Seismic Scale Factor, S_F

Figure 2-4 Achieved Risk Reduction Ratios R_R for Variable Scale Factors from Equations (15a) and (15b)

Table 2-1

Typical Ground Motion Ratios and Hazard Slope Parameters

Hazard curve (Figure 2-1)	Probability Range	A_R	K_H
A (Higher Seismicity)	10^{-3} to 10^{-4}	2.0	3.32
A (Higher Seismicity)	10^{-4} to 10^{-5}	1.67	4.49
B (Lower Seismicity)	10^{-3} to 10^{-4}	2.31	2.75
B (Lower Seismicity)	10^{-4} to 10^{-5}	2.13	3.05

Table 2-2

Range of Maximum and Minimum Factors f over $\beta = 0.3$ to 0.6 for Various K_H and X_P Values

		Range: Maximum - Minimum f					
A_R	K_H	$C_{50\%}$ $X_P = 0$	$C_{20\%}$ $X_P = 0.842$	$C_{15\%}$ $X_P = 1.037$	$C_{10\%}$ $X_P = 1.282$	$C_{5\%}$ $X_P = 1.645$	$C_{1\%}$ $X_P = 2.326$
4.00	1.66	0.37	0.05	0.14	0.27	0.45	0.79
3.75	1.74	0.41	0.05	0.13	0.26	0.45	0.81
3.50	1.84	0.46	0.04	0.12	0.25	0.45	0.83
3.25	1.95	0.51	0.05	0.10	0.24	0.45	0.85
3.00	2.10	0.60	0.09	0.08	0.21	0.44	0.87
2.75	2.28	0.70	0.14	0.06	0.18	0.42	0.89
2.50	2.51	0.85	0.22	0.11	0.14	0.39	0.90
2.25	2.84	1.09	0.37	0.22	0.09	0.31	0.89
2.00	3.32	1.49	0.65	0.46	0.25	0.21	0.83
1.75	4.11	2.28	1.24	1.00	0.70	0.34	0.60
1.50	5.68	4.36	2.92	2.59	2.17	1.55	0.59

Table 2-3
Risk Reduction Ratio R_R Achieved as a Function of Scale Factor SF

		Risk Reduction Ratio, R_R															
		SF = 0.67				SF = 1.0				SF = 1.25				SF = 1.6			
A_R	K_H	$\beta=.3$	$\beta=.4$	$\beta=.5$	$\beta=.6$	$\beta=.3$	$\beta=.4$	$\beta=.5$	$\beta=.6$	$\beta=.3$	$\beta=.4$	$\beta=.5$	$\beta=.6$	$\beta=.3$	$\beta=.4$	$\beta=.5$	$\beta=.6$
1.5	5.68	2.14	1.43	0.70	0.24	20.81	13.94	6.76	2.38	73.90	49.50	24.02	8.44	300.23	201.11	97.58	34.29
1.75	4.11	2.32	2.17	1.72	1.15	12.05	11.29	8.93	5.96	30.18	28.28	22.37	14.94	83.33	78.08	61.77	41.25
2	3.32	2.22	2.31	2.15	1.80	8.40	8.74	8.14	6.79	17.62	18.34	17.08	14.25	40.02	41.63	38.79	32.37
2.25	2.84	2.10	2.28	2.29	2.11	6.56	7.12	7.12	6.58	12.36	13.41	13.43	12.40	24.91	27.03	27.06	25.00
2.5	2.51	2.00	2.22	2.30	2.25	5.48	6.06	6.30	6.14	9.60	10.62	11.04	10.76	17.86	19.76	20.52	20.01
2.75	2.28	1.92	2.15	2.28	2.29	4.78	5.34	5.67	5.70	7.95	8.88	9.41	9.48	13.94	15.57	16.51	16.63
3	2.10	1.86	2.08	2.24	2.30	4.30	4.82	5.18	5.32	6.86	7.70	8.26	8.49	11.51	12.91	13.86	14.24
3.25	1.95	1.80	2.03	2.19	2.28	3.94	4.43	4.79	4.99	6.10	6.85	7.41	7.72	9.87	11.10	12.01	12.50
3.5	1.84	1.76	1.98	2.15	2.26	3.67	4.13	4.49	4.72	5.53	6.22	6.76	7.11	8.71	9.79	10.64	11.19
3.75	1.74	1.72	1.93	2.11	2.23	3.45	3.88	4.24	4.48	5.10	5.73	6.25	6.61	7.83	8.81	9.61	10.16
4	1.66	1.69	1.90	2.07	2.20	3.28	3.69	4.03	4.28	4.75	5.34	5.84	6.20	7.16	8.05	8.79	9.35

Table 2-4

**Recommended Constant Scale Factors SF
to Achieve Various Risk Reduction Ratios**

Risk Reduction Ratio, R_R	Constant Scale Factor, SF	Range of Achieved R_R to Target R_R *
20	1.6	40% to 210%
10	1.25	50% to 185%
5	1.0	70% to 175%
2	0.67	85% to 115%

* Over the A_R range from 2 to 4

Table 2-5

Recommended Variable Scale Factor

Parameters to use in Equations (15a) and (15b)

Risk Reduction Ratio, R_R	Scale Factor Parameter, SF_1	Scale Factor Exponent, α	Range of Achieved R_R to Target R_R *
20	1.15	1.1	78% to 116%
10	1.0	0.8	79% to 116%
5	0.87	0.5	86% to 117%
2	0.67	-	85% to 115%

* Over the A_R range from 2 to 4

Table 2-6
Achieved Risk Reduction Ratios Obtained Using Variable Scale Factor from Equations (15a) & (15b)

A_R	K_H	Risk Reduction Ratio, R_R											
		Target $R_R = 5$				Target $R_R = 10$				Target $R_R = 20$			
		$\beta=.3$	$\beta=.4$	$\beta=.5$	$\beta=.6$	$\beta=.3$	$\beta=.4$	$\beta=.5$	$\beta=.6$	$\beta=.3$	$\beta=.4$	$\beta=.5$	$\beta=.6$
1.5	5.68	9.44	6.32	3.07	1.08	20.81	13.94	6.76	2.38	46.02	30.83	14.96	5.26
1.75	4.11	6.79	6.37	5.04	3.36	12.05	11.29	8.93	5.96	21.41	20.06	15.87	10.60
2	3.32	5.29	5.50	5.13	4.28	9.71	10.10	9.41	7.85	19.37	20.16	18.78	15.67
2.25	2.84	4.86	5.28	5.28	4.88	9.70	10.53	10.54	9.74	19.35	21.00	21.03	19.42
2.5	2.51	4.80	5.31	5.52	5.38	9.58	10.60	11.01	10.74	19.11	21.15	21.97	21.42
2.75	2.28	4.73	5.28	5.60	5.64	9.44	10.54	11.17	11.25	18.83	21.03	22.30	22.45
3	2.10	4.66	5.23	5.61	5.77	9.30	10.43	11.20	11.50	18.55	20.81	22.34	22.95
3.25	1.95	4.60	5.17	5.59	5.82	9.17	10.31	11.15	11.61	18.29	20.56	22.25	23.17
3.5	1.84	4.54	5.10	5.55	5.83	9.05	10.18	11.07	11.64	18.07	20.32	22.09	23.22
3.75	1.74	4.49	5.04	5.50	5.82	8.95	10.07	10.98	11.61	17.86	20.08	21.90	23.17
4	1.66	4.44	4.99	5.45	5.80	8.86	9.96	10.88	11.57	17.68	19.86	21.71	23.08

Table 2-7

Probabilities for Hazard Curves A and B from Figure 2-1

Curve A		Curve B	
Acceleration a (g)	Exceedance Probability H(a)	Acceleration a (g)	Exceedance Probability H(a)
0.14	500×10^{-5}	0.07	500×10^{-5}
0.23	200×10^{-5}	0.10	200×10^{-5}
0.30	100×10^{-5}	0.13	100×10^{-5}
0.38	50×10^{-5}	0.17	50×10^{-5}
0.50	20×10^{-5}	0.24	20×10^{-5}
0.60	10×10^{-5}	0.30	10×10^{-5}
0.71	5×10^{-5}	0.38	5×10^{-5}
0.87	2×10^{-5}	0.51	2×10^{-5}
1.00	1×10^{-5}	0.64	1×10^{-5}
1.14	0.5×10^{-5}	0.78	0.5×10^{-5}
1.33	0.2×10^{-5}	0.97	0.2×10^{-5}
1.49	0.1×10^{-5}	1.12	0.1×10^{-5}

Table 2-8a

Minimum Acceptable Fragility Factor & Resulting Annual
Probability of Unacceptable Performance P_F for Hazard Curve A

Target Performance Goal	P_H and R_R	DBE (g) Eqn (6)	$C_{10\%}$ (g) Eqn (13) & Fig. 2-3	Logarithmic Std. Dev.	$C_{50\%}$ (g) Eqn (10)	P_F Eqn (16)
$P_F = 1 \times 10^{-5}$	$P_H = 2 \times 10^{-4}$ $R_R = 20$	0.50	SF = 1.15 $C_{10\%} = 0.85$	0.3	1.249	0.88×10^{-5}
				0.4	1.42	0.94×10^{-5}
				0.5	1.614	1.13×10^{-5}
$P_F = 1 \times 10^{-4}$	$P_H = 1 \times 10^{-3}$ $R_R = 10$	0.30	SF = 1.0 $C_{10\%} = 0.45$	0.3	0.661	1.19×10^{-4}
				0.4	0.752	1.10×10^{-4}
				0.5	0.854	1.09×10^{-4}
$P_F = 5 \times 10^{-4}$	$P_H = 1 \times 10^{-3}$ $R_R = 2$	0.30	SF = 0.67 $C_{10\%} = 0.30$	0.3	0.441	4.4×10^{-4}
				0.4	0.501	3.9×10^{-4}
				0.5	0.570	3.6×10^{-4}

Table 2-8b

Minimum Acceptable Fragility Factor & Resulting Annual
Probability of Unacceptable Performance P_F for Hazard Curve B

Target Performance Goal	P_H and R_R	DBE (g) Eqn (6)	$C_{10\%}$ (g) Eqn (13) & Fig. 2-3	Logarithmic Std. Dev.	$C_{50\%}$ (g) Eqn (10)	P_F Eqn (16)
$P_F = 1 \times 10^{-5}$	$P_H = 2 \times 10^{-4}$ $R_R = 20$	0.24	SF = 1.37 $C_{10\%} = 0.49$	0.3	0.72	1.02×10^{-5}
				0.4	0.818	0.94×10^{-5}
				0.5	0.93	0.96×10^{-5}
$P_F = 1 \times 10^{-4}$	$P_H = 1 \times 10^{-3}$ $R_R = 10$	0.13	SF = 1.16 $C_{10\%} = 0.225$	0.3	0.330	1.10×10^{-4}
				0.4	0.376	1.00×10^{-4}
				0.5	0.427	0.98×10^{-4}
$P_F = 5 \times 10^{-4}$	$P_H = 1 \times 10^{-3}$ $R_R = 2$	0.13	SF = 0.67 $C_{10\%} = 0.13$	0.3	0.191	5.0×10^{-4}
				0.4	0.217	4.5×10^{-4}
				0.5	0.247	4.3×10^{-4}

Table 2-9

Example Solution of Equation (16) for Hazard Curve B,
Target Performance Goal of 1×10^{-4} , $C_{50} = 0.376g$, $\beta = 0.40$

Acceleration a (g)	Exceedance Probability H(a) ($\times 10^{-5}$)	C.G. Acceleration a _{cg} (g)	(1) Conditional Failure Probability P _F /a _{cg}	(2) Probability Hazard Within Range [H(a _i) - H(a _{i+1})] ($\times 10^{-5}$)	(1) x (2) ($\times 10^{-5}$)
.07	500				
		.0828	.0000775	300	.02
.10	200				
		.113	.00133	100	.13
.13	100				
		.148	.00988	50	.49
.17	50				
		.200	.0573	30	1.72
.24	20				
		.266	.193	10	1.93
.30	10				
		.335	.386	5	1.93
.38	5				
		.435	.642	3	1.93
.51	2				
		.568	.849	1	.85
.64	1				
		.700	.940	.5	.47
.775	.5				
		.858	.980	.3	.29
.97	.2				
		1.036	.994	.1	.10
1.12	.1				
			≈ 1.0	.1	.10

$$P_F = \sum (1) \times (2) = 9.96 \times 10^{-5}$$

3. Deterministic Seismic Acceptance Criteria for Performance Categories 1 and 2

For Performance Categories (PC) 1 and 2, the risk reduction ratio, R_R , is 2 as specified in Table 1-2. For $R_R = 2$, from Table 2-4, the scale factor SF is 0.67 and, thus the required factor of safety, $F_R = 1.0$. Thus, to achieve a risk reduction ratio of 2, there needs to be about a 10% probability of unacceptable performance when a SSC is subjected to the DBE. Based on experience from past earthquakes, normal building codes such as the Uniform Building Code (Reference 5) result in less than a 10% probability of unacceptable performance when a SSC is subjected to the DBE which corresponds to Z for the Uniform Building Code.

Therefore, DOE-STD-1020 specifies that the Uniform Building Code provisions be used for the seismic evaluation of PC 1 and 2 SSCs, except that the DBE specified by Equation (6a) be used for Z in the code equations.

4. Deterministic Seismic Acceptance Criteria for Performance Category 3 and Higher

4.1 Overview of Deterministic Seismic Criteria

DOE-STD-1020 specifies for PC 3 and higher that the seismic evaluation must be performed by a dynamic analysis approach. By this approach, an elastic response analysis to determine the elastic-computed seismic demand D_s from the DBE is first performed. The elastic seismic demand is computed in accordance with the seismic analysis requirements of ASCE Standard 4-86 (Ref. 6) with one exception. ASCE Standard 4-86 requires that the design response spectrum be defined using mean-plus-one-standard-deviation-level amplification factors. Unfortunately, this requirement is not compatible with the DBE being defined at a specified mean annual probability of exceedance, as is required in DOE-STD-1020. Mixing these two requirements would lead to a DBE response spectrum which has a variable mean annual probability of exceedance over the natural frequency range of interest, ranging from the specified mean annual probability at high natural frequencies (above about 33 Hz) to substantially less than the specified mean annual probability at natural frequencies of 9 Hz and lower. The resulting variable conservatism cannot be easily accommodated in probabilistic performance goal-based criteria. Therefore, in DOE-STD-1020, a median site-specific response spectrum shape is used so as to maintain a consistent mean annual probability of exceedance over the entire natural frequency range of interest. Even with the above exception, elastic seismic demands are slightly conservatively computed. Note that ASCE 4 is currently being revised and the new version will recommend a median amplification spectral shape.

Next, this elastic computed seismic demand D_s is factored by dividing it by a permissible inelastic demand/capacity factor, F_μ (also often called an inelastic energy absorption factor or ductility factor) and multiplying by an appropriate seismic scale factor, SF, to obtain an inelastic-factored estimate of the seismic demand D_{SI} , i.e.:

$$D_{SI} = SF \frac{D_s}{F_\mu} \quad (18)$$

where SF must correspond to the desired R_R ratio, as given in Table 2-4 or Figure 2-3. The F_μ values given in DOE-STD-1020 are aimed at achieving a R_R ratio of 10 for A_R

values less than or equal to 2.0. The SF factor in Table 2-4 or 2-3 are used to adjust the risk reduction ratios to other values as previously shown in Section 2. The establishment of F_{μ} values is further discussed in Section 5.

The total inelastic-factored demand D_{TI} is then given by:

$$D_{TI} = D_{NS} + D_{SI} \quad (19)$$

where D_{NS} represents the best-estimate of all non-seismic demands expected to occur concurrently with the DBE. This total inelastic-factored demand D_{TI} is held less than the code-specified minimum ultimate or limit-state capacity C_C , i.e.:

$$C_C \geq D_{TI} \quad (20)$$

Equations (19) and (20) represent the DBE load-combination and seismic acceptance criteria appropriate for the DBE, respectively. For use in Equation (20), C_C must include the code specified capacity reduction factor, ϕ .

The deterministic seismic acceptance criteria summarized above and defined in detail in DOE-STD-1020 is aimed toward approximately satisfying Equation (13a) and thus achieving the desired risk reduction R_R ratio as is demonstrated in the next subsection. However, considerable judgment and use of estimated factors of conservatism and variabilities from past seismic probabilistic risk assessment studies (e.g., Ref. 7) is necessary. Therefore, great rigor or quantitative accuracy in achieving these seismic risk reduction factors should not be implied. The factors merely served as target goals in developing the criteria. It is expected that the specified risk reduction R_R ratio is achieved by the above deterministic seismic acceptance criteria at least within a factor of two accuracy, which should be adequate.

4.2 Benchmarking Studies for Deterministic Seismic Acceptance Criteria

4.2.1 Basic Derivation

If F_S is defined as the median seismic factor of safety, then

$$F_S = \frac{C_{50} - D_{NS50}}{D_{S50}} F_{\mu 50} \quad (21)$$

where C_{50} , D_{NS50} , D_{S50} , and $F_{\mu 50}$ are median estimates of the capacity, non-seismic demand, seismic demand, and inelastic energy absorption factor, respectively. In turn,

$$\begin{aligned} C_{50} &= F_C C_C \\ D_{NS50} &= F_{NS} D_{NS} \\ D_{S50} &= D_S / F_R \\ F_{\mu 50} &= F_I F_{\mu} \end{aligned} \quad (22)$$

where C_C , D_{NS} , D_S , and F_{μ} are the capacity, non-seismic demand, seismic demand, and inelastic energy absorption factor, respectively, computed in accordance with the guidance of DOE-STD-1020 as summarized in Subsection 4.1 and F_C , F_{NS} , and F_R and F_I are the estimated median factors of conservatism associated with this guidance for each of these terms. Combining Equations (21) and (22) and rearranging:

$$F_S = \frac{F_R F_I \left[F_C - F_{NS} \left(\frac{D_{NS}}{C_C} \right) \right]}{\left[1 - \left(\frac{D_{NS}}{C_C} \right) \right]} \quad (23)$$

The variability of this factor of safety may be defined in terms of its logarithmic standard deviation β_{FS} given by:

$$\beta_{FS} = \left(\beta_R^2 + \beta_I^2 + \beta_{CS}^2 \right)^{1/2} \quad (24)$$

where β_R , β_I and β_{CS} are the logarithmic standard deviations for the response, inelastic energy absorption, and seismic capacity, respectively. In turn, β_{CS} may be approximated by:

$$\beta_{CS} = \frac{\left\{ \left(F_C \beta_C \right)^2 + \left[F_{NS} \beta_{NS} (D_{NS} / C_C) \right]^2 \right\}^{1/2}}{F_C - F_{NS} (D_{NS} / C_C)} \quad (25)$$

where β_C and β_{NS} are the logarithmic standard deviations for capacity and non-seismic demand, respectively.

Based upon combining Equations (10), (13a), and (13b), the required median factor of safety F_{SRqd} needed to achieve the desired risk reduction ratio is:

$$F_{SRqd} = 1.5 S F e^{1.282 \beta_{FS}} \quad (26)$$

The ratio of F_S from Equation (23) to F_{SRqd} from Equation (26):

$$R_{FS} = \frac{F_S}{F_{SRqd}} \quad (27)$$

defines the adequacy of the deterministic seismic criteria. The value of R_{FS} should be close to unity. If it is substantially less than unity, the criteria are nonconservative. If it substantially exceeds unity, the criteria are more conservative than necessary.

In order to evaluate R_{FS} , factors of conservatism and variabilities must be estimated for seismic demand (response), non-seismic demand, capacity and inelastic energy absorption (ductility). Such estimates are made in the following subsections.

4.2.2 Seismic Demand (Response)

As summarized in Section 4.1, in DOE-STD-1020 the elastic-computed seismic demand is to be obtained in accordance with the requirements of ASCE 4-86, except that median input spectral amplifications are to be used instead of median-plus-one-standard-deviation amplification factors. Based upon Reference 8, the ratio of median-plus-one-standard-deviation to median spectral acceleration amplification factor averages about 1.22 over the 7% to 12% damping range applicable for most structures. In addition, as noted in its foreword, ASCE 4-86 is aimed at achieving about a 10% probability of the actual seismic response exceeding the computed response, given the occurrence of the DBE. Thus, the median response factor of safety F_R can be estimated from:

$$F_R = \frac{e^{1.282\beta_R}}{1.22} \quad (28)$$

Past seismic probabilistic risk assessments indicate a response variability logarithmic standard deviation β_R of about 0.30 for structures and about 0.35 for equipment mounted on structures. Thus:

<u>Structures</u>	<u>Equipment</u>	
$F_R = 1.2$	1.28	(29)
$\beta_R = 0.3$	0.35	

4.2.3 Non-Seismic Demand

The load combination criteria of DOE-STD-1020 states that the best-estimate non-seismic demand, D_{NS} , should be combined with the seismic demand. Since D_{NS} is a best estimate, $F_{NS} = 1.0$, i.e., there is no conservatism introduced. The variability of non-seismic demand is expected to be reasonably low, i.e., β_{NS} is expected to be less than about 0.20. Thus:

$$\begin{aligned} F_{NS} &= 1.0 \\ \beta_{NS} &= 0.20 \end{aligned} \quad (30)$$

However, because of a high degree of uncertainty on β_{NS} , results will also be presented for $\beta_{NS} = 0.40$ to show the lack of sensitivity of the conclusions to β_{NS} .

4.2.4 Capacity

Past seismic probabilistic risk assessment studies indicate that the capacity variability logarithmic standard deviation β_C is typically about 0.20. The conservatism in the capacity factors based on the minimum strengths specified in the design codes is substantial and increases with increasing β_C . In order to avoid low-ductility failure modes, the median factor of safety F_C for such modes is much greater than for ductile failure modes.

Based upon a review of median capacities from past seismic probabilistic risk assessment studies versus code specified ultimate capacities for a number of failure modes, it is judged that for ductile failure modes when the conservatism of material strengths, code capacity equations, and seismic strain-rate effects are considered, the code capacities have at least a 98% probability of exceedance. For low ductility failure modes, an additional factor of conservatism of about 1.33 is typically introduced. Thus:

<u>Ductile</u>	<u>Low Ductility</u>
$F_C = e^{2.054\beta_C}$	$F_C = 1.33e^{2.054\beta_C}$
$F_C = 1.5$	$F_C = 2.0$
$\beta_C = 0.2$	$\beta_C = 0.2$

(31)

The following low-ductility example of a longitudinal shear failure of a fillet weld connection is illustrative of the evaluations which have led to the estimates given in Equation (31). Note that the transverse shear capacity of a fillet weld exceeds the longitudinal shear capacity, yet the code capacity is the same in both directions. Therefore, basing this example on a longitudinal shear failure mode produces a lower estimated capacity factor of safety F_C than for a transverse shear failure mode.

Based upon extensive testing of fillet welds under longitudinal shear reported in Reference 9 and 12, the median shear strength, $\hat{\tau}_W$, of the fillet weld can be defined in terms of the median ultimate strength, $\hat{\sigma}_U$, of the electrode by:

$$\hat{\tau} = 0.84\hat{\sigma}_U \quad (32)$$

with an equation logarithmic standard deviation, β_{EQN} , of 0.11. The median ultimate strength is defined in terms of the minimum code nominal tensile strength, F_{EXX} , by:

$$\hat{\sigma} = 1.1F_{EXX} \quad (33)$$

with a logarithmic standard deviation, β_{MAT} , of 0.05. In addition, a logarithmic standard deviation, β_{FAB} , of 0.15 due to fabrication tolerances should be considered for normal welding practice. The code shear capacity τ_c specified in AISC-LRFD (Ref. 10) for the limit state strength approach for design is:

$$\tau_c = 0.75(0.6)F_{EXX} \quad (34)$$

Thus the median capacity factor of safety F_C is :

$$F_C = \frac{\hat{\tau}_w}{\tau_c} \frac{1.1(0.84)}{0.75(0.6)} = 2.05 \quad (35)$$

with the capacity logarithmic standard deviation, β_C , estimated to be:

$$\begin{aligned} \beta_C &= (\beta_{EQN}^2 + \beta_{MAT}^2 + \beta_{FAB}^2)^{1/2} \\ &= [(0.11)^2 + (0.05)^2 + (0.15)^2]^{1/2} = 0.19 \end{aligned}$$

Many other ductile and low-ductility failure mode capacity examples which also support the reasonableness of the estimates presented in Equation (31) are available in reported seismic probabilistic risk assessment studies such as Reference 7 and in Reference 13.

4.2.5 Inelastic Energy Absorption Factor

Based upon the seismic demand conservation estimated in Equation (29) of Subsection 4.2.2 and the capacity conservatism estimated in Equation (31) of Subsection 4.2.5, it has been found that to obtain a ratio R_{FS} for obtained to required median factors of safety of about one or more, the inelastic energy absorption factor, F_{μ} should be defined by:

$$F_{\mu} = F_{\mu 5\%} \quad (37)$$

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. The adequacy of Equation (37) will be illustrated in the next subsection.

With the inelastic factored seismic demand, D_{SI} , defined by Equation (18) and F_{μ} defined by Equation (37), the resulting median inelastic factor of safety, F_I is:

$$F_I = SF \left(\frac{F_{\mu 50\%}}{F_{\mu 5\%}} \right) = SF e^{1.65 \beta_I} \quad (38)$$

The inelastic variability logarithmic standard deviation β_I will increase with increasing $F_{\mu 5\%}$. For a low-ductility failure mode where $F_{\mu 5\%}$ is conservatively specified to be 1.0, in order to be consistent β_I must be set to zero since F_{μ} cannot drop below 1.0. However, for a ductile failure mode for which $F_{\mu 5\%} = 1.75$, β_I is estimated to be about 0.20. This estimate corresponds to a median $F_{\mu 50\%} = 2.4$ and a 1% failure probability estimate of $F_{\mu 1\%} = 1.5$ which are reasonable for $F_{\mu 5\%} = 1.75$. For this demonstration, both ductile and low-ductility failure modes will be investigated with the following F_I and β_I factors being used:

		<u>Low Ductility Case</u>	<u>Ductile Case</u>
$F_{\mu 5\%}$	=	1.0	1.75
β_I	=	0	0.20
F_I	=	SF	1.4 SF

4.2.6 Comparison of Seismic Criteria Factor of Safety with Required Factor of Safety

The individual median factors of conservatism F_R , F_{NS} , F_C and F_I and corresponding logarithmic standard deviations estimated in Sections 4.2.2 through 4.2.5 are summarized in Table 4-1. Using these estimates, the seismic criteria factor of safety F_S (from Equation 23) and the required factor of safety F_{SRqd} (from Equation 26) are shown in Tables 4-2 and 4-3 for the low-ductility and ductile failure cases, respectively, for (D_{NS}/C_C) from 0 to 0.6. In order to satisfy non-seismic load combinations and acceptance criteria, the expected non-seismic demand D_{NS} should not exceed 60% of the code strength capacity C_C . Therefore, Tables 4-2 and 4-3 cover the full expected range of (D_{NS}/C_C) . Both the required safety factor, F_{SRqd} , and F_I used in Equation 22 to define the achieved safety factor F_S are proportional to the seismic scale factor SF . Therefore, SF may be dropped out of the comparisons. Tables 4-2 and 4-3 are for $SF = 1.0$, but the resulting ratio R_{FS} of F_S to F_{SRqd} is also applicable at other seismic scale factors.

For the ductile failure mode (Table 4-3), the achieved factor of safety and required factor of safety are in close agreement over the entire range of (D_{NS}/C_C) . Similar close agreement exists for the low-ductility failure mode (Table 4-2) up to a (D_{NS}/C_C) value of 0.4. For (D_{NS}/C_C) values beyond 0.4 and low-ductility failure modes, the seismic criteria become more conservative than desired. However, the conservatism cannot be removed without becoming nonconservative in other cases if simple deterministic seismic criteria are to be maintained.

In order to study the sensitivity of these conclusions to the assumed value of $\beta_{NS} = 0.20$ shown in Table 4-1, the low ductility and ductile failure mode cases shown in Tables 4-2 and 4-3, respectively, were repeated for $\beta_{NS} = 0.40$ with all other parameters held at the values shown in Table 4-1. The achieved safety factors F_S shown in tables 4-2 and 4-3 are not influenced by β_{NS} so that they remain unchanged. At $(D_{NS}/C_C) = 0$, the required safety factors F_{SRqd} are also not influenced by β_{NS} so that they also remain unchanged. The largest change occurs for F_{SRqd} at $(D_{NS}/C_C) = 0.6$. At this value, for the ductility failure mode, F_{SRqd} is increased to 2.67 for $\beta_{NS} = 0.4$ versus 2.58 shown in Table 4-2 for $\beta_{NC} = 0.2$. Similarly, for the ductile failure mode, F_{SRqd} is increased to 3.07 versus 2.88 shown in Table 4-3. In both cases, F_{SRqd} remains below the achieved safety factor F_S and the conclusions of the previous paragraph remain unaltered. In fact, the agreement between F_S and F_{SRqd} is improved over the entire range of (D_{NS}/C_C) ratios. Therefore, even when the non-seismic demand is highly uncertain, only the best-

estimate (no intentional conservatism) non-seismic demand should be combined with the seismic demand.

Thus, the deterministic seismic acceptance criteria defined in DOE-STD-1020 for PC 3 and higher categories either achieves or exceeds the required degree of conservatism defined by Equation (13a).

4.3 Minimum Required Response Spectrum for Equipment Qualified by Test

For equipment qualified by test, the minimum ratio of the required response spectrum (RRS) to the in-structure response spectrum (IRS) at the component attachment point needed to achieve the seismic margin specified by Equations (13a) and (13b) is defined by:

$$(RRS/IRS) = \frac{1.5 SF e^{1.282\beta_{FS}}}{F_R F_C} \quad (40)$$

where β_{FS} is defined by Equation 24 and F_R and F_C are defined in Equation (22). Estimates of the median response factor of safety F_R and variability for equipment are presented in Equation (29).

An estimate of the median capacity factor of safety F_C is impossible to make for equipment qualified by test. All that can be estimated from such a test is a lower bound on F_C , and even this estimate is difficult. Standard test procedures use broader frequency content and longer duration input than is likely from an actual earthquake; and to pass the test, the equipment must function during and after such input. Therefore, F_C must substantially exceed unity. However, such tests do not typically address the possible sample-to-sample variability in the seismic capacity of the tested equipment, because it is typical to test three or fewer samples of a component. Based upon Appendices J and Q of Reference 13, it is judged that such qualification testing provides somewhere between 90% and 98% confidence of acceptable equipment performance at the RRS level, or failure probabilities for equipment that passed such a test between 2% and 10%. Thus:

$$F_C \geq e^{X_P \beta_C} \quad (41)$$

Where X_P is the standard normal distribution factor associated with an assumed failure probability. Based upon a review of fragility results presented in Bandyopadhyay et al. (Reference 14), β_C is estimated to be about 0.20 for equipment qualified by test.

For equipment qualified by test:

$$\beta_{FS} = (\beta_R^2 + \beta_C^2)^{1/2} = ((0.35)^2 + (0.20)^2)^{1/2} = 0.40 \quad (42)$$

Thus, from Equation (40) with $F_R = 1.28$ from Equation (29):

Assumed Failure Probability P	X_P	Lower Bound F_C (Eq. 41)	(RRS/IRS)/SF
2%	2.054	1.5	1.3
5%	1.645	1.4	1.4
10%	1.282	1.3	1.5

Using the midpoint value within this range:

$$(RRS / IRS) = 1.4 \text{ SF} \quad (43)$$

and:

R_R	Constant SF*	(RRS / IRS)*
20	1.6	2.25
10	1.25	1.75
5	1.0	1.4

* Improved estimates of SF as a function of A_R can be obtained from Figure 2-3 and these improved SF estimates will change the required (RRS/IRS) ratio.

Table 4-1

Estimated Factors of Conservatism and Variability

Factor	Low Ductility Mode $F_{\mu 5\%} = 1.0$	Ductile Mode $F_{\mu 5\%} = 1.75$
Seismic Demand (Response)		
F_R	1.2	1.2
β_R	0.3	0.3
Non-Seismic Demand		
F_{NS}	1.0	1.0
β_{NS}	0.2	0.2
Capacity		
F_C	2.0	1.5
β_C	0.2	0.2
Inelastic Energy Absorption		
F_I	SF	1.4SF
β_I	0	0.2

Table 4-2

**Comparison of Achieved Safety Factor to Required
Safety Factor for Low-Ductility Failure Mode**
($F_{\mu 5\%} = 1.0$; $SF = 1.0$)

D_{NS} / C_C	β_{CS}	β_{FS}	Required Safety Factor F_{SRqd}	Achieved Safety Factor F_S	R_{FS} $= F_S / F_{SRqd}$
0	0.20	0.36	2.38	2.40	1.01
0.1	0.21	0.37	2.40	2.53	1.05
0.2	0.22	0.37	2.42	2.70	1.12
0.3	0.24	0.38	2.45	2.91	1.19
0.4	0.25	0.39	2.48	3.20	1.29
0.5	0.27	0.41	2.53	3.60	1.42
0.6	0.30	0.42	2.58	4.20	1.63

Table 4-3

**Comparison of Achieved Safety Factor to Required
Safety Factor for Ductile Failure Mode**
($F_{\mu 5\%} = 1.75$; $SF = 1.0$)

D_{NS} / C_C	β_{CS}	β_{FS}	Required Safety Factor F_{SRqd}	Achieved Safety Factor F_S	R_{FS} $= F_S / F_{SRqd}$
0	0.20	0.41	2.54	2.52	0.99
0.1	0.21	0.42	2.57	2.61	1.02
0.2	0.23	0.43	2.60	2.73	1.05
0.3	0.25	0.44	2.64	2.88	1.09
0.4	0.28	0.46	2.70	3.08	1.14
0.5	0.32	0.48	2.77	3.36	1.21
0.6	0.36	0.51	2.88	4.78	1.31

5. Establishment of F_μ Values in DOE-STD-1020

The F_μ values given in DOE-STD-1020 (reproduced herein as Table 5-1) are based on:

1. An estimate of the inelastic energy absorption factor associated with a level of inelastic distortions corresponding to about a 5% failure probability level for typical structures of the type being considered.
2. The high risk μ values given in the DOD essential facilities seismic provisions (Reference 15).
3. Values of F_μ back-computed from the R_W values given in the *Uniform Building Code* (Reference 5).

In general, the F_μ values back-computed from the *Uniform Building Code* R_W values were given the greatest weight. Therefore, the basis for this back-computation will be described herein.

Performance Category (PC) 2 and 3 SSC differ in both their performance goal descriptions (See Table 1-1) and required risk reduction in R_R ratio (See Table 1-2). These two differences must be considered when converting the R_W value from the *Uniform Building Code* (UBC) to an F_μ value.

The UBC based seismic demand for PC 2 can be approximated by:

$$D_2 = (LF) (I_2) (DBE)_2 (DAF_{5\%}) (W) / R_W \quad (44)$$

Where

$$LF = \text{UBC Seismic Load Factor}$$

$$I_2 = \text{Importance Factor for PC 2} = 1.25$$

DAF_{5%} = Dynamic amplification factor from the 5 percent ground response spectrum at the natural period of the facility

W = Total weight of the facility

R_W = Reduction coefficient accounting for available energy absorption (Reference 5)

Expressed in a similar manner, the PC 3 required seismic demand can be approximated as:

$$D_3 = SF_3 (DBE)_3 (m) (DAF_{5\%}) (W) / F_\mu \quad (45)$$

Where:

m = A factor accounting for the difference in spectral amplification from 5 percent to the damping appropriate for the facility in accordance with DOE-STD-1020 e.g.,

m = 0.9 for 7 percent damping

m = 0.8 for 10 percent damping

m = 0.7 for 15 percent damping

F_μ = Inelastic energy absorption factor

SF₃ = Scale factor for PC 3

However, D_3/DBE_3 must exceed D_2/DBE_2 by the ratios of the required factor of safety in Table 2-3 for PC 3 (1.5 SF₃) versus PC 2 (1.0) and by the ratio of PC 3 to PC 2 importance factors (I_3 / I_2) required because of the more stringent performance goal description for PC 3. Thus:

$$\frac{D_3/DBE_3}{D_2/DBE_2} = \left(\frac{F_{R3}}{F_{R2}} \right) \left(\frac{I_3}{I_2} \right) \quad (46)$$

$$\left(\frac{F_{R3}}{F_{R2}}\right) = 1.5 SF_3 \quad (47)$$

and from judgment:

$$\left(\frac{I_3}{I_2}\right) = 1.33 \quad (48)$$

Combining Equation (44), (45), and (46):

$$F_{\mu} = \frac{SF_3 m R_W}{\left(\frac{I_3}{I_2}\right) \left(\frac{F_{R3}}{F_{R2}}\right) (LF) (I_2)} \quad (49)$$

Substituting in the appropriate values of $I_2 = 1.25$, $\left(\frac{I_3}{I_2}\right) = 1.33$, and

$$\left(\frac{F_{R3}}{F_{R2}}\right) = 1.5 SF_3$$

$$F_{\mu}/R_W = \frac{m}{2.5(LF)} \quad (50)$$

where:

m	=	0.9 for steel (7% damping)
m	=	0.8 for concrete (10% damping)
m	=	0.75 for masonry (12% damping)
m	=	0.7 for wood (15% damping)
LF	=	1.3 for steel
LF	=	1.4 for concrete and masonry
LF	=	1.5 for wood

Values of inelastic demand-capacity ratio, F_{μ} , from Equation (50) along with values from the DOD essential facilities seismic provisions (Ref. 15), are presented in Table 5-2 for many structural systems, materials, and construction. Note that these values are used differently in that the F_{μ} value in DOE-STD-1020 is applied to response due to seismic loads only; while, by Reference 15, the inelastic demand-capacity ratio is applied to response due to total load. Thus, the Ref. 15 results must be somewhat increased to be comparable.

The inelastic demand-capacity ratios from Equation (50) are based on the structural systems for which reduction coefficients, R_W , are given in the UBC provisions. These provisions give different reduction coefficients for bearing wall systems and for building frame systems in which gravity loads are carried by structural members that are different from the lateral force resisting system. In addition, the UBC provisions distinguish between different levels of detailing for moment resisting space frames, between eccentric and concentric braced frames, and between single and dual lateral load resisting systems. Consequently, Equation (50) results in more inelastic demand-capacity ratios than Reference 15, which does not make the above distinctions. On the other hand, DOD provisions give different inelastic demand-capacity ratios for individual members of the lateral load-resisting system, while UBC reduction coefficients refer to all members of the lateral load resisting system.

In general, there is reasonable agreement between the inelastic demand-capacity ratios from Reference 15 and those computed from Equation (50). For example, the DOD inelastic demand-capacity ratio for concrete shear walls is between the values for bearing and non-bearing walls from the equations. The DOD values are much lower than the values computed when shear walls act as a dual system with ductile moment-resisting space frames to resist seismic loads. The inelastic demand-capacity ratios for braced frames agree fairly well when the bracing carries no gravity loads. When bracing carries gravity loads, values for steel braced frames are in good agreement, but based on Equation (50), no inelastic behavior would be permitted for concrete braced frames or wood trusses. The DOD inelastic demand-capacity ratio for beams in a ductile moment-resisting frame fall between values from the equations for special and intermediate moment-resisting space frames (SMRF and IMRF as defined in Reference 5). However, the DOD values for columns are low compared to values derived from the code reduction coefficients.

Based upon the data presented in Table 5-2, the inelastic demand-capacity ratios for seismic design and analysis of PC 3 or higher SSC presented in Table 5-1 have been

selected. Because of the reasonable agreement with the DOD values from Reference 15 combined with the capability to distinguish between a greater number of structural systems, the values derived from Equation (50) have been given somewhat more weight for Table 5-1 than Reference 15 values. The only major exception is that Reference 15 values for columns have been utilized. Increased conservatism for columns as recommended in the DOD manual is retained. In addition, Reference 15 provides slightly different values for different members making up braced frames, and these differences are retained.

Table 5-1

Code Reduction Coefficients, R_w and Inelastic Demand Capacity Ratios, F_μ

Structural System (terminology is identical to Ref. 1)	R_w	F_μ
MOMENT RESISTING FRAME SYSTEMS - Beams		
Steel Special Moment Resisting Space Frame (SMRF)	12	3.0
Concrete SMRF	12	2.75
Concrete Intermediate Moment Frame (IMRF)	8	1.5
Steel Ordinary Moment Resisting Space Frame	6	1.5
Concrete Ordinary Moment Resisting Space Frame	5	1.25
SHEAR WALLS		
Concrete or Masonry Walls	8 (6)	
In-plane Flexure		1.75
In-plane Shear		1.5
Out-of-plane Flexure		1.75
Out-of-plane Shear		1.0
Plywood Walls	9 (8)	1.75
Dual System, Concrete with SMRF	12	2.5
Dual System, Concrete with Concrete IMRF	9	2.0
Dual System, Masonry with SMRF	8	1.5
Dual System, Masonry with Concrete IMRF	7	1.4
STEEL ECCENTRIC BRACED FRAMES (EBF)		
Beams and Diagonal Braces	10	2.75
Beams and Diagonal Braces, Dual System with Steel SMRF	12	3.0
CONCENTRIC BRACED FRAMES		
Steel Beams	8 (6)	2.0
Steel Diagonal Braces	8 (6)	1.75
Concrete Beams	8 (4)	1.75
Concrete Diagonal Braces	8 (4)	1.5
Wood Trusses	8 (4)	1.75
Beams and Diagonal Braces, Dual Systems		
Steel with Steel SMRF	10	2.75
Concrete with Concrete SMRF	9	2.0
Concrete with Concrete IMRF	6	1.4
METAL LIQUID STORAGE TANKS		
Moment and Shear Capacity		1.25
Hoop Capacity		1.5

Note: Values herein assume good seismic detailing practice along with reasonably uniform inelastic behavior. Otherwise, lower values should be used. Values in parentheses apply to bearing wall systems or systems in which bracing carries gravity loads.

F_μ for columns of all structural systems is 1.0 for axial compression and 1.5 for flexure.

Connections for steel concentric braced frames should be designed for the lesser of:

the tensile strength of the bracing

the force in the brace corresponding to F_μ of unity.

the maximum force that can be transferred to the brace by the structural system

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

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

Table 5-2

Inelastic Energy Absorption Factors from Equation 50 and Reference 15

Structural System	R_w	R15	Eqn(50)
MOMENT RESISTING FRAME SYSTEMS			
Columns	*	1.5	*
Beams			
Steel Special Moment Resisting Frame (SMRF)	12	2.5	3.32
Concrete SMRF	12	2.5	2.74
Concrete Intermediate Moment Frame (IMRF)	8	-	1.60
Steel Ordinary Moment Resisting Frame	6	-	1.66
Concrete Ordinary Moment Resisting Frame	5	-	1.14
SHEAR WALLS			
Concrete Bearing Walls	6	1.5	1.37
Concrete Non-Bearing Walls	8	1.5	1.83
Masonry Bearing Walls	6	1.25	1.29
Masonry Non-Bearing Walls	8	1.25	1.71
Plywood Bearing Walls	8	2.5	1.49
Plywood Non-Bearing Walls	9	2.5	1.68
Dual System, Concrete with SMRF	12	1.5	2.74
Dual System, Concrete with Concrete IMRF	9	1.5	2.06
Dual System, Masonry with SMRF	8	1.25	1.71
Dual System, Masonry with Concrete IMRF	7	1.25	1.50
CONCENTRIC BRACED FRAMES (Bracing Carries Gravity Loads)			
Steel Beams	6	1.75	1.66
Steel Diagonal Braces	6	1.5	1.66
Steel Columns	6	1.5	1.66
Connections of Steel Members	6	1.25	1.66
Concrete Beams	4	1.75	<1
Concrete Diagonal Braces	4	1.5	<1
Concrete Columns	4	1.5	<1
Connections of Concrete Members	4	1.25	<1
Wood Trusses	4	1.75	<1
Wood Columns	4	1.5	<1
Connections in Wood (other than nails)		1.5	<1
CONCENTRIC BRACED FRAMES (No Gravity Loads)			
Steel Beams	8	1.75	2.22
Steel Diagonal Braces	8	1.5	2.22
Steel Columns	8	1.5	2.22
Connections of Steel Members	8	1.25	2.22
Concrete Beams	8	1.75	1.83
Concrete Diagonal Braces	8	1.5	1.83
Concrete Columns	8	1.5	1.83
Connections of Concrete Members	8	1.25	1.83
Wood Trusses	8	1.75	1.49
Wood Columns	8	1.5	1.49
Connections in Wood (other than nails)	8	1.5	1.49
Beams and Diagonal Braces, Dual Systems			
Steel with Steel SMRF	10	-	2.77
Concrete with Concrete SMRF	9	-	2.06
Concrete with Concrete IMRF	6	-	1.37
STEEL ECCENTRIC BRACED FRAMES (EBF)			
Columns	*	1.5	*
Beams and Diagonal Braces	10	-	2.77
Beams and Diagonal Braces, Dual System with Steel SMRF	12	-	3.32

Note: R15 values are inelastic energy absorption factors from Ref. 15. Eqn (50) values are inelastic energy absorption factors calculated from Eq. 50.

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

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Appendix A

Seismic Hazard Curve Slopes

At most DOE sites, modern seismic hazard assessments have not yet been completed. However, the Nuclear Regulatory Commission has recently issued draft NUREG-1488 (Ref. 16) which gives seismic hazard curves for peak ground acceleration at 69 nuclear power plant sites across the eastern United States. NUREG-1488 provides seismic hazard curves based on the most up to date methodology by Lawrence Livermore National Laboratory (LLNL). In addition, seismic hazard curves are available at these eastern U.S. nuclear power plant sites from the Electric Power Research Institute (EPRI). A_R values from NUREG-1488 (New LLNL) mean seismic hazard curves and from EPRI mean seismic hazard curves are presented in Table A-1. A_R values are given for the decades in probability from 1×10^{-4} to 1×10^{-5} (A_5/A_4) and from 1×10^{-3} to 1×10^{-4} (A_4/A_3) in this table.

Averages and standard deviations for these A_R values are given at the bottom of Table A-1. Statistical values are given both for all sites and for those sites where ground motion is 0.1g or more. The shaded areas in the table are the places where the peak ground acceleration at a probability of 1×10^{-4} is less than 0.1g. There was little effect of eliminating the very low ground motion sites. The statistical values demonstrate that A_R values fall in the range of about 2.5 to 4 using new LLNL methodology and of about 2 to 6 using EPRI methodology over the probability range of 1×10^{-3} to 1×10^{-5} . It is expected that seismic hazard curves for DOE sites in the eastern United States will have A_R values in the range of those indicated in Table A-1 if modern seismic hazard assessment methodology is followed.

These A_R values are illustrated in Figure A-1 for both new LLNL and EPRI methods. Figure A-1a demonstrates that A_R values from new LLNL methodology fall in the range of 2.5 to 4 with values in the 1×10^{-3} to 1×10^{-4} generally between 3.5 and 4 and values in the 1×10^{-4} to 1×10^{-5} range generally between 2.5 and 3.5. Figure A-1b demonstrates that A_R values from EPRI methodology fall in the range of 2 to 6 with values in the 1×10^{-3} to 1×10^{-4} generally between 3 and 6 and values in the 1×10^{-4} to 1×10^{-5} range generally between 2 and 3. The conclusion from Table A-1 and Figure A-1 is that in lower seismic locations such as these eastern U.S. nuclear power plant sites, the slope coefficients, A_R , from modern technology range from 2 to 6, but most slope coefficients are in the range from 2 to 4.

Table A-1 A_p Values from Modern Seismic Hazard Curves for the Eastern U.S.

Plant	New LLNL		EPRI		Plant	New LLNL		EPRI	
	A5/A4	A4/A3	A5/A4	A4/A3		A5/A4	A4/A3	A5/A4	A4/A3
1 Arkansas	2.62	3.66	2.38	3.05	38 Nine Mile Point	2.65		2.48	3.28
2 Beaver Valley	3.23		2.75	4.73	39 North Anna	2.45	3.92	2.30	3.18
3 Bellefonte	2.51	3.85	2.43	3.39	40 Oconee	2.51	3.66	2.35	3.21
4 Big Rock Point	3.99				41 Oyster Creek	3.00		2.04	3.75
5 Braidwood	2.94		2.77	4.34	42 Palisades	3.61			
6 Brown Ferry	2.65		2.48	3.21	43 Peach Bottom	2.57	4.07	2.65	4.56
7 Brunswick	3.02	3.70	2.38	4.36	44 Perry	2.94		2.89	5.34
8 Byron	2.85		2.77	5.76	45 Pilgrim		4.16	1.74	3.13
9 Callaway	2.42	2.87			46 Point Beach	3.81		3.42	6.24
10 Calvert Cliffs	3.11		2.04	3.89	47 Prairie Island	3.97		3.71	4.48
11 Catawba	2.55	3.58	2.52	3.49	48 Quad Cities	3.02		2.76	5.34
12 Clinton	2.85	3.69	1.87	3.13	49 River Bend	3.67		3.45	5.71
13 Comanche Peak	3.35		3.45	3.98	50 Robinson		3.76	2.06	3.38
14 Cook	3.38				51 Salem	2.95		2.08	4.08
15 Cooper	3.10	5.23			52 Seabrook	2.34	3.46	2.15	2.94
16 Crystal River	3.84		4.03	5.68	53 Sequoyah	2.47	3.82	2.30	3.32
17 Davis Besse	2.52	3.59	2.54	4.15	54 Shearon Harris	2.87		2.68	3.68
18 Dresden	2.95		2.80	4.57	55 Shoreham	2.93	4.63		
19 Duane Arnold	3.46				56 South Texas	4.01		4.57	
20 Farley	3.52		3.17	4.61	57 St. Lucie	4.24			
21 Fermi	2.74		2.82	4.61	58 Summer	2.47	3.36	2.51	4.43
22 Fitzpatrick	2.64		2.48	3.28	59 Surry	3.12		2.00	3.72
23 Ft. Calhoun	3.28				60 Susquehanna	2.67		2.58	4.27
24 Ginna	2.71		2.54	4.07	61 Three Mile Island	2.59	4.04	2.72	4.78
25 Grand Gulf	3.38				62 Turkey Point	4.05			
26 Haddam Neck	2.60	4.04	2.23	3.38	63 Vermont Yankee	2.62	3.56	2.35	2.85
27 Hatch	3.18		2.17	4.34	64 Vogtle	2.69	3.44	2.04	3.20
28 Hope Creek	2.94		2.08	4.08	65 Waterford	3.61		4.05	5.33
29 Indian Point	2.58	3.97	2.46	3.63	66 Watts Bar	2.50	3.87	2.25	3.11
30 Kewaunee			3.38	5.88	67 Wolf Creek	3.02		2.75	4.89
31 Lacrosse	3.93				68 Yankee Rowe	2.79	3.76	2.02	3.14
32 LaSalle	3.20		2.30	4.79	69 Zion	3.33		2.40	6.24
33 Limerick	2.53	3.95	2.51	3.97	average	3.02	3.81	2.61	4.08
34 Maine Yankee	2.60	3.72	2.38	3.20	std. dev.	0.50	0.43	0.56	1.02
35 McGuire	2.52	3.52	2.44	3.21	average (all > 0.1g)	3.02	3.81	2.47	4.04
36 Millstone	2.63		2.35	3.28	std. dev. (all > 0.1g)	0.50	0.43	0.39	0.92
37 Monticello	3.83		3.11	5.91					

Shaded areas have PGA at 1×10^{-4} less than 0.1g

a. NUREG-1488 or New LLNL Methodology

b. EPRI Methodology

Figure A-1 A_R Values for Eastern U.S. Power Plant Sites

There are modern seismic hazard curves available at a few DOE sites. Slope coefficients, A_R for such sites are presented in Table A-2. In addition, slope coefficients, A_R from modern seismic hazard assessments at several sites in California, a high seismic region, are shown on the table. A_R values for eastern U.S. DOE sites such as Oak Ridge, Paducah, Portsmouth, and Savannah River range from about 2 to 3.5. A_R values for western U.S. DOE sites such as INEL, LANL, and Hanford range from about 1.75 to 3. These are western sites where the seismicity is not governed by faults associated with tectonic plate boundaries. A_R values for California sites (LLNL and other locations) range from about 1.5 to 2.25, which illustrates

that high seismic western sites with seismicity governed by tectonic plate boundaries exhibit lower slope coefficients. The data presented in Table A-2 provide a further demonstration that A_R values can vary over a wide range. This data also demonstrates that the range for A_R extends down to about 1.5 for high seismic sites.

Table A-2 A_R Values from Modern Seismic Hazard Curves at Several Sites

Eastern DOE Sites	A5/A4	A4/A3
Oak Ridge (EPRI)	2.26	3.18
Paducah (Rock, EPRI)	2.05	2.59
Portsmouth (Rock, EPRI)	2.56	3.44
Savannah River (1993, LLNL New)	2.23	2.38
Western DOE Sites (not near tectonic plate boundaries)	A5/A4	A4/A3
INEL (WCC, 1993)	1.99	2.38
LANL (WCC, 1993)	1.78	2.14
Hanford (Geomatrix, 1993 Draft)	2.01	2.78
High Seismic Sites (near tectonic plate boundaries)	A5/A4	A4/A3
LLNL (Geomatrix, 1990)	1.55	1.65
Diablo Canyon Nuclear Power Plant (PG&E, 1988)	1.36	1.68
SF Bay Bridge (Geomatrix, 1993)		1.57
Dumbarton Bridge (Geomatrix, 1993)		1.53
Benicia - Martinez Bridge (Geomatrix, 1993)		1.70
L.A. Vincent Thomas Bridge (WCC, 1993)		2.12
S.D. Coronado Bay Bridge (WCC, 1993)		2.28

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