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BASIS FOR SEISMIC PROVISIONS

OF UCRL-15910

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1. Introduction

UCRL-15910 (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 presents the seismic annual probability performance goals for PC #1 through #4 for which specific seismic design and evaluation criteria are presented in UCRL-15910. In addition, UCRL-15910 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. These 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 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, i.e.:

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

The required degree of conserv[illegible] in the deterministic acceptance criteria is a function of the specified risk [illegible]on 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.,:

$$P_H = R_R(P_F) \tag{2}$$

Table 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. Sufficient conservation 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 conservation 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.

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

2.1 Derivation

Figure 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 western higher seismicity site. Curve B represents a typical hazard estimate for an eastern lower seismicity site.

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

$$H_{(a)} = K_1 a^{-k_H} \qquad (3)$$

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:

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

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 1 comes the ground motion ratios and hazard slope parameters shown in Table 3. These results are typical. For western higher seismicity sites, the A_R ratios for mean hazard curves will range from about 2.0 to as low as about 1.5 within the probability range from 10^{-3} to 10^{-5}. For eastern lower seismicity sites, the corresponding A_R ratios will range from about 2.0 to as high as about 3.75. 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 standard deviation will generally lie within the range of 0.3 to 0.5.

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:

$$P_F = -\int_0^{+\infty} \left(\frac{d\,H_{(a)}}{da} \right) P_{F/a}\, da \qquad 5a$$

or

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

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:

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

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

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

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

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

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

or from the previous definitions of M:

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

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

$$K_1 = H_D [DBE]^{K_H} \qquad 5g$$

from which:

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

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.

In order to accommodate ground motion, A_R, ratios ranging from 1.5 to 3.75 while achieving a given required risk reduction ratio R_R by a specified degree of seismic design conservatism, it has been found that the DBE must be defined as the larger of:

$$DBE \geq a_{PH} \tag{6a}$$

$$DBE \geq f_a a_{PF} \tag{6b}$$

where a_{PH} and a_{PF} are the mean ground motions at the seismic hazard probability, P_H, and probability performance goal, P_F, respectively, and f_a is an empirically derived factor to enable the required risk reduction ratio, R_R to be approximately achieved over the wide range of A_R values. When the DBE is defined by Equation (6a):

$$H_D = P_H \qquad 7a$$

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:

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

Alternatively, when the DBE is defined by Equation (6b):

$$H_D = P_F \left[\frac{DBE}{a_{PF}}\right]^{-K_H} = P_F (f_a)^{-K_H} \qquad 7b$$

Substituting Equation (7b) into Equation (5) leads to:

$$(f_a)^{-K_H} = (C_{50} / DBE)^{K_H} e^{-1/2(K_H \beta)^2} \qquad 8b$$

The required ratio (C_{50}/DBE) is given by Equation (8a) when Equation (7a) produces a lower exceedance probability, H_D, than does Equation (7b). When (7b) produces the lower, H_D, then (C_{50}/DBE) is controlled by Equation (8b).

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

$$C_P = F_{PR}(DBE) \tag{9}$$

where F_{PR} 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:

$$C_{50\%} = C_p e^{X_p \beta} \tag{10}$$

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 either Equation (6a) or (8b) whichever controls with Equations (9) and (10), the required safety factor, F_{PR} is:

$$F_{PR} = \frac{NUM}{DEN} \tag{11}$$

$$NUM = \text{smaller of } \left[R_R^{1/K_H} \text{ or } 1/fa \right] \tag{12}$$

$$DEN = e^{[x_p\beta - (K_H/2)\beta^2]} \tag{13}$$

Tables 4a, 4b, 4c and 4d present the maximum and minimum values of F_{PR} within the range of $1.5 \leq A_R \leq 3.75$ and $0.3 \leq \beta \leq 0.5$ required to achieve R_R ratios of 20, 10, 5 and 2, respectively, for various failure probability "P" and f_a values. Also presented in these tables are the ratios of maximum to minimum F_{PR} for each condition.

As an example, for $R_R = 20$, $f_a = 0.45$, and a 10% failure probability, the maximum F_{PR} occurs when $A_R = 1.847$ (i.e., $K_H = 3.752$) and $\beta = 0.5$. Thus, NUM = 2.222 from Equation (12) and DEN = 1.188 from Equation (13) which leads to the maximum F_{PR} of 1.87 shown in Table 4a for this case. Similarly, for this same case the minimum F_{PR} occurs when $A_R = 3.75$ (i.e., $K_H = 1.742$) and $\beta = 0.5$ for which NUM = 2.222, DEN = 1.527 and the minimum $F_{PR} = 1.46$ also shown in Table 4a.

As another example, for $R_R = 20$, $f_a = 0.5$, and a 20% failure probability, the minimum F_{PR} occurs when $A_R = 3.75$ (i.e., $K_H = 1.742$) and $\beta = 0.4833$ for which NUM = 2.0, DEN = 1.226 and the minimum $F_{PR} = 1.63$. However, when this case is slightly changed to $R_R = 20$, $f_a = 0.45$ and a 20% failure probability, the minimum F_{PR} occurs when $A_R = 1.5$ (i.e., $K_H = 5.679$) and $\beta = 0.3$ for which NUM = 1.695, DEN = 0.997 and the minimum $F_{PR} = 1.70$.

Thus, the maximum and minimum F_{PR} values shown in Tables 4a through 4d come from differing combinations of A_R and β for the cases considered. In general, the entire sample space of $1.5 \leq A_R \leq 3.75$ and $0.3 \leq \beta \leq 0.5$ must be searched to find the maximum and minimum F_{PR} values for each case.

From Tables 4a through 4d, note that for each R_R ratio, the maximum to minimum range on F_{PR} is minimum when the 10% failure capacity is used. 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_{PR} 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.

In addition, the maximum to minimum F_{PR} ratio is also minimum for each R_R ratio when the f_a values listed in Table 5 are used to define the DBE in Equation (6b). Note for A_R ratios of 3.75 or less that for $R_R = 2$ Equation (6b) with $f_a \leq 0.65$ never controls the DBE. Therefore, for $R_R = 2$, the DBE can always be defined by Equation (6a) and f_a is not applicable.

In conclusion, it is recommended that the DBE level be defined by the larger of Equations (6a) or (6b) using f_a from Table 5, and the minimum required 10% probability of failure capacity $C_{10\%}$ be given by:

$$C_{10\%} \geq F_R \text{ (DBE)} \qquad (14)$$

Table 6 shows the required F_R values to achieve the desired R_R ratio for various A_R and β values. The required factors of safety F_R provided in Table 5 are roughly the midpoint values shown in Table 6 with greatest weight being placed are the $1.65 \leq A_R \leq 3.25$ ratios within which the vast majority of probabilistic seismic hazard curves are expected to lie. The F_R values in Table 6 range from 86% to 110% of the required F_R values listed in Table 5 except for the $R_R = 2$ case with an unlikely very steep hazard curve ($A_R = 1.5$) combined with an unlikely high logarithmic standard deviation ($\beta = 0.5$) for which F_R exceeds the value in Table 5 by 21%.

Equation (14) may be alternately written as:

$$C_{10\%} = 1.5L_S \text{ (DBE)} \qquad (15)$$

$$L_S \approx F_R / 1.5 \qquad (15a)$$

where L_S is the seismic load factor defined in Table 5 for the specified risk reduction ratio R_R. This alternate format is used in subsequent sections and in UCRL-15910 (Ref. 1).

2.2 Validation

However, actual hazard curves are not perfectly linear when plotted on a log-log scale (for example, see Figure 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 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 in Table 5 which approximate the values given in Table 6 will be shown in this section using both hazard curves A and B from Figure 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):

$$P_F = \sum_{i=1}^{\infty}\left[H_{(a_i)} - H_{(a_{i+1})}\right] P_{Fla_{cg_i}} \qquad (16)$$

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

$$acgi = \frac{\int_{ai}^{ai+1} H_{(a)} a \, da}{\int_{ai}^{ai+1} H_{(a)} \, da} \qquad (16a)$$

and $P_{Fla_{cg_i}}$ 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:

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

then Equation (16a) gives:

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

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

The hazard exceedance values versus peak ground acceleration from Figure 1 are tabulated in Table 7 for seismic hazard curves A and B. Table 8a presents for seismic hazard curve A (Figure 1) and performance categories 2, 3 and 4 the required DBE level (from Equation (6a) and (6b)), the required 10% probability of failure capacity $C_{10\%}$ (from Equation (14) and Table 5)), 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 8b presents similar results for seismic hazard curve B (Figure 1). Table 9 presents an example solution of Equation (16) for the case of seismic hazard curve B, performance category 3, and logarithmic standard deviation, ß, of 0.4 to illustrate how the resulting P_F results given in Tables 8a and 8b were obtained.

For the 18 cases presented in Tables 8a and 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.

2.3 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 Equations (6a) and (6b) where f_a is defined as a function of R_R in Table 5.

2. The required 10% probability of failure seismic capacity $C_{10\%}$ is defined by Equation (14) where F_R is also defined as a function of R_R in Table 5.

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.:

$$SDBE = F_R \ (DBE) \tag{17}$$

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 UCRL-15910 (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, UCRL-15910 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 = 10) and #4 (R_R = 20).

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 2. For $R_R = 2$, from Table 5, $F_R = 1.0$. 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 UCRL-15910 specifies that the Uniform Building Code provisions be used for the seismic evaluation of PC #1 and #2 SSC, 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

UCRL-15910 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 UCRL-15910. 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 UCRL-15919 a mean (or 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.

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 load factor, L_S, to obtain an inelastic-factored estimate of the seismic demand D_{SI}, i.e.:

$$D_{SI} = L_S \frac{D_S}{F_\mu} \qquad (18)$$

where L_S must correspond to the desired R_R ratio, as given in Table 5. The F_μ values given in UCRL-15910 are aimed at achieving a R_R ratio of 10. The L_S factor in Table 5

are used to adjust the risk reduction ratios to other values as previously shown in Section 2.1. The establishment of F_μ values is further discussed in Section 5.

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

$$D_{TI} = D_{NS} + D_{SI} \tag{19}$$

where D_{NS} represents the best-estimate of all non-seismic demands expected to occur concurrent 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.:

$$C_c \geq D_{TI} \tag{20}$$

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 UCRL-15910 is aimed toward approximately satisfying Equation (14) 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 Bench Marking Studies for Deterministic Seismic Acceptance Criteria

4.2.1 Basic Derivation

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

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

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

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

where C_C, D_{NS}, and F_μ are the capacity, non-seismic demand, seismic demand, and inelastic energy absorption factor, respectively, computed in accordance with the guidance of UCRL-19510 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:

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

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

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

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:

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

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

Based upon combining Equation (10) and (15), the required median factor of safety $F_{S_{Rqd}}$ needed to achieve the desired risk reduction ratio is:

$$F_{S_{Rqd}} = 1.5L_S e^{1.282\beta_{FS}} \tag{26}$$

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

$$R_{FS} = \frac{F_S}{F_{SRqd}} \tag{27}$$

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 UCRL-15910 the elastic-computed seismic demand is to be obtained in accordance with the requirements of ASCE 4-86, except that mean input spectral amplifications are to be used instead of mean-plus-one-standard-deviation amplifications factors. Based upon Reference 8, the ratio of mean-plus-one-standard-deviation to mean 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:

$$F_R = \frac{e^{1.282\beta_R}}{1.22} \tag{28}$$

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:

	Structures	Equipment	
F_R =	1.2	1.28	(29)
β_R =	0.3	0.35	

4.2.3 Non-Seismic Demand

The load combination criteria of UCRL-15910 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:

$$F_{NS} = 1.0$$
$$\beta_{NS} = 0.20 \qquad (30)$$

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:

Ductile	Low Ductility	
$F_C = e^{2.054\beta_c}$	$F_C = 1.33e^{2.054\beta_c}$	
$F_C = 1.5$	$F_C = 2.0$	(31)
$\beta_C = 0.2$	$\beta_C = 0.2$	

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 tr 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 13, 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:

$$\hat{\tau} = 0.84\hat{\sigma}_U \tag{32}$$

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:

$$\hat{\sigma} = 1.1F_{EXX} \tag{33}$$

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. C.2) for the limit state strength approach for design is:

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

Thus the median capacity factor of safety F_C is :

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

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

$$\beta_C = \left(\beta^2_{EQN} + \beta^2_{MAT} + \beta^2_{FAB}\right)^{1/2}$$
$$= \left[(0.11)^2 + (0.05)^2 \ (0.15)^2\right]^{1/2} = 0.19$$

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 conservation 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:

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

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:

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

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:

		Low Ductility Case	Ductile Case
$F_{\mu 5\%}$	=	1.0	1.75
β_I	=	0	0.20
F_I	=	L_S	1.4 L_S

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 10. 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 11 and 12 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 11 and 12 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 load factor L_S. Therefore, L_S may be dropped out of the comparisons. Tables 11 and 12 are for L_S = 1.0, but the resulting ratio R_{FS} of F_S to F_{SRqd} is also applicable at other seismic load factors.

For the ductile failure mode (Table 12), 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 11) 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 10, the low ductility and ductile failure mode cases shown in Tables 11 and 12, respectively, were repeated for $\beta_{NS} = 0.40$ with all other parameters held at the values shown in Table 10. The achieved safety factors F_S shown in tables 11 and 12 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 11 for $\beta_{NC} = 0.2$. Similarly, for the ductile failure mode, F_{SRqd} is increased to 3.07 versus 2.88 shown in Table 12. 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 UCRL-15910 for PC#3 and higher categories either achieves or exceeds the required degree of conservatism defined by Equation (14).

4.3 Minimum Required Ratio of TRS To RRS for Equipment Qualified by Test

For equipment qualified by test, the minimum ratio of the TRS to the RRS needed to achieve the seismic margin specified by Equation (15) is defined by:

$$(TRS\,/\,RRS) = \frac{1.5 L_S\, e^{1.282\beta_{FS}}}{F_R\; F_C} \tag{40}$$

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 TRS level, or failure probabilities for equipment that passed such a test between 2% and 10%. Thus:

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

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:

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

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

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

Using the midpoint valve within this range:

$$(TRS / RRS) = 1.4 \, L_S \qquad (43)$$

and:

R_R	L_S	(TRS / RRS)
20	1.15	1.6
10	1.0	1.4
5	0.87	1.2

5. Establishment of F_μ Values in UCRL - 15910

The F_μ values given in Table 4-11 of UCRL-15910 (reproduced herein as Table 13) 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.

Both Performance Category (PC) #2 and #3 SSC specify the seismic hazard annual exceedance probability to be 1×10^{-3}. However, these two performance categories differ in both their performance goal descriptions (See Table 1) and required risk reduction in R_R ratio (See Table 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:

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

Where

LF = UBC Seismic Load Factor

I_2 = 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:

$$D_3 = (DBE)\,(m)\,(DAF_{5\%})\,(W) / F_\mu \qquad (45)$$

Where:

m = A factor accounting for the difference in spectral amplification from 5 percent to the damping appropriate for the facility in accordance with UCRL-15910 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

However, D_3 must exceed D_2 by the ratios of the required factor of safety in Table 5 for PC#3 (R_R = 10) versus PC#2 (R_R = 2) 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:

$$\frac{D_3}{D_2} = \left(\frac{F_{R3}}{F_{R2}}\right)\left(\frac{I_3}{I_2}\right) \qquad (46)$$

From Table 5:

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

and from judgment:

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

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

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

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

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

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

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

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 14 for many structural systems, materials, and construction. Note that these values are used differently in that the F_{μ} value in UCRL-15910 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 14, the inelastic demand-capacity ratios for seismic design and analysis of PC#3 on higher SSC presented in Table 13 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 13 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.

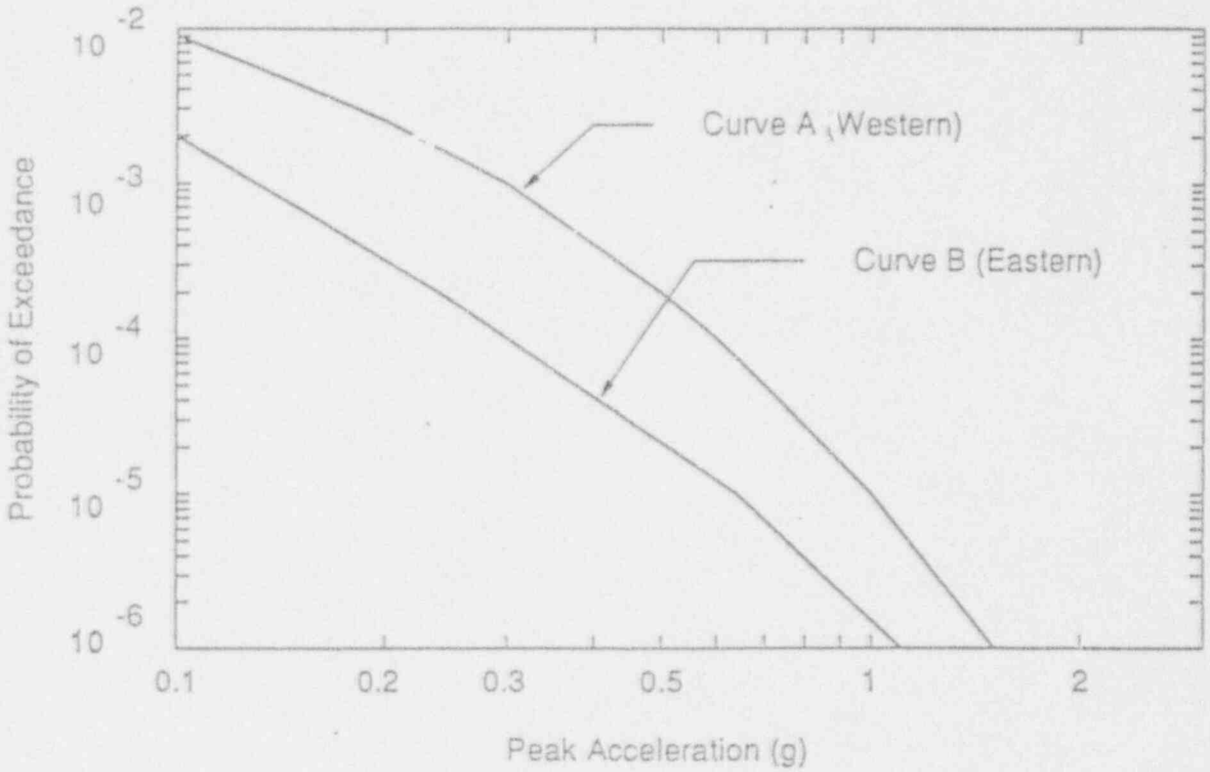

Figure 5-1: Typical Probabilistic Seismic Hazard Curves

Table 5-1
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	~ 10^{-3} of the Onset of SSC[1] Damage to the Extent that Occupants Are Endangered
2	Occupant Safety, Continued Operation with Minimal Interuuption	~ 5×10^{-4} of SSC Damage to the Extent that the Component Cannot Perform its Function
3	Occupant Safety, Continued Function, Hazard Confinement	-10^{-4} of SSC Damage to the Extent that the Component Cannot Perform its Function
4	Occupant Safety, Continued Function, 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 Components (Equipment)

Table 5-2

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	1x10-3	2x10-3	2
2	5x10-3	1x10-3	2
3	1x10-3	1x10-3	10
4	1x10-5	2x10-4	20

Table 5-3

Typical Ground Motion Ratios and Hazard Slope Parameters

Hazard curve	Probability Range	A_R	K_H
A (Western)	10^{-3} to 10^{-4}	2.0	3.32
A (Western)	10^{-4} to 10^{-5}	1.67	4.49
B (Eastern)	10^{-3} to 10^{-4}	2.31	2.75
B (Eastern)	10^{-4} to 10^{-5}	2.13	3.05

Table 5-4a

Maximum and Minimum Required Safety Factors F_{pr} To Achieve a Risk Reduction Ratio of $R_R = 20$ For Capacities C_p Defined at Various Failure Probabilities and Various f_a Values

Parameter Range: $1.5 \leq A_R \leq 3.25$ $0.3 \leq \beta \leq 0.5$

Capacity C_p Probability	F_{PR} Range*			
	$f_a = 0.5$	$f_a = 0.45$	$f_a = 0.4$	$f_a = 0.35$
1%	$\frac{1.21}{0.78} = 1.55$	$\frac{1.31}{0.86} = 1.52$	$\frac{1.44}{0.97} = 1.48$	$\frac{1.62}{1.05} = 1.54$
5%	$\frac{1.51}{1.09} = 1.39$	$\frac{1.61}{1.21} = 1.33$	$\frac{1.77}{1.34} = 1.32$	$\frac{1.98}{1.34} = 1.48$
10%	$\frac{1.82}{1.31} = 1.39$	$\boxed{\frac{1.87}{1.46} = 1.28}$	$\frac{1.98}{1.49} = 1.33$	$\frac{2.21}{1.49} = 1.48$
15%	$\frac{2.05}{1.48} = 1.39$	$\frac{2.11}{1.60} = 1.32$	$\frac{2.24}{1.60} = 1.40$	$\frac{2.43}{1.60} = 1.52$
20%	$\frac{2.26}{1.63} = 1.39$	$\frac{2.33}{1.70} = 1.37$	$\frac{2.47}{1.70} = 1.45$	$\frac{2.68}{1.70} = 1.58$
50%	$\frac{3.45}{2.16} = 1.60$	$\frac{3.55}{2.19} = 1.62$	$\frac{3.76}{2.19} = 1.72$	$\frac{4.08}{2.19} = 1.86$

* Key to read table: $\frac{\text{Max}F_{PR}}{\text{Min}F_{PR}}$ = Ratio

$\boxed{}$ = Lowest ratio case

Table 5-4b

Maximum and Minimum Required Safety Factors F_{pr} to Achieve a Risk Reduction Ratio Of $R_R = 10$ for Capacities C_p Defined at Various Failure Probabilities And Various f_a Values

Parameter Range: $1.5 \leq A_R \leq 3.25$ $0.3 \leq \beta \leq 0.5$

Capacity C_P Probability	F_{PR} Range*			
	$f_a = 0.55$	$f_a = 0.5$	$f_a = 0.45$	$f_a = 0.4$
1%	$\frac{1.08}{0.71} = 1.52$	$\frac{1.16}{0.78} = 1.49$	$\frac{1.26}{0.86} = 1.47$	$\frac{1.39}{0.92} = 1.51$
5%	$\frac{1.34}{0.99} = 1.35$	$\frac{1.42}{1.09} = 1.30$	$\frac{1.54}{1.18} = 1.31$	$\frac{1.71}{1.18} = 1.45$
10%	$\frac{1.61}{1.19} = 1.35$	$\boxed{\frac{1.61}{1.31} = 1.23}$	$\frac{1.72}{1.32} = 1.30$	$\frac{1.91}{1.32} = 1.45$
15%	$\frac{1.82}{1.35} = 1.35$	$\frac{1.82}{1.42} = 1.28$	$\frac{1.90}{1.42} = 1.34$	$\frac{2.05}{1.42} = 1.44$
20%	$\frac{2.00}{1.48} = 1.35$	$\frac{2.00}{1.50} = 1.33$	$\frac{2.09}{1.50} = 1.39$	$\frac{2.25}{1.50} = 1.50$
50%	$\frac{3.05}{1.94} = 1.57$	$\frac{3.05}{1.94} = 1.57$	$\frac{3.19}{1.94} = 1.64$	$\frac{3.42}{1.94} = 1.76$

* Key to read table: $\frac{\text{Max}F_{PR}}{\text{Min}F_{PR}}$ = Ratio

$\boxed{}$ = Lowest ratio case

Table 5-4c

Maximum and Minimum Required Safety Factors F_{pr} to Achieve A Risk Reduction Ratio of $R_R = 5$ for Capacities C_p Defined at Various Failure Probabilities and Various f_a Values

<u>Parameter Range</u>: $1.5 \leq A_R \leq 3.25$ $0.3 \leq \beta \leq 0.5$

Capacity C_P Probability	F_{PR} Range*			
	$f_a = 0.65$	$f_a = 0.6$	$f_a = 0.55$	$f_a = 0.5$
1%	$\frac{0.91}{0.60} = 1.52$	$\frac{0.96}{0.65} = 1.48$	$\frac{1.02}{0.71} = 1.44$	$\frac{1.10}{0.77} = 1.43$
5%	$\frac{1.19}{0.84} = 1.42$	$\frac{1.19}{0.91} = 1.31$	$\frac{1.25}{0.99} = 1.26$	$\frac{1.36}{1.05} = 1.30$
10%	$\frac{1.42}{1.01} = 1.41$	$\frac{1.42}{1.09} = 1.30$	$\boxed{\frac{1.42}{1.17} = 1.21}$	$\frac{1.51}{1.17} = 1.29$
15%	$\frac{1.61}{1.14} = 1.41$	$\frac{1.61}{1.23} = 1.31$	$\frac{1.61}{1.26} = 1.28$	$\frac{1.63}{1.26} = 1.29$
20%	$\frac{1.77}{1.26} = 1.40$	$\frac{1.77}{1.33} = 1.33$	$\frac{1.77}{1.33} = 1.33$	$\frac{1.77}{1.33} = 1.33$
50%	$\frac{2.70}{1.66} = 1.63$	$\frac{2.70}{1.71} = 1.58$	$\frac{2.70}{1.71} = 1.58$	$\frac{2.70}{1.71} = 1.58$

* Key to read table: $\frac{MaxF_{PR}}{MinF_{PR}}$ = Ratio

$\square$ = Lowest ratio case

Table 5-4d

Maximum and Minimum Required Safety Factors F_{pr} to Achieve a Risk Reduction Ratio of $R_R = 2$ for Capacities C_p Defined at Various Failure Probabilities and Various f_a Values

<u>Parameter Range</u>: $1.5 \leq A_R \leq 3.25$ $0.3 \leq \beta \leq 0.5$

Capacity C_p Probability	F_{PR} Range*			
	$f_a = 0.8$	$f_a = 0.75$	$f_a = 0.7$	$f_a = 0.65$
1%	$\frac{0.73}{0.49} = 1.49$	$\frac{0.74}{0.52} = 1.42$	$\frac{0.78}{0.56} = 1.39$	$\frac{0.80}{0.56} = 1.43$
5%	$\frac{1.01}{0.68} = 1.49$	$\frac{1.01}{0.73} = 1.38$	$\frac{1.01}{0.78} = 1.29$	$\frac{1.01}{0.79} = 1.28$
10%	$\frac{1.21}{0.82} = 1.48$	$\frac{1.21}{0.87} = 1.39$	$\frac{1.21}{0.94} = 1.29$	$\boxed{\frac{1.21}{0.95} = 1.27}$
15%	$\frac{1.37}{0.93} = 1.47$	$\frac{1.37}{0.99} = 1.38$	$\frac{1.37}{1.04} = 1.32$	$\frac{1.37}{1.04} = 1.32$
20%	$\frac{1.51}{1.02} = 1.48$	$\frac{1.51}{1.09} = 1.39$	$\frac{1.51}{1.11} = 1.36$	$\frac{1.51}{1.11} = 1.36$
50%	$\frac{2.30}{1.35} = 1.70$	$\frac{2.30}{1.42} = 1.62$	$\frac{2.30}{1.42} = 1.62$	$\frac{2.30}{1.42} = 1.62$

* Key to read table: $\frac{\text{Max}F_{PR}}{\text{Min}F_{PR}}$ = Ratio

$\boxed{}$ = Lowest ratio case

Table 5-5

DBE Factors, Factors of Safety, and Seismic Load Factors
Required to Achieve Various Risk Reduction Ratios

Risk Reduction Ratio, R_R	DBE Factor, f_a	Required Factor of Safety, F_R	Seismic Load Factor, L_S[1]
20	0.45	1.7	1.15
10	0.5	1.5	1.0
5	0.55	1.3	0.87
2	N.A.[2]	1.0	0.67

1 $L_s = F_{R/1.5}$

2 N.A. = Not Applicable

Table 5-6

Required Safety Factors F_{pr} to Achieve Risk Reduction Ratios R_R of 20,10,5 and 2 Corresponding to 10% Failure Probability Capacities $C_{10\%}$ and $\mathcal{F}_a$ Values from Table 5-5 for Various A_R and ß Values

A_R	K_H	F_{PR}											
		R = 20			R = 10			R = 5			R -2		
		ß = .30	ß = .40	ß = .50	ß = .30	ß = .40	ß = .50	ß = .30	ß = .40	ß = .50	ß = .30	ß = .40	ß = .50
3.75	1.74	1.64	1.53	1.46	1.47	1.38	1.31	1.34	1.25	1.19	1.10	1.03	0.98
3.25	1.95	1.65	1.56	1.49	1.49	1.40	1.34	1.35	1.27	1.22	1.06	1.00	0.96
2.75	2.28	1.68	1.60	1.56	1.51	1.44	1.40	1.37	1.31	1.27	1.02	0.97	0.95
2.25	2.84	1.72	1.67	1.67	1.55	1.50	1.50	1.36	1.32	1.32	0.99	0.96	0.96
2.05	3.21	1.75	1.72	1.75	1.57	1.55	1.57	1.30	1.28	1.30	0.98	0.96	0.98
1.85	3.74	1.79	1.79	1.87	1.49	1.49	1.56	1.24	1.24	1.29	0.97	0.97	1.01
1.65	4.60	1.61	1.66	1.80	1.38	1.43	1.54	1.19	1.23	1.33	0.97	1.01	1.09
1.50	5.68	1.49	1.60	1.82	1.32	1.41	1.61	1.17	1.25	1.42	0.99	1.07	1.21

Table 5-7

Probabilities for Hazard Curve A and B from Figure 1

Curve A		Curve B	
Acceleration a (g)	Exceedance Probability H(a) x (10^{-5})	Acceleration a (g)	Exceedance Probability H(a) x (10^{-5})
14	500	0.07	500
.225	200	0.10	200
.30	100	0.13	100
.38	50	0.17	50
.50	20	0.24	20
.60	10	0.30	10
.71	5	0.38	5
.87	2	0.51	2
1.00	1	0.64	1
1.14	0.5	0.775	0.5
1.33	0.2	0.97	0.2
1.49	0.1	1.12	0.1

Table 5-8a

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

Performance Category	Goals	DBE (g) Eqn's (6a) & (6b)	$C_{10\%}$ (g) Eqn (14) & Table 5-5	Logarithmic Std. Dev.	$C_{50\%}$ (g) Eqn (10)	P_F
#4	$P_H = 2\times10^{-4}$	0.50	0.85	0.3	1.249	0.88×10^{-5}
	$R_R = 20$	(6a)		0.4	1.42	$0.94\ 10^{-5}$
	$P_F = 1\times10^{-5}$			0.5	1.614	1.13×10^{-5}
#3	$P_H = 1\times10^{-3}$	0.30	0.45	0.3	0.661	1.19×10^{-4}
	$R_R = 10$	(6a)		0.4	0.752	1.10×10^{-4}
	$P_F = 1\times10^{-4}$			0.5	0.854	1.09×10^{-4}
#2	$P_H = 1\times10^{-3}$	0.30	0.30	0.3	0.441	4.4×10^{-4}
	$R_R = 2$	(6a)		0.4	0.501	3.9×10^{-4}
	$P_F = 5\times10^{-4}$			0.5	0.570	3.6×10^{-4}

Table 5-8b

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

Performance Category	Goals	DBE (g) Eqn's (6a) & (6b)	$C_{10\%}$ (g) Eqn (14) & Table 5-5	Logarithmic Std. Dev.	$C_{50\%}$ (g) Eqn (10)	P_F
#4	$P_H = 2 \times 10^{-4}$	0.288	0.49	0.3	0.72	1.02×10^{-5}
	$R_R = 20$	(6b)		0.4	0.818	$0.94\ 10^{-5}$
	$P_F = 1 \times 10^{-5}$			0.5	0.93	0.96×10^{-5}
#3	$P_H = 1 \times 10^{-3}$	0.15	0.225	0.3	0.330	1.10×10^{-4}
	$R_R = 10$	(6b)		0.4	0.376	1.00×10^{-4}
	$P_F = 1 \times 10^{-4}$			0.5	0.427	0.98×10^{-4}
#2	$P_H = 1 \times 10^{-3}$	0.13	0.13	0.3	0.191	5.0×10^{-4}
	$R_R = 2$	(6b)		0.4	0.217	4.5×10^{-4}
	$P_F = 5 \times 10^{-4}$			0.5	0.247	4.3×10^{-4}

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Table 5-9

Example Solution of Equation (16) for Hazard Curve B, Performance Category 3, $C_{50} = 0.376g$, $\beta = 0.40$

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	[illegible]
.97	.2				
		1.036	.994	.1	.10
1.12	.1				
				≈ 10	.10

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

Table 5-10

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	L_s	$1.4L_s$
β_I	0	0.2

Table 5-11

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

D_{NS} / C_C	β_{CS}	β_{FS}	Required Safety Factor $F_{S_{Rqd}}$	Achieved Safety Factor F_s	$R_{FS} = F_S / F_{S_{Rqd}}$
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 5-12

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

D_{NS} / C_C	β_{CS}	β_{FS}	Required Safety Factor $F_{S_{Rqd}}$	Achieved Safety Factor F_S	$R_{FS} = F_S / F_{S_{Rqd}}$
0	0.20	0.41	2.54	2.52	0.99
0.1	0.[illegible]1	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

Table 5-13

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)	7	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 tranferred to the brace by the structural system

Connections for steel moment frames and eccentric braced frames and connections for concrete, masonry, and wood structural systems should follow Reference 1 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 5-14

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)	7	-	1.50
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 Diagonal 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 Diagonal 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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