Risk-Consistent Seismic Design of Nuclear Facilities

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ABSTRACT

Federal Regulation 10CFR70.61 establishes performance requirements that must be complied with by licensees authorized to possess greater than a threshold mass quantities of Special Nuclear Material. The main thrust of the performance requirements is to assure that certain consequences are sufficiently unlikely. Appendix B of Standard Review Plan, Review of an Application for a Mixed Oxide (MOX) Fuel Fabrication Facility (NUREG/SR-1718), describes, in general terms, an iterative approach to seismic compliance. This paper presents an alternative direct method to achieve the objectives of 10CFR70, Subpart H. The design process starts with the estimation of the exposure (rems) of the public outside the controlled area with the Leak Path Factor (LPF) as the variable. LPF is then expressed in terms of damage indices, and in the space of the Likelihood of Occurrence of Consequence (LOC) vs. Damage Index (DI) two controlling design points become necessary to satisfy the performance requirements of 10CFR70.61. The LOC is obtained from the convolution of the seismic hazard curve with the LPF-consistent limit state fragility of the structure. This integration leads to a relationship of LOC, in terms of the design ground motion annual exceedance probability and associated Design Basis Earthquake (DBE), and the required design factors.

INTRODUCTION

Federal Regulation 10CFR70.61 establishes performance requirements that must be complied with by licensees authorized to possess greater than a threshold mass quantities of Special Nuclear Material. The main thrust of the performance requirements is to assure that certain consequences are sufficiently unlikely. A recently published Standard Review Plan, Review of an Application for a Mixed Oxide (MOX) Fuel Fabrication Facility (NUREG/SR-1718), describes methods acceptable to NRC to satisfy these requirements. Appendix A of NUREG/SR-1718 presents, in detail, the graded approach for design against operational accident sequences. With respect to Natural Phenomena Hazard induced accidents, Appendix B describes, in very general terms, an iterative approach for seismic compliance. This paper presents an alternative direct method to achieve the objectives of 10CFR70, Subpart H. Due to page limitations the paper deals only with the *public* consequences of an earthquake induced accident, and the *last* confinement barrier of a facility. The interest here is for seismically induced accidents with the outer shell of the facility being the only barrier for the protection of the offsite public. To also include internal mitigation of an accident is simple.

10CFR70.61 PERFORMANCE REQUIREMENTS

Simply put, the 10CFR70.61 performance requirements are as follows:

High consequence accidents must be "highly unlikely"
Intermediate consequence accidents must be "unlikely"
Low consequence accidents must be "not likely"

Appendix A of NUREG/SR-1718 provides numerical guidelines for both the consequence and failure categories. These are summarized in Table 1.

Table 1. Guidelines for Consequence Severity and Likelihood of Occurrence for Offsite Public Exposure

Consequence Category	High	Intermediate	Low
Radiation Dose (TEDE)	$D \geq 25$ rem	5 rem $\leq D <$ 25 rem	$D < 5$ rem
Likelihood: frequency/year/accident	$< 10^{-5}$	$< 4 \times 10^{-4}$	$> 4 \times 10^{-4}$

DOSE: SOURCE TERM AND ATMOSPHERIC DISPERSION

The design process starts with the estimation of the exposure (rems), with or without any internally credited mitigation, of a member of the public outside the controlled area. The definitions and descriptions of parameters used in Source Term calculations are given in Table 2. The source term represents the fraction of a facility's material inventory that, due to accident conditions, will breech all facility mitigative controls and become available for transport to receptors via atmospheric dispersion or other modes of environmental transport.

Table 2. Definitions and Descriptions of Source Term Parameters

	Definition	Description
ST	Source Term (Ci or kg)	Amount of material available for environmental transport that is capable of contributing to human inhalation radiation dose
MAR	Material at Risk (Ci or kg)	Fraction of full inventory of radionuclide material within a facility with the potential to be involved in an accident
DR	Damage Ratio (Unitless)	Fraction of a facility's MAR that could conceivably be involved in a given accident scenario
ARF	Aerosolized Release Fraction (Unitless)	Fraction of material involved in a given accident scenario which becomes aerosolized
RF	Respirable Fraction (Unitless)	Fraction of material aerosolized in given accident scenario that is in particulate form small enough to be inhaled by humans
LPF	Leak Path Factor (Unitless)	Fraction of material involved in a given accident that is not attenuated from reaching the environment for subsequent transport due to the physical presence of the facility

The standard analysis method for estimating source terms is given by Eq. 1

$$ST = MAR \cdot DR \cdot ARF \cdot RF \cdot LPF \tag{1}$$

There is controversy in the safety analysis community over the appropriate parameter values to be used in the above equation for various accident scenarios. This issue is beyond the scope of this paper. Since material at risk, damage ratio, aerosolized release fraction, and respirable fraction are facility-specific given parameters, the only parameter under the control of the structural engineer is the Leak Path Factor (LPF). If the LPF≈0.0, such as for a containment structure of a nuclear power plant reactor, the off-site dose would be zero. On the other hand, LPF=1.0 means unmitigated consequences.

MACCS2, a code developed and maintained by Sandia National Laboratory, can be used to compute the atmospheric dispersion of the source term radionuclides. In the MACCS2 calculations, the dispersion of radionuclides is mapped onto a polar grid. This grid is centered on the facility release point, extends 50 miles, and is divided into 16 angular regions. The radial divisions are site specific because the population grids, with which they must coincide, are site specific. Atmospheric dispersion is by nature a stochastic process, and the MACCS2 code addresses this by yielding results at various levels of confidence. The MACCS2 code uses dose conversion factors to translate the radionuclide distribution in each node of the 50 mile polar grid into an estimate of both the Total Effective Dose Equivalent (TEDE) and Committed Effective Dose Equivalent (CEDE) based on the inhalation pathway. Implicit in this translation are the standard assumptions with regard to human anatomy and biological processes.

The MACCS2 code was envisioned to be very robust, hence there are numerous parametric options that allow the dispersion calculations to account for a wide variety of physical situations that may be encountered. Examples of these parameters are weather stability class, dispersion parameterization, plume duration, release height, building wake, mixing height, plume initial size and energy, deposition, and evacuation.

DESIGN SPACE AND CONTROLLING DESIGN POINTS

Figure 1 is used to march through the logic of the design process. In the upper-right quadrant the site boundary dose can be correlated to LPF given the other constant parameters of Eq. 1 and the atmospheric conditions at the site. The scale of the axes in this quadrant is logarithmic. As shown in the lower-right quadrant, the LPF is then related to the Damage Index (DI) of the structure. Damage indices are measures of irrecoverable (inelastic) deformations. A damage index of zero means essentially elastic recoverable response (public exposure would be zero), and a damage index of unity means total collapse (public exposure would be the total unmitigated calculated dose). The curve shown is dependent on the type and configuration of structure. For structural design, damage indices can be correlated to design parameters, such as ductility ratio, μ, interstory displacement, etc., as given by Eq. 2,

$$DI = \left(\frac{d_{cal} - d_o}{d_u - d_o} \right)^{\alpha} \tag{2}$$

where d is a generic damage variable, d_{cal} is the calculated value, d_o is the value at DI=0, and d_u is the value at DI=1.0 [1]. α is an exponent that must be obtained from earthquake and other experimental data for each class of structures.

The upper-right quadrant is subdivided along the dose axis into three regions corresponding to the Consequence Categories of Table 1. For this illustrative example the unmitigated dose at the site boundary is 100 rems at LPF=1.0, and the boundaries of the consequence regions map into LPF_{25}=0.25 and LPF_5=0.05, where the LPF subscripts refer to the lower limits of High and Intermediate public consequences (rem), respectively. The required DI values are estimated by further tracing of these dose boundary lines into the lower-right quadrant to intersect the curve relating LPF to DI. The DI values for this example are DI_{25}=0.45 and DI_5=0.15.

The lower-left quadrant is the Design Space. The horizontal axis shows the Likelihood of Occurrence of Consequence (LOC) in logarithmic scale and the vertical axis is the DI. Along the DI axis, the range of the DI, 0.45-1.0 for this example, relates to the High Consequence region. Any design at this consequence level should achieve the Highly Unlikely occurrence target of $<10^{-5}$ (Table 1). A vertical line, AB, thus separates the space at this DI range into acceptable (to the left of AB) and unacceptable (to the right of AB) subspaces. Similarly, the Intermediate Consequence range along the DI axis, 0.15-0.45 for this example, can be separated into acceptable and unacceptable subspaces at the Unlikely occurrence target of $<4\text{x}10^{-4}$ by the vertical line CD. And finally, for the Low Consequence range along the DI axis, 0.0-0.15 for this example, all of the space to the left of the DI axis would be acceptable. The LOC vs. DI space is thus divided into two subspaces of acceptable (shaded) and unacceptable (the rest) performance. Clearly, in the space of the Likelihood of Occurrence of Consequence vs. Damage Index only two controlling design points become necessary to satisfy the performance requirements of 10CFR70.61: LOC of $4\text{x}10^{-4}$ at DI_5 and LOC of 10^{-5} at DI_{25}.

A third design level seems to be useful. Referring to Fig. 1 note that no upper bound on the occurrence of low consequence accidents is placed. In seismically active regions, the impact of small earthquakes occurring frequently may not be acceptable purely on operational considerations, and an upper bound occurrence of consequence frequency may need to be established. For example, setting this limit at $<10^{-2}$ establishes the third controlling design point, E: LOC of 10^{-2} at DI_0, where DI_0 signifies elastic response. This concept is similar to the Operating Basis Earthquake used in nuclear power plant designs.

DETERMINATION OF DESIGN CAPACITY

Although a designer is interested in a certain load intensity to complete his design (say, ground motion parameters at 10% in 50 years exceedance probability), it should be noted that design to a specific ground motion parameter in terms of an exceedance probability all by itself is not a sufficient determinant of the problem at hand. Any one structure, in effect, is potentially exposed, throughout its design life, to *all* the possibilities of the occurrence of ground motion intensities at a given site as characterized by the site seismic hazard curve. A seismic hazard curve for a given site is simply the quantification of the probability of exceedance of a specific ground motion intensity. And, therefore, a proper design in reliability space should consider all possible ground motions as characterized by a seismic hazard curve, and not just a single point on it, for *each and every* selected performance limit state, DI_i.

Earthquake "failure" probability, where "failure" is a generic term defining non-performance at a pre-selected limit state, such as DI_5 or DI_{25}, is obtained by the convolution of the seismic hazard curve with the selected limit state fragility of the structure. Fragility is the function of the conditional (given that ground motion **a** or greater occurs) "failure" probability of the system. In the present context "failure" probability is the LOC. Failure probability can be calculated from Eq. 3

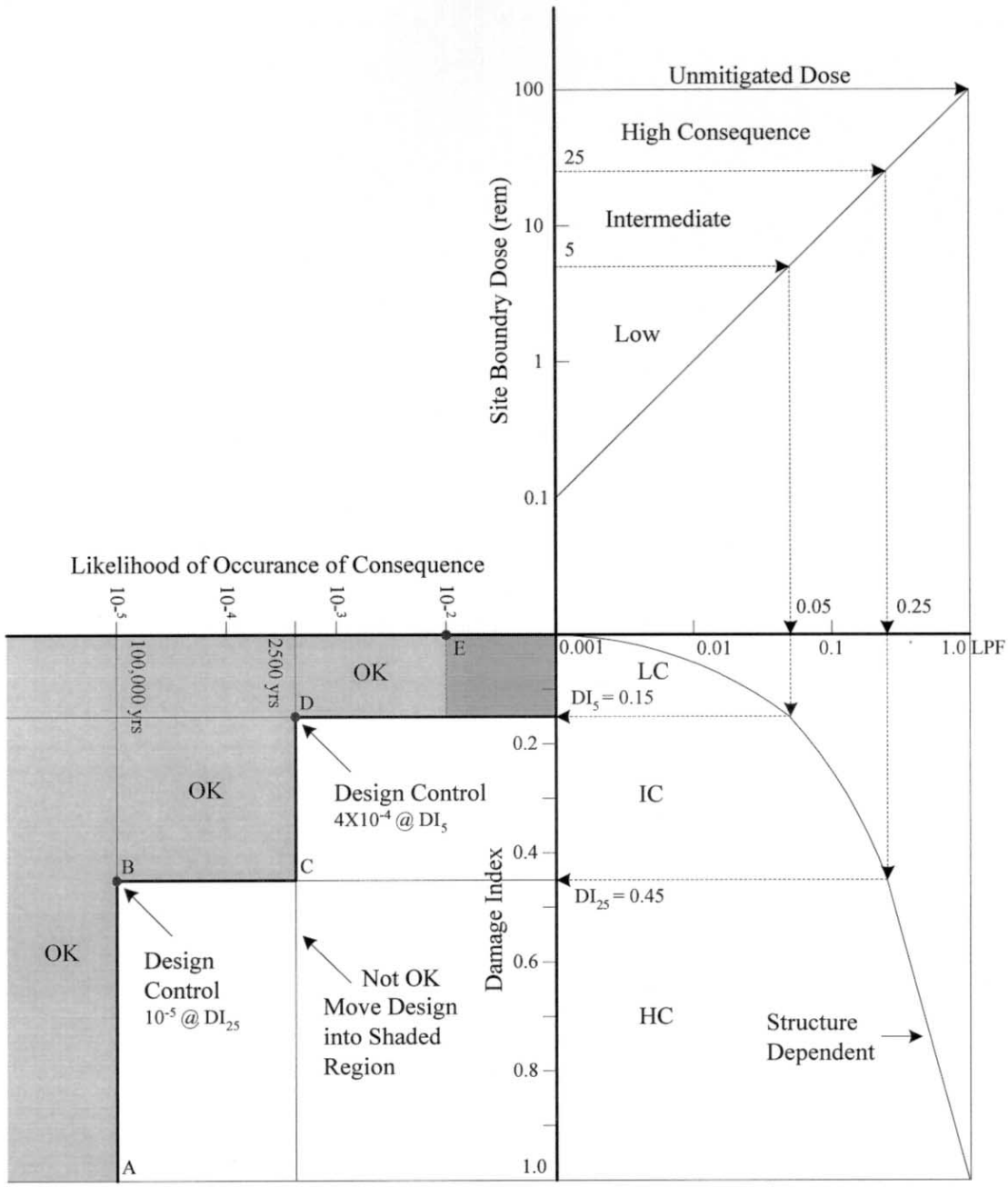

Fig. 1 Steps in the Determination of Design Control Points

$$P_F = \int_o^\infty H(a)\, f(a)\, da \tag{3}$$

where H(a) is the seismic hazard curve of the ground motion parameter under consideration, and f(a) is the system fragility density function.

Making some simplifying assumptions, Eq. 3 can be integrated [2], resulting in the following failure probability relationship

$$P_F = \frac{H_D \, e^{1/2(K_H \zeta)^2}}{(F_{50}/DBE)^{K_H}} \tag{4}$$

where H_D = annual frequency of exceedance of the selected Design Basis Earthquake (DBE) ground motion level (e.g., 4×10^{-4}, as in the USGS national seismic hazard maps), F_{50} = median of the applicable *in-situ* limit state fragility required to achieve the design objective (assumed lognormally distributed), $\zeta^2 = \zeta_F^2 + \zeta_H^2$ = variance of the fragility distribution and the seismic hazard[1], and K_H = a slope parameter of the hazard curve, H(a), defined at the vicinity of the DBE (for details see [3]). The in-situ median fragility, F_{50}, should be distinguished from building code design capacity. F_{50} is a probabilistic assessment of the "failure" of a system under a specified excitation.

It should be noted from Eq. 4 that the design level exceedance probability, H_D, and the failure probability, P_F, can be significantly different (numerically) depending on the slope parameter of the hazard curve, K_H, the in-situ median fragility, F_{50}, and the total variance, ζ^2. This point, illustrated in Fig. 2, needs to be emphasized. Fig. 2 shows plots of the P_F/H_D ratio as a function of the *design factor*, F_{50}/DBE, based on typical seismic hazard curves (Fig. 2a) for Western US and Eastern US (see [3] for details).

When $P_F/H_D > 1.0$ the failure probability, P_F, is larger than the exceedance probability of the DBE; and, vice versa. P_F/H_D depends, in a complex fashion, on the variation of the slope of the hazard curve (calculated at H_D) along any one of the hazard curves; and, for any ground motion design basis exceedance probability (each one of the curves in b, c and d), different failure probabilities are obtained than the respective exceedance probabilities of the DBE as a function of both the design factor and ζ. As H_D decreases (or return period increases) the impact of the design factor on the P_F/H_D ratio increases dramatically, and could vary by orders of magnitude. Note that the ordinate of the plots in Fig. 2 is in log scale. A comparison of Figs. 2b and 2c, for the same ζ, shows that P_F/H_D is more sensitive to changes in the design factor for the Western US relative to the Eastern US. And finally, a comparison of Figs. 2c and 2d shows a significant dependence of P_F/H_D on the variability parameter, ζ. Thus, simply designing to a specific annual exceedance probability of the ground motion, say, 10% in 50 yrs, does not directly provide any information on failure probability. To make rational risk-consistent decisions for life-safety, damage control (e.g. confinement) or functionality limit states, what is needed is an estimate of the *failure* (non-performance) probability, P_F, for each specific limit state.

Thus, given LOC and the selected design ground motion annual exceedance probability, H_D, say 4×10^{-4} (or annual return period of 2500 years), the required design factors (F_{50}/DBE) on the in-situ median fragilities at DI_5 and DI_{25} are calculated. Multiplying the design factor by the DBE determines the required in-situ median fragility, F_{50}. Based on F_{50}, the building code specified capacity can be calculated as discussed below [3].

Building Code Specified Capacity

Once the design factor, $DF_{50} = \frac{F_{50}}{DBE}$, is calculated, the required median in-situ fragility, F_{50}, is obtained from, $F_{50} = DF_{50} \times DBE$. The next step is to translate F_{50} into a resistance-related parameter for design. The conditional failure probability $F_{P/a}$ (any one point on the fragility curve) is calculated by the convolution of load, S_{50}^a (for each sequentially selected *a*), and applicable resistance distribution function. When the ratio $\frac{S_{50}^a}{R_{50}}$ is small, the failure probability would be small. As S_{50}^a is increased and becomes equal to R_{50}, then $\ln(R_{50}/S_{50}) = \ln 1 = 0$, and the conditional failure probability is 0.50 (irrespective of ζ). In other words, at F_{50}, the median load and median resistance are identically equal, i.e. $R_{50} \equiv F_{50}$. The in-situ median resistance is thus given by

$$R_{50} = DF_{50} \cdot DBE \qquad (for\ all\ \zeta) \tag{5}$$

This step transfers the problem of determining the resistance from fragility space into resistance space. Any further manipulation of results should consider only the variance of the resistance, i.e. ζ_R^2, and not ζ_F^2 of the fragility curve.

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[1]In computations of failure probabilities, uncertainties associated with the seismic hazard are routinely accounted for by the use of the mean in lieu of the median hazard curve and ζ_H.

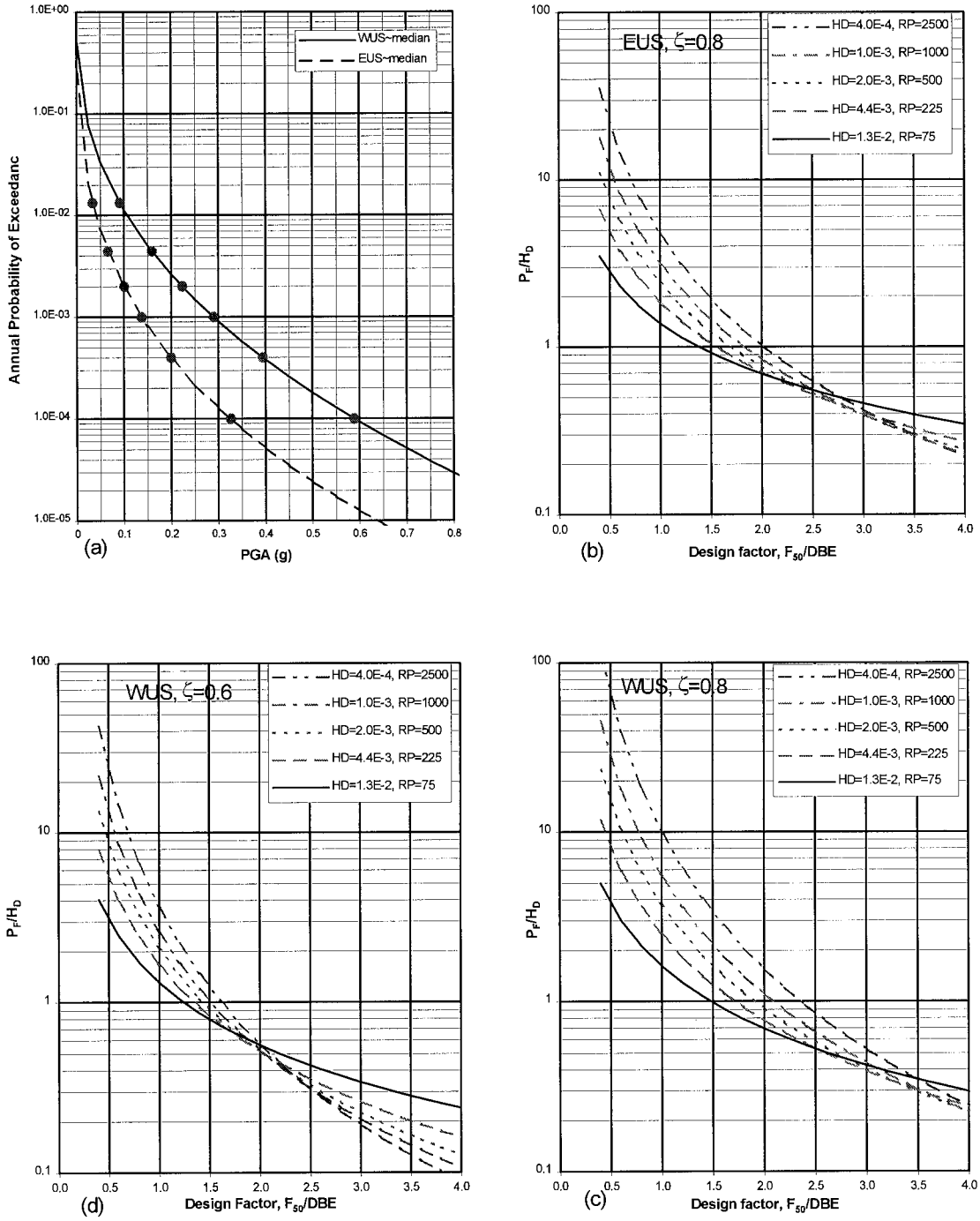

Fig. 2. P_F/H_D as a Function of Design Factor and Total Variance

In practice, mean rather than median values of loads and resistances are used based on empirical data, and a lognormal distribution of resistance is usually considered appropriate. Thus, it is a simple matter to calculate the required mean resistance, $\bar{R}$, from the following relationship of the lognormal distribution

$$F_{50} = \frac{\bar{R}}{\sqrt{1 + COV_R^2}} \tag{6}$$

where COV_R is the coefficient of variation, and $\bar{R}$ is still the required in-situ mean resistance.

There are a series of implicit and explicit conservatisms throughout the design process, such as response analysis models and methods, any safety factors used in design, implicit design conservatisms, lower bound estimates of material properties and capacities, smoothing of member dimensions for ease of construction, redundancy, quality aspects of design and construction, etc. These conservatisms need to be quantified in terms of the in-situ mean capacity leading to an aggregated factor of conservatism, C_R. C_R is a generalization of the overstrength issue at the global structure level. Other significant parameters contributing to C_R are the gravity to seismic load effects ratio and the type of structural configuration. The estimation of C_R is a rather subjective art. In any event, the required mean design-based resistance, $\bar{R}_D$, can be calculated from Eqs. 5 and 6,

$$\bar{R}_D = \frac{DF_{50}.DBE\sqrt{1 + COV_R^2}}{C_R} \tag{7}$$

As code-based designs, at the structural member level, are usually performed with "nominal" values of load effects and resistances, Eq. 7 is modified based on the mean-to-nominal ratio, $N_R = \frac{\bar{R}_D}{R_n}$, to

$$R_n = \left(\frac{DF_{50}\sqrt{1 + COV_R^2}}{N_R C_R}\right) DBE \tag{8a}$$

Data on COV_R suggests that its impact in Eq. 8a would be negligible, and, therefore, Eq. 8a can be simplified to

$$R_n = \left(\frac{DF_{50}}{N_R C_R}\right) DBE \tag{8b}$$

Given the confines of this paper, only seismic loads are considered. The related design equation in code format would be $\phi R_n = \gamma_E E_{50}$ (a load combination similar to $\phi R_n = 1.4 D_n$) where γ_E is the sought for design load factor on the *median* seismic load. We will continue to characterize the DBE as the specific median value from the hazard curve rather than a"nominal" seismic load of usually undefined pedigree. Therefore, from Eq. 8b, the seismic load factor becomes

$$\gamma_E = \frac{\phi}{N_R C_R} DF_{50} \tag{9}$$

Design codes specify different ϕ for different actions. The ACI code, for example, specifies $\phi = 0.9$ for flexure and 0.6 for shear in high seismic regions. In addition to reflecting a slightly larger COV (0.20 vs 0.13), the smaller ϕ for shear more importantly reflects the design profession's intent to make seismically induced shear failure significantly less likely than flexural failure. Thus, it may be prudent not to meddle with currently used ϕ values and to normalize γ_E with the largest ϕ (i.e. 0.9). The expected relative reliabilities for other ϕ (actions) would thus be maintained.

To complete the design specification, DI_5 and DI_{25} need to be converted to design ductilities. This step is illustrated below.

Example

Reinforced concrete flexure: $COV_R = 0.13$, $\phi = 0.90$ and $N_R = 1.12$
$LOC_{25} = 10^{-5}$ and $LOC_5 = 4 \times 10^{-4}$
$H_D = 4 \times 10^{-4}$ (2500 year return period, as in the USGS national seismic hazard maps)
$K_H = 3.72$ [3] and $\zeta = 0.5$.
Substituting the above parameter values into Eq. 4 we obtain

$$(DF_{50})^{3.72} = \frac{4 \times 10^{-4}}{10^{-5}} e^{1/2(3.72 \times 0.5)^2}$$

and thus,

$DF_{50} = 4.29$ for LOC_{25}, and similarly $DF_{50} = 1.59$ for LOC_5.

Assuming $C_R = 2.0$, the design load factors from Eq. 9 are

$$\gamma_E = \frac{0.9}{1.12 \times 2} \times 4.29 = 1.72 \; for \; LOC_{25}$$

and similarly $\gamma_E = 0.64$ for LOC_5.

Maximum allowed ductilities:

Assume $\alpha = 1.5$ in Eq. 2, ultimate ductility, $d_u = 6.0$, and $d_o = 1.0$
Substituting the above parameter values into Eq. 2 we obtain

$$DI_{2S} = 0.45 = \left(\frac{d_{cal} - 1.0}{6.0 - 1.0} \right)^{1.5}$$

and thus,

$d_{cal} = 3.9$ for DI_{25}, and similarly $d_{cal} = 2.4$ for DI_5.

Therefore, the following design points would satisfy 10CFR70.61 performance requirements: $\gamma_E = 1.72$, $\mu = 3.9$, $\phi = 0.9$, and $\gamma_E = 0.64$, $\mu = 2.4$, $\phi = 0.9$, with ground motion parameters from the seismic hazard curve at $H_D = 4 \times 10^{-4}$ (2500 year return period) for both design control points.

CONCLUSIONS

A direct design procedure is proposed to satisfy Federal Regulation 10CFR70.61 performance requirements. In the Likelihood of Occurrence of Consequence vs. Damage Index space two controlling design points are identified that satisfy these performance requirements: LOC of 4×10^{-4} at DI_5 and LOC of 10^{-5} at DI_{25}. Given a site seismic hazard curve, a design factor is calculated to satisfy these controlling design points. The design factor is then expressed in terms of building code format nominal resistance and load factor. The estimation of two important parameters in the solution require further study: overstrength of structures and damage indices.

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