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U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

ATTENTION: T. R. QUAY

SUBJECT: WESTINGHOUSE AP600 SEISMIC MARGIN HCLPF VALUES METHODOLOGY AND RESPONSES TO MEETING OPEN ITEMS

Dear Mr. Quay:

The NRC performed an audit of the Westinghouse AP600 seismic margin high-confidence/low-probability of failure (HCLPF) calculations on February 8 and 9, 1995. The enclosed document provides the Westinghouse response to the open items presented in the NRC meeting minutes (NRC letter dated March 6, 1995) for that audit. Also included in the enclosure is the methodology Westinghouse will use to revise the seismic margin HCLPF values to address the NRC open issues.

Please contact Cynthia L. Haag on (412) 374-4277 if you have any questions concerning this transmittal.

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Advanced Plant Safety and Licensing

/nja

Enclosures

Attachment

cc: D. Jackson, NRC (1 copy Enclosure)
N. J. Liparulo, Westinghouse (w/o enclosure)

Enclosure to Westinghouse
Letter NTD-NRC-96-4625

January 17, 1996

AP600 SEISMIC MARGIN HCLPF VALUES METHODOLOGY

JANUARY 1996

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AP600 SEISMIC MARGIN HCLPF VALUES METHODOLOGY

1.0 PURPOSE

To define the methodology for revising the seismic margin high-confidence/low-probability of failure (HCLPF) values developed in 1994 to address U.S. Nuclear Regulatory Commission (NRC) open issues.

2.0 BACKGROUND

An audit of the Westinghouse seismic margin HCLPF calculations was performed by the NRC on February 8 and 9, 1995 (Reference 1). The specific issues and responses are given below:

- The NUREG/CR-0098 (Reference 2) median shape response spectrum for the review level earthquake of 0.5g on the basis of NUREG-1407 (Reference 3) is questionable. NUREG-1407 is not appropriate for the advanced passive design of AP600, nor is it appropriate to use the median shape spectrum from Reference 2.

  Response - The one-sigma (84.1 percent) Reference 2 spectra will be reflected in the seismic margin HCLPF values. (Section 3.2)

- A rationale must be provided for determining uncertainty parameters (random and uncertainty lognormal standard deviations identified as βr and βu, respectively). Consistency amongst similar structural elements and components must be maintained.

  Response - Consistency will be maintained. (Section 3.3 covers fragility methodology.)

- A concern exists that local failures or other failure modes could take over and produce a lower HCLPF value. Provide a rationale for using the additional conservatism factors and for modifying the ductility factors.

  Response - The potential failure modes will be examined, and the most critical one will be considered. (Section 3.3 presents margin factors that include the ductility factors.)

- Safe shutdown earthquake (SSE) and design basis accident (DBA) related loads are not combined in a consistent manner for the margins assessment of various components. Provide the rationale for not applying the same considerations uniformly for all components (i.e., reactor vessel supports, fuel assemblies, and containment structure).

  Response - Seismic HCLPF values will be uniformly developed without the combination of seismic and DBA (loss-of-coolant accident [LOCA]) loads since the probability of the seismic loads occurring at the same time as the LOCA loads is very small (remote). This same procedure is followed in Reference 5.

- Provide documentation that discusses the bases for all factors included in the HCLPF calculation for the fuel assembly.

  Response - The bases for all of the factors included in the HCLPF calculations for the fuel assembly will be documented in the calculations performed.

- The latest seismic model and seismic response analyses results, including consideration of various site conditions, should be used in performing the seismic margins calculations.

  Response - The actual calculation of revised seismic margin HCLPF values will be performed using the latest analyses that reflect the various site conditions.

- Appendix H editorial and clarification comments need to be addressed.

  Response - The editorial and clarification comments associated with the submittal will be specifically addressed when revising Appendix H.

3.0 METHODOLOGY

3.1 HCLPF Determination

In the development of the HCLPF values, consistency among similar structural elements and components will be maintained. The uncertainty parameters will be based on methods, procedures, and applicable data recognized and accepted in the public domain. The AP600 design and analysis documentation that considers the various site conditions will be used.

In the calculation of the HCLPF value for a system, structure, or component, the governing failure mode will be established by examining the different failure modes possible. For instance, these include: buckling, yielding, shear failure, and loss of function (electrical component). Each failure mode has a different reserve margin. As an example, ductility may be very large for tension failure, but low for a buckling failure. The effect of potential local failures must be assessed on the overall structural integrity of a given system, structure, or component.

Seismic margin HCLPF values will be defined using the following methods:

- Use generic data that are representative of supplied equipment for Westinghouse plants.

  This procedure is used to obtain a representative (lower-bound) estimate of the component HCLPF value when design or qualification data are not available. This procedure is also used to obtain a lower-bound, but realistic, HCLPF value for many system configurations where it is not possible to use completed design/analysis information (e.g., piping systems, cable trays). References 4, 5, and 21 are typical.

- Define a lower-bound HCLPF value when umbrella seismic spectra or seismic design levels are used. This is a deterministic approach that also include appropriate deterministic strength factors (see Subsection 3.3.1).

- Use the fragility analysis to define the HCLPF value based on AP600 design specific information. Note that this procedure is used for the majority of the systems, structures, and components.

  Lognormal probability distribution is used to define the HCLPF value. The formulation is given below.

  When random and standard deviations, βr and βu, are available:

$$\text{HCLPF} = \text{Median Capacity} \times e^{[-1.65 \times (\beta r + \beta u)]} \qquad (1)$$

  where:

  βr = Standard deviation associated with randomness
  βu = Standard deviation associated with uncertainty

  When composite standard deviation is available and not the random and uncertainty standard deviations:

$$\text{HCLPF} = \text{Median Capacity} \times e^{[-2.3 \times \beta c]} \qquad (2)$$

where:

  βc = Composite Standard Deviation

  Fragility analysis margin factors and variability (βr, βu) are discussed in Section 3.3.

3.2 Response Spectra

The seismic margin HCLPF values will be defined for the Reference 2 one-sigma (84.1 percent) spectrum. The following procedure will be used dependent on whether the seismic margin HCLPF is defined using deterministic or fragility analysis methods. The response spectra associated with 5 percent equipment damping are used. Examples of both methods are provided in this section.

Deterministic Method

When deterministic methods are employed, a factor will be used to adjust the HCLPF from the AP600 design spectrum to the NUREG/CR-0098 one-sigma ground response spectrum defined in Reference 2.

Fragility Analysis Method

When fragility analysis methods are employed, margin factors will be used to adjust the fragility data from the AP600 design spectrum to the NUREG/CR-0098 median spectrum. However, a standard deviation (classified as random, βr) will be used to reflect the variability of the NUREG/CR-0098 spectrum shape so that the HCLPF values are based on the one-sigma spectrum described in Reference 2.

Example

The following examples are given to show how the deterministic and fragility analysis spectra adjustments are employed.

Deterministic Method

Consider a structure or component whose seismic response is one-mode dominant. The frequency is 5 hz. The item is excited by a ground response spectrum input. The HCLPF value based on the NUREG/CR-0098 median spectrum is equal to 0.9g. The HCLPF value is adjusted to NUREG/CR-0098 one-sigma spectrum as follows:

Ground Response Spectrum Acceleration Levels at 5 hz

NUREG/CR-0098 median spectrum = 1.06g

NUREG/CR-0098 one-sigma spectrum = 1.36g

Adjusted HCLPF Value

$$(HCLPF)_{adj} = 0.9g \times (1.06g/1.36g) = 0.7g$$

Fragility Analysis Method

Consider the same example as above, except that the HCLPF value is calculated by fragility analysis with the following fragility parameters:

Median = 1.6g
βr = 0.15

$\beta u \quad = \quad 0.2$

$HCLPF = 1.6g\ e^{[-1.65(0.15+0.2)]} = 0.9g$

It is recognized that the HCLPF value of 0.9g is based on spectra factors defined using the NUREG/CR-0098 median spectrum. To adjust the fragility data and calculate a new HCLPF based on the NUREG/CR-0098 one-sigma spectrum, an additional random standard deviation is introduced.

$(Sa)_{84\%}$ = Spectral acceleration value at 5 hz from 84 percent Reference 2 5 percent damping spectrum

$(Sa)_{84\%} = 1.36g$

$(Sa)_{median}$ = Median Reference 2 spectral acceleration value at 5 hz from 5 percent damping spectrum

$(Sa)_{median} = 1.06g$

$(\beta r)_{spectra} = \ln[(Sa)_{84\%}/(Sa)_{median}]$

$(\beta r)_{spectra} = \ln(1.36/1.06) = 0.25$

$(\beta r)_{revised} = (0.15^2 + 0.25^2)^{1/2} = 0.29$

$HCLPF_{revised} = 1.6g\ e^{[-1.65(0.29+0.2)]} = 0.71g$

Note that the deterministic and fragility analysis methods give similar revised HCLPF values.

3.3 Fragility Analysis

Fragility analysis is used to calculate the HCLPF value for systems, structures, and components. Provided below is an overview of the seismic margins and associated variability that are considered in the development of the HCLPF values using this method. The basic grouping of margin factors are:

- Deterministic strength factors
- Variable strength factors
- Modeling and analysis
- Loading
- Inelastic energy absorption
- Testing
- Spectra

These factors are discussed in the following subsections, with the exception of spectra. Spectra margin and variability are discussed in Section 3.2.

3.3.1 Deterministic Strength Factors

Margin factors that have no variability exist in the designs. They must be considered in the seismic fragility analysis so that the median capacity can be properly established. Three types of factors are considered:

- Margin to code allowable

- Analysis assumptions that are conservative when compared to design criteria (e.g., lower damping value used than allowed by criteria)

- Factors of safety associated with code allowable or design loads

The code-related factors of safety that are reflected in the margin analysis are dependent on the controlling load combinations and associated allowables. Types of these factors of safety are:

- Margin to yield
- Margin to critical buckling
- Minimum factors of safety as given in design criteria documents

These sets of margin factors are related to strength and are included with the strength margin factors.

3.3.2 Variable Strength Factors

The reserve strength inherent in a structure is a function of the material properties, ultimate capacity, and ability of the load to redistribute with the occurrence of localized failures within a given structure. The localized failures may or may not have significant effects on the seismic margin of the structure. This is a function of its design. Each of these items contributing to strength have variability and are discussed below. Note that strength factors do not consider energy loss due to inelastic ductile behavior. This additional margin is discussed in Subsection 3.3.5 (Inelastic Energy Absorption).

3.3.2.1 Material Properties

Mechanical material properties of steel as well as the compressive strength of concrete has variability. The statistical estimates of the mean and standard deviation of the material properties are available in the public domain. References 6 and 8 provide summary tables of some of the statistical data available in the literature for steel and reinforced concrete. This information is used to define lognormal distribution parameters (mean and standard deviation). No increase in the steel yield stress due to the application of dynamic loads is considered in the seismic margin assessment. Variation in material properties can be given in terms of coefficients of variation. The coefficients of variation are usually less than 0.2. Therefore, the lognormal standard deviation can be considered equal to the coefficient of variation with less than 2 percent error (Subsection 3.3.2 of Reference 17).

These material property data are also used in defining other margin factors related to strength as defined in the following subsections.

3.3.2.2 Ultimate Capacity

Ultimate capacity is a function of the controlling structural element related to collapse or limit load. It is a function of the material (steel or concrete), load resistance behavior (i.e., shear, compression, bending), and design characteristics (i.e., determinate or indeterminate structures) that allows for load redistribution. The strength factor (Fs), which is a function of the ultimate capacity, is defined as:

$$Fs = [\text{Ultimate Capacity - Normal Load}] / (\text{Design Basis Seismic Load})$$

To define the strength factor, the controlling failure mode must be defined so that the ultimate capacity can be calculated. This is a function of the design Fragility parameters associated with concrete shear walls are discussed below for the AP600 plant.

3.3.2.2.1 Concrete Shear Wall

The ultimate capacity can be related to a shear failure. For the AP600, two types of shear walls are common to the design: low-rise concrete shear walls with boundary elements; and concrete containment tangential shear capacity. Their fragility parameters are discussed below.

Low-Rise Concrete Shear Walls

An expression for the ultimate shear capacity per unit length of low-rise shear walls that can be found in nuclear power plants has been developed by Barda (Reference 11). It is reproduced below, as defined in Subsection 6.2.1 of Reference 9, using English units (i.e., lb, in.):

$$Vu = \varphi\, d\, v_u \quad (3)$$

$$v_u = v_c + v_s \quad (4)$$

where:

v_c = $[8.3 - 3.4(hw/Lw - 0.5)]\, (f'_{cm})^{1/2}$
v_s = $\rho_n\, f_{ym}$

and:

$$v_u = \{[8.3 - 3.4(hw/Lw - 0.5)]\ (f'_{cm})^{1/2}\} + \rho_n\, f_{ym} \qquad (5)$$

where:

Vu	=	Ultimate shear capacity
φ	=	Strength uncertainty factor
d	=	Effective depth
v_u	=	Ultimate shear stress
hw	=	Wall height
Lw	=	Wall length
f'_{cm}	=	Median concrete ultimate compression strength
ρ_n	=	Vertical steel reinforcement ratio
f_{ym}	=	Median steel yield strength

An additional term can be added to Equation 5 that is representative of additional concrete strength with the addition of compressive axial load, N. This term is (References 5 and 10):

$$N / (4\ Lw\ t) \qquad (6)$$

The term t represents the thickness of the shear wall. The value of N is positive when the axial load is compressive. In the seismic margin analysis performed for the AP600 plant, the value of N is conservatively assumed to be zero since seismic loads will decrease the portion of N, which is due to dead weight.

As stated in Reference 9, page 172, the median ultimate shear capacity is defined by Equation 5 with φ equal to 1.0. Reference 9, page 172, gives a logarithmic standard deviation of 0.15 for the variability in ultimate shear capacity. This is similar to considering the capacity with φ equal to 0.85 to be about one standard deviation from φ equal to 1.0. This standard deviation is related to uncertainty (βu) since the equation is based on data curve fitting (same approach used in Reference 5). Therefore:

$$\beta u = 0.15 \qquad (7)$$

Note that the actual material properties should be used to determine the strength factor using Equation 5. Since actual material properties are not known, an estimate of the median values are made (Subsection 3.3.2.1). As stated in Reference 9, page 154, the variability in material properties (standard deviation) is considered random (βr) when calculating strength. Since Equation 5 is a function of both concrete compressive strength and steel yield stress, the standard deviation due to material must reflect a weighted average of the associated standard deviations related to concrete and steel material strengths. Such a weighted expression is given in Reference 5, page 3-15, and based on the lognormal distribution and the approximation that the lognormal standard deviation is equal to the coefficient of variation.

$$\beta m = \beta r = [\{\beta_{f'c}(v_c)\}^2 + \{\beta_{fy}(v_s)\}^2]^{1/2} / v_u \quad (8)$$

where:

$\beta_{f'c}$ = Standard deviation associated with concrete strength, f'c
β_{fy} = Standard deviation associated with steel yield strength, f_y
v_c , v_s , v_u = Median values as defined above

In Reference 12, page 2, it is stated that "structural walls designed according to the 1971 American Concrete Institute Building Code will attain their design strength in both flexure and shear." Further, on page 77 of Reference 12, conclusions based on test results show that the shear capacity is equal to or less than that associated with flexure strength. Since the Category I concrete structures are designed to ACI 349 and the design requirements in this code for shear and moment have not significantly changed from those given in the 1971 ACI, it is realistically assumed that shear is controlling and it is not necessary to examine the ultimate moment capacity of shear walls.

Tangential Concrete Containment Shear Capacity

As stated in Reference 15, the unit ultimate tangential shear strength, v_u, can be expressed as:

$$v_u = v_c + v_{so} + v_{si} \quad (9)$$

where:

v_c = Unit tangential shear strength provided by concrete
v_{so} = Unit tangential shear strength provided by orthogonal (hoop and Meridional) reinforcement
v_{si} = Unit tangential shear strength provided by diagonal reinforcement

The term v_{so} can be expressed as follows based on Reference 13:

$$v_{so} = \{[\rho_h + \rho_m]/2\} fy - \{[\sigma_h + \sigma_m]/2\} \quad (10)$$

where:

ρ_h and ρ_m = Hoop and meridional reinforcement ratios, respectively
σ_h and σ_m = Containment wall hoop and meridional (tension positive) stresses, respectively
fy = Yield stress capacity of the reinforcing steel

The term v_{si} is expressed as:

$$v_{si} = \rho_i \, fy \tag{11}$$

where ρ_i is the diagonal reinforcement ratio.

A significant amount of testing has been performed on scale models of reinforced concrete containment structures to define tangential shear capacity given cyclic loadings (References 13 and 14). Using the results presented in Reference 13, Figure 15, an equation can be given that defines a median estimate of v_u in terms of psi:

$$v_u = 0.8 \, (f'_{cm})^{1/2} + v_{so} \leq 21.1 \, (f'_{cm})^{1/2} \tag{12}$$

Estimates of the median material properties are used in place of the actual material properties in the same manner as that described for Equation 5.

The variability (standard deviations) is represented by both random and uncertainty parameters. The variability in material properties (Subsection 3.3.2.1) is used to calculate the random standard deviation, βr. If the upper limit of Equation 12 is not controlling, the random standard deviation is defined following the weighted average method given by Equation 8 using the terms that are affected by material properties.

$$\beta r = [\{\beta_{f'c} \, (0.8 \, (f'_{cm})^{1/2})\}^2 + \{V_{so}\}^2]^{1/2} \, / \, V_u \tag{13}$$

If the upper limit of Equation 12 is controlling, then:

$$\beta r = \beta_{f'c} \tag{14}$$

To reflect in the seismic margin evaluation the strength reduction factor φ, which is equal to 0.85, an uncertainty standard deviation, βu, is introduced to reflect its effect. The strength reduction factor provides an 84-percent exceedance probability in the strength prediction equation. This is equivalent to a one standard deviation variability. Therefore:

$$\beta u = \ln(1/0.85) = 0.16 \tag{15}$$

Note that Reference 15 reports, based on referenced tests, that the "suggested maximum unit shear strength for concrete containments with orthogonal reinforcement may reach the following value without concrete crushing or sliding shear failures."

$$(v_u)_{max} = 0.25 \, f'_c \tag{16}$$

Considering the upper limit value given in Equation 12 to control, the HCLPF value for v_u is the following considering both βu as defined by Equation 15, and the variability in concrete compressive strength. From Reference 6, Table 3, a lognormal standard deviation for a reinforced concrete containment wall is defined as 0.10. Therefore, recognizing that V_u is a function of the square root of the concrete compressive strength:

$$\beta r = 0.10 / 2 = 0.05$$

$$(v_u)_{HCLPF\ Value} = 21.1(f'_{cm})^{1/2}\ e^{[-1.65 \times (0.16 + 0.05)]} = 14.9\ (f'_{cm})^{1/2} \quad (17)$$

Considering f'_{cm} equal to 4000 psi, for this example:

$$(v_u)_{max} = 0.25\ f'_{cm} = 0.25 \times 4000 = 1000\ psi \quad (18)$$

$$(v_u)_{HCLPF\ Value} = 14.9(f'_{cm})^{1/2} = 14.9 \times (4000)^{1/2} = 942\ psi \quad (19)$$

Therefore, the value for $(v_u)_{max}$, equivalent to the conservative deterministic failure margin, is similar to $(v_u)_{HCLPF\ Value}$.

3.3.2.2.2 Structural Systems

Many of the AP600 structures, steel or concrete, have inherent reserve strength since a margin exists between the level determined to cause failure of a part of the structure and the level that causes the complete loss of structural integrity and safety function of the building or structure. When part of the structure begins to fail, it does not mean that the structure cannot adequately perform its safety function. Inherent reserve strength can exist due to redistribution of the load to another portion of the structure. The following method is used to reflect this behavior in the seismic margin evaluation.

Reserve Margin - Redundancy

A reserve margin redundancy factor (F) is defined by establishing the added capacity when a structural element has reached its limit defined by yield stress or plastic moment, or lost its capacity defined by buckling. Since the calculation of this factor is dependent on the structural system design, it is not possible to give a specific expression for these factors. The added seismic capacity is defined by the available capacity in adjacent structural elements or a plate structure when a portion of this structure is subject to a local failure. Load transfer is based on relative stiffnesses of adjacent elements, simplified limit analyses, or simplified plate and shell formulations. This factor is deterministic; however, reflecting median material property (Subsection 3.3.2.1), this factor is changed to a median capacity.

Variability - Redundancy with Material Variability

The material standard deviations are defined as given in Subsection 3.3.2.1. Since material variability is related to strength, it is treated as a random variable consistent with Subsection 3.3.2.2.1. An uncertainty is also present associated with redundancy. Considering that the capacity just prior to redistribution is representative of a conservative 98-percent failure probability, a standard deviation associated with uncertainty is defined using two standard deviations:

$$\beta u = \ln(F)/2 \qquad (20)$$

Example - Redundancy

An example is provided of the above method to define fragility parameters considering redundancy and material effects.

Based on an evaluation of the redundancy in a structural system that has a number of parallel shear walls, it is determined that the system can carry 25 percent more seismic load prior to loss of structural integrity. This evaluation is based on minimum concrete design material properties. The allowable stress is based on the square root of the concrete compressive strength. Using Reference 6, Table 3, the material median factor is 1.09 with a lognormal standard deviation, βm, of 0.15. Recognizing that the allowable stress is based on the square root of the concrete compressive strength, the random standard deviation for strength is defined as:

$$\beta r = \beta m/2 = 0.075 \qquad (21)$$

In summary:

Material Strength Factor = $(1.09)^{1/2}$ = 1.04
Redundancy Strength Factor = F = 1.25
$\beta u = \ln(1.25)/2 = 0.11$
$\beta r = 0.075$
Median Capacity Factor = $(1.09)^{1/2}$ x F = 1.04 x 1.25 = 1.3
HCLPF = 1.3 x $e^{[-1.65(0.075+0.11)]}$ = 0.96

3.3.3 Modeling and Analysis

Variability in the response characteristics are most likely due to uncertainty in the models, such as: as-designed and as-built conditions; boundary conditions between assumed and actual; weight distributions, etc. Variabilities and standard deviations are introduced into the fragility analysis to account for these effects. The modeling and analysis methods used for a nuclear power plant should be median-centered; however, variability is present. These variabilities are described below:

Analysis Error

In Reference 6, variability associated with analysis error that exists between in-place and design conditions is provided. Note that analysis error is subjective in Reference 6. Reference 6 focuses on containment reliability, however, the variabilities provided are considered to be representative of nuclear structures and analytical methods. Values given in Reference 6 are typical and are provided as a guide. Those applicable to the AP600 are given below with the lognormal standard deviation:

Axisymmetric; finite element model	0.08
Buckling	0.15

The standard deviations are considered equivalent to βc.

In an example given in Reference 6, coefficients of variations, equivalent to lognormal standard deviations, are given for randomness and uncertainty analysis error. These variabilities are:

$\beta r = 0.12$
$\beta u = 0.08$
$\beta c = [0.12^2 + 0.08^2]^{1/2} = 0.144$

These standard deviations are similar to the typical values given above when using βc as a comparison. Therefore, in the development of HCLPF values for the AP600 seismic margin assessment, these βr and βu values are used to represent potential analysis errors as appropriate.

Modeling Error

Reference 9, pages 143 to 145, discuss modeling. It is stated that "assuming that the analyst does his best job of modeling, modeling accuracy could be median-centered, with variability in each of the modeling parameters amounting to variability in calculated mode shapes and frequencies." These variabilities are related to uncertainty. The recommendations given in Reference 9 are used to reflect modeling errors.

Mode Shapes

The following standard deviations are used to reflect modeling errors in the dynamic model where mode shapes are used in the analytical method to calculate seismic loads:

Multidegree of freedom system model:	$\beta u = 0.15$
System that responds predominantly in one mode:	$\beta u = 0.10$

Modal Frequency Variability

Shifts in frequency affect spectral acceleration levels and introduce error. This is reflected in the seismic margin analysis by using a standard deviation calculated as the ratio of the spectral acceleration value considering a one-sigma variation in frequency and the spectral acceleration value at the median-centered frequency. The standard deviation of frequency, β, is estimated as 0.3. (References 8 and 9)

$$\beta u = \ln\{S_{f\beta} / S_f\} \qquad (22)$$

where:

$S_{f\beta}$ = AP600 design spectral acceleration value considering a one-sigma variation in frequency, f_β

S_f = AP600 design spectral acceleration value at median-centered frequency

f = median-centered frequency

f_β = one-sigma frequency variation frequency = $f\ e^{[(+/-)\beta]}$

3.3.4 Loading

Variations in the loading are included in the other factors that address spectral shape, modeling errors, and analysis errors. It is not necessary to specifically include additional variability in reference to loading.

3.3.5 Inelastic Energy Absorption

A ductile structure has additional margin due to its ability to absorb energy. Seismic design criteria impose ductile requirements. The AP600 design also requires ductile designs.

Margin Factor

The margin factor, Fμ, recommended by Riddel and Newmark is used (References 5, page 3-11; Reference 16, pages 3-17 to 3-19).

Region of Amplification:

$$F\mu = \{(q+1)\mu - q\}^{r} \qquad (23)$$

where:

μ = ductility factor
q = $3.0\zeta^{-0.30}$
r = $0.48\zeta^{-0.08}$
ζ = Elastic damping in percent of critical (ζ = 5% for AP600 seismic margin assessment)

Rigid Region (frequency $\geq$ 33 hz):

$$F\mu = \mu^{0.13} \quad (24)$$

Equations 23 and 24 for inelastic margin do not account for the hysteresis effect (pinching) present in concrete structures (e.g., shear walls). Time history studies have been conducted and are reported in Reference 16. Equations 23 and 24 are adjusted based on Reference 16 results (page 5-2) to account for the hysteresis effect in concrete structures in the region of amplification.

$$(F\mu)_{adj} = 1 + C_D (F_\mu - 1) \quad (25)$$

where:

$$C_D \approx 0.6$$

Variability

Variability is based on the recommendations given in Reference 18, pages 5-17 and 5-18:

$$\beta r = 0.11(F\mu - 0.50) \quad (26)$$

$$\beta u = \ln\{F\mu_{median} / F\mu_{-1\beta}\} \quad (27)$$

where:

$F\mu_{median}$ = $F\mu$ calculated based on μ_{median}
$F\mu_{-1\beta}$ = $F\mu$ calculated based on $\mu_{-1\beta}$ value

and $\mu_{-1\beta}$ compared to μ_{median} are (based on Reference 18, page 5-18):

μ_{median}	$\mu_{-1\beta}$
3.0	2.40
2.0	1.72
1.5	1.36

This is a linear relationship where:

$$\mu_{-1\beta} = 0.68 \ \mu_{median} + 0.36 \tag{28}$$

Equation 28 is used to define $\mu_{-1\beta}$.

Ductility

Ductility, μ, is determined based on the AP600 design characteristics and procedures given in the public domain. The ductility values established are considered median-centered.

Buildings

For building structures, ductility is established based on total system response. A representative median ductility factor is established from story drift based on permissible total story distortion as a percentage of story height. Reference 7 or similar document will be used to establish permissible story drift.

For the containment vessel, Reference 6 is used to establish strain limits of failure based on yield strain, and past buckling strains from which a ductility factor can be defined.

Steel Structures

Acceptable ductility factors for steel members will be used as defined in Reference 20, Appendix A.

Concrete Structures

Median ductility factors will be established using methods given in Reference 19 or using Reference 16 to define an acceptable ductility for shear walls. It is stated in Reference 16, page 3-10, that "for shear wall structures designed to remain essentially elastic for the design earthquake" (SSE) a ductility equal to 1.85 is appropriate. Higher ductility levels (ductility equal to 4.27) "represents a conservative lower bound on the deformations which correspond to significant strength degradation under a small number (3 to 5) of strong nonlinear response cycles which might occur during strong ground shaking. ... Thus, the low (1.85) and high (4.27) ductility levels considered in this study are believed to bound the ductility range of interest for nuclear plant shear wall type structures."

3.3.6 Testing

Fragility data is defined based on generic tests on similar equipment. The tests define known seismic levels for which the equipment will operate. Typically, seismic testing is performed to show that the equipment will operate under specified seismic conditions; it is not intended to show the highest level at which the equipment will operate. Thus, the reserve margin can only be estimated based on experience. The main cause of equipment failure during a seismic event is failure of the devices mounted therein to operate, commonly called chatter.

In a number of cases, the expected seismic capacity of the equipment with solid-state electronics has been based on previous seismic testing of similar equipment. The AP600 cabinets used for this equipment will be similar of design to those tested and qualified previously. In addition, the electronics used will be of a more recent generation.

The equipment rarely fails to perform its intended function because of structural failure. Based on past experience it has been concluded that a reasonable estimate for the median seismic margin factor (covering both strength and operability) is 1.1. Assuming that the seismic level associated with the equipment being tested has a 98 percent failure probability, an uncertainty, βu, can be defined using two standard deviations between the test level and the median level. Therefore:

$$\beta u = \ln(1.1)/2 = 0.048 \quad (29)$$

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