RIPB Seismic Safety Approach (Integration of ASCE 43 Design Criteria with the LMP Framework)

SOUTHWEST RESEARCH INSTITUTE®

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- Reported results are preliminary, and part of an ongoing research program.
- The expressed views do not necessarily reflect the views or regulatory position of the U.S. Nuclear Regulatory Commission.

Part 2 – Demonstration of Feasibility through Simple Example Problems

Three Examples

- Example 1: Simple structural element (a typical interior shear wall) is designed using selected combinations of SDC and LS categories and using the ASCE 43 and 4 standards. The same element is also designed using the conventional approach. Fragilities are developed for each case and compared and used to compute failure probabilities.
- Example 2: Generic fragility calculations are performed for selected combinations of SDC and LS categories using the assumptions outlined in the ASCE 43 and 4 standards with respect to performance goals.
- Example 3: Simple examples of selected individual sequences exhibit effects of alternative selections of ASCE 43 SDC and LS categories for design. These results for individual sequences are shown as frequencies and consequences (either, doses or core damage). The results demonstrate, at a very conceptual level, whether the proposed new approaches to design are feasible, and the associated variation of event sequence frequencies.

Example 1 – Shear Wall

- Shear walls are major structural elements in low aspect nuclear power plant structures that are used to resist seismic loads;
- Our simplified shear wall represents a relatively common design element that can be used to evaluate the various combinations of SDC and LS and resulting fragilities within the ASCE 43 and ASCE 4 design framework;
- In several past and recent SPRAs, shear wall failures under seismic loads have been a significant contributor to both CDF and LERF; and
- Results from this simplified problem provide useful insights into how to adjust existing designs (and hence SSC fragilities) to incorporate the various combinations of SDC design ground motions and damage limit states.

Shear Wall Characteristics

- The shear wall is assumed located on a hard rock site (site A in the report)
- The shear wall dimensions are typical for interior shear walls in a nuclear power plant, with an aspect ratio less than two;
- The initial shear wall resonant frequency is around 8 Hz. We placed additional mass at the top of the wall to obtain the desired fundamental frequency and substantial in-plane shear forces;
- Only in-plane failure modes and designs were explored, and the top mass was assumed to be restrained for the out-of-plane motion;
- Only in-plane and vertical excitations were considered. The design motions are in accordance with the DRS for various SDC categories
- The height and width dimensions of the shear wall were fixed in our sensitivity studies, but the reinforcement ratios and thicknesses were varied to account for chosen combinations of SDC levels and LS categories.
- Walls were designed to ASCE 43 and 4, and ACI codes

Shear Wall Cases Examined

- The following ASCE 43 SDC and LS categories are evaluated
  - SDC-5 and LS-D
  - SDC-5 and LS-C
  - SDC-4 and LS-D

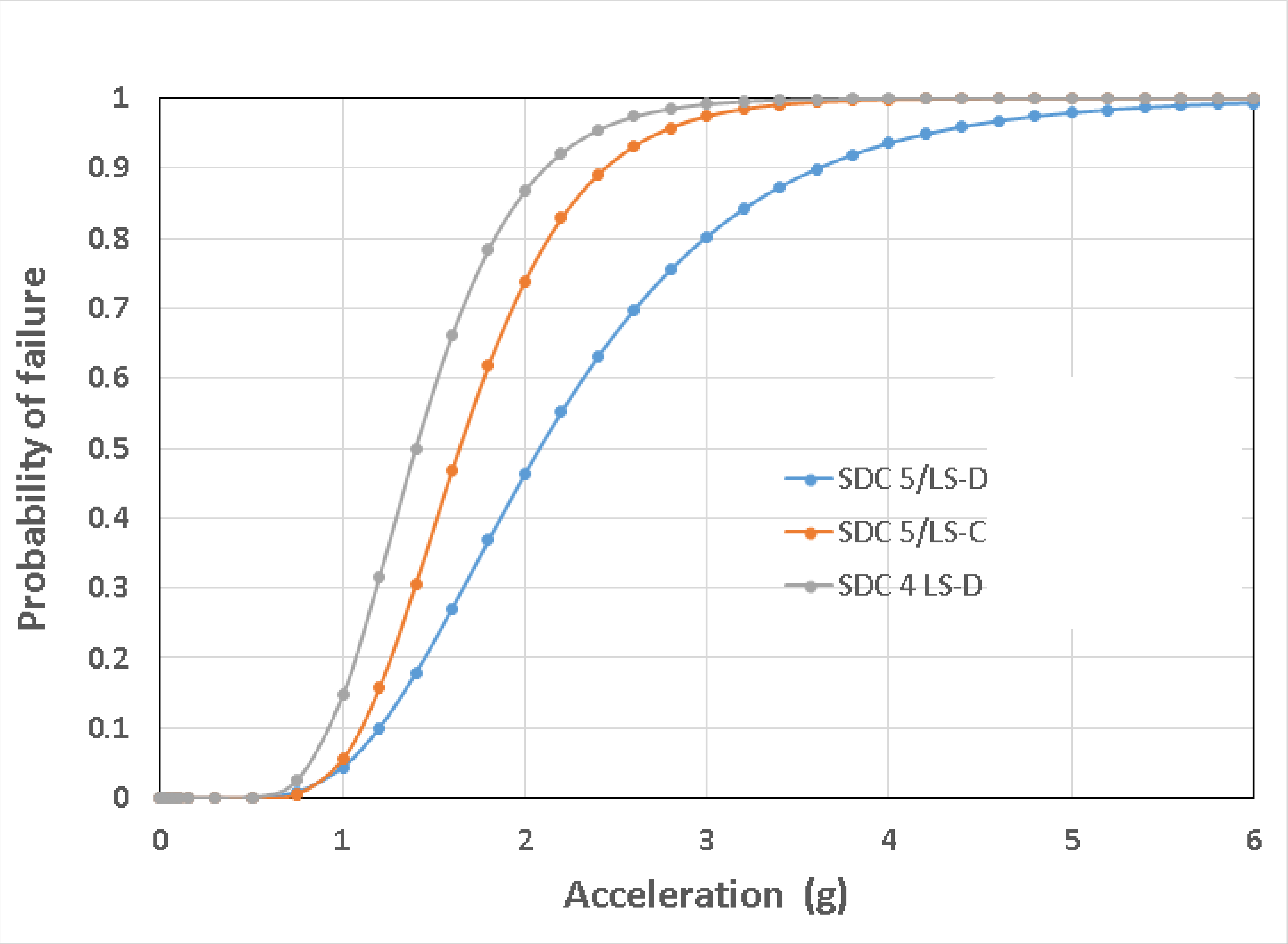
- RG 1.60 spectra anchored to site SSE PGA and traditional design criteria (LS-D)

Shear Wall Fragility Results for Cases Evaluated

A_m	2.08	1.64	1.40
β_R	0.25	0.27	0.25
β_U	0.28	0.19	0.21
βc	0.38	0.31	0.32
HCLPF	0.76	0.8	0.66
Failure Frequency/yr	2.86×10^{-6}	4.64×10^{-6}	6.87×10^{-6}
	SDC5/LS-D	**SDC5/LS C**	**SDC4/LS D**
Ratio	1.00	1.62	2.40

For Fμ = 2.0

Shear Wall Fragilities



Example 2 – Development of Fragilities based on Design Criteria

- Example of derivation of the fragilities of SSCs that are designed to ASCE 43 requirements
- It is assumed that the SSC will be designed to the full limits of design criteria
- For the sake of simplicity, SSC fragility is calculated in terms of median ground acceleration capacity (Am) and the composite variability (βc)
- Three cases are considered
  - Structural fragility
  - Equipment functional fragility
  - Anchorage fragility

ASCE 43 Design Criteria

Seismic design of SSCs according to ASCE 43 is summarized by the following steps:

1. Assume SCD 5 for safety related SSC
2. Performance goal for SDC 5, PF = 10^{-5} per year
3. DBE Design Response Spectrum (DRS) = SF × UHRS for PF where SF is the scale factor and UHRS is the uniform hazard response spectrum at exceedance frequency HP =PF
    - For a rock site selected DRS pga = 0.5 g
4. ASCE 43 specifies additional performance targets: 1% probability of unacceptable performance for DBE shaking and 10% probability of unacceptable performance at 1.5 DBE shaking.
5. Select a limit state

ASCE 43 Design Criteria (contd.)

- Perform seismic response analysis following ASCE 4: 80% probability of non-exceedance response given the DBE shaking
- Design structural elements (e.g., shear walls, beams, columns, tanks etc.) using ACI 349 and AISC codes as per ASCE 43
- For equipment qualified by testing, use the test response spectrum (TRS) as 1.33 times the Required Response Spectrum (RRS); RRS at the equipment mounting (floor) level is obtained for the DBE DRS and seismic response analysis per ASCE 4

Development of Structural Fragility

- Assume a shear wall in a safety related building in the plant. Its median ground acceleration capacity can be written as:
  - $Am = F_T \times \text{DBE PGA}$
  - where $F_T = F_{Strength}\ F\mu\ F_R$
- Strength factor reflects the uncertainty in the material property (reinforcing steel) and in the shear failure formula based on EPRI TR-103959
  - $F_{mat} = 1.20$; $\beta c = 0.10$
  - $F_{formula} = 2.0$; $\beta c = 0.20$
  - Inelastic energy absorption factor $F\mu = 1.80$; $\beta c = 0.20$
- Response factor FR is obtained by invoking the ASCE 4-16 goal of achieving the 80% probability of non-exceedance of response for DBE shaking
  - $F_R = \exp(0.842\ \beta_R)$ where $\beta_R = 0.35$
  - $F_R = 1.34$
  - Total Factor of Safety = $1.20 \times 2.00 \times 1.80 \times 1.34 = 5.80$; $\beta c = 0.46$
- The median ground acceleration capacity of the shear wall designed to Limit State D is given by (5.8×0.50 =) 2.9 g
- HCLPF Capacity = 1.0 g

Assess the fragility if the shear wall is designed to Limit State C

- If the shear wall is designed to Limit State C, the design demand is reduced by a factor representing the inelastic energy absorption (see Eq. 5-1a of ASCE 43). All other things being the same, the median ground acceleration capacity will also be reduced by this factor
- Table 5-1 of ASCE 43 gives this reduction factor as 1.5 for Limit State C
- Therefore, the median ground acceleration capacity of the shear wall designed to Limit State C is given by (2.9 g/1.5 =) 1.93 g
- HCLPF capacity = 1.0 g/1.5 =0.67 g
- Note by designing to Limit State C, the shear wall will have less reinforcement (other design features such as span, height and wall thickness may not change). Designing for a lower limit state would generally result in cost savings

Assess the fragility if the shear wall is designed to SDC 4

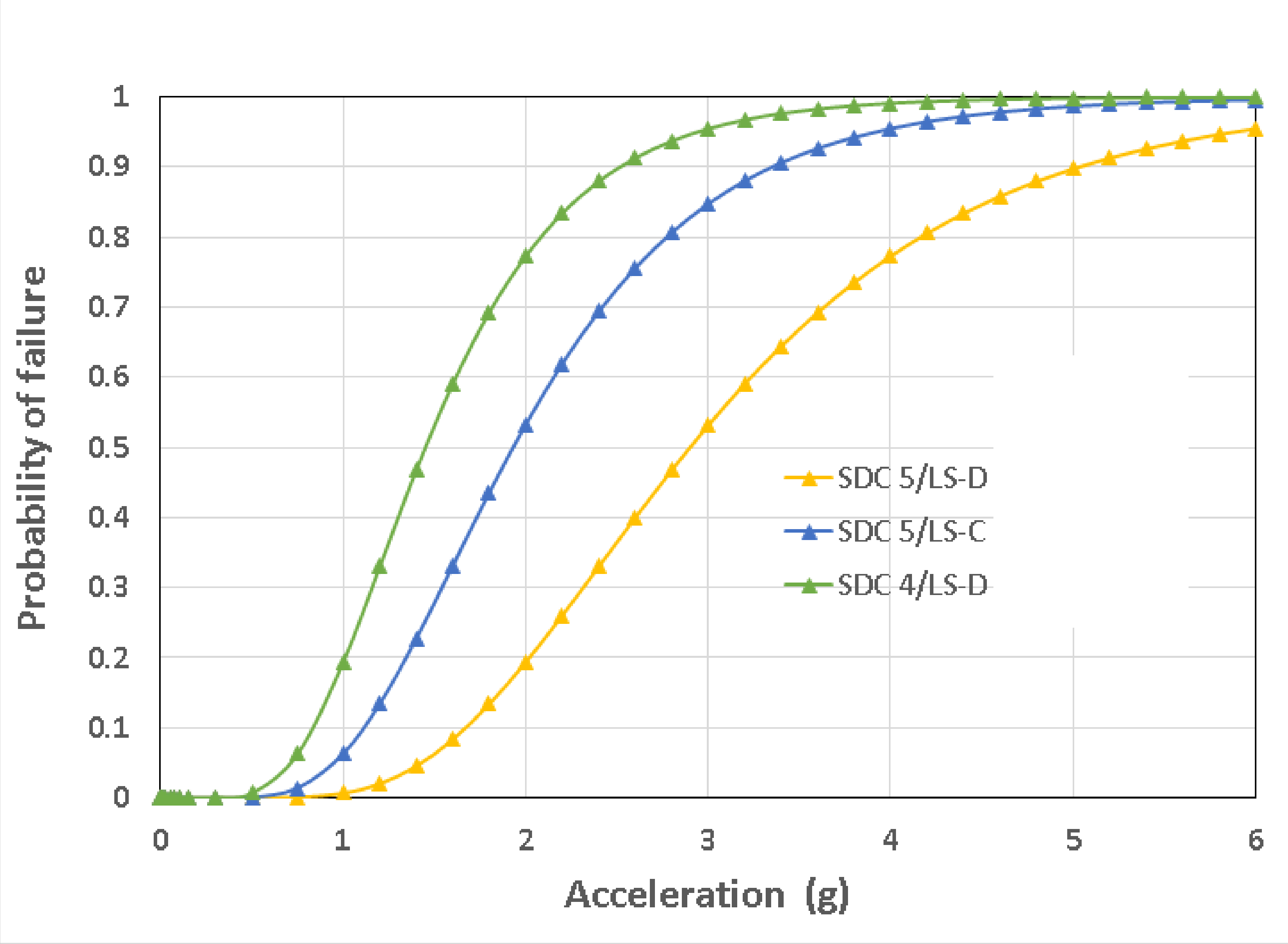
- If the shear wall is designed at SDC 4 for Limit State D, the input to the seismic demand analysis will be based on the performance goal of 4×10^{-5} per year. The DBE PGA is 0.25g
- The median ground acceleration capacity of the SCD 4 shear wall is given by (2.9 g × (0.25/.50) =) 1.45g
- HCLPF capacity = 0.50 g
- Note by designing the wall as SDC 4, the shear wall will have less reinforcement (other design features such as span, height and wall thickness may not change) since the DRS input is lower
- Designing for a lower SDC would generally result in cost savings

Comparison of Failure Frequencies

- The shear wall fragility is convolved with the site-specific seismic hazard to obtain the failure frequency

Design Criteria	Median Capacity PGA, g	Failure Frequency/yr
SDC 5 LS D	2.90	1.31×10^{-6}
SDC 5 LS C	1.93	3.89×10^{-6}
SDC 4 LS D	1.45	7.73×10^{-6}

Shear Wall Fragilities (Example 2)



Example 3 – Evaluate Selected Event Sequences to Examine Impacts of Alternate SDC and LS Categories

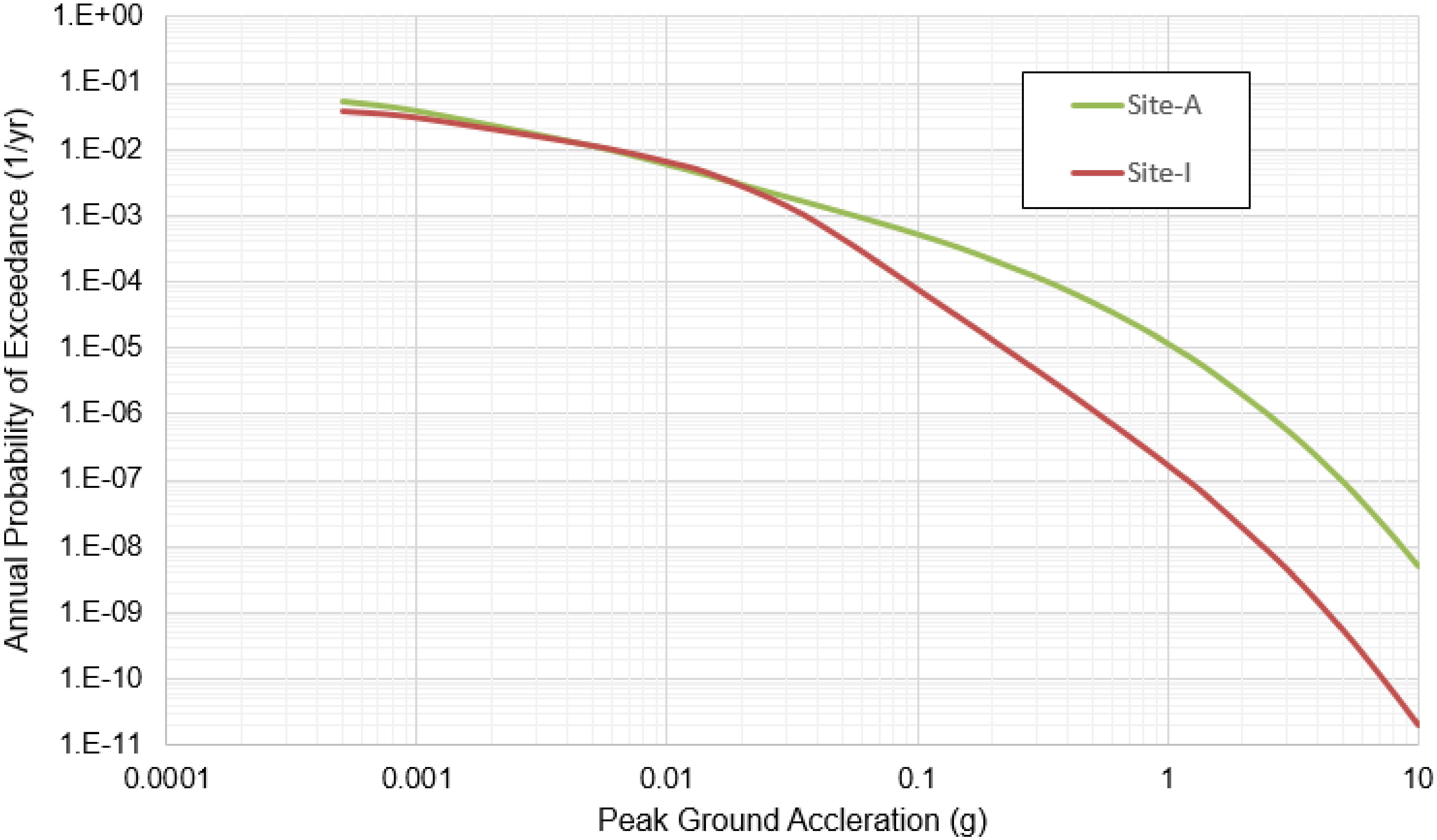
- Use available PRA information to develop simplified functionally coherent event sequences for an advanced non-LWR design and a large LWR design;
- Select and simplify the sequences that result in consequences in-terms of doses or core damage;
- Select initial generic fragility values as if the design reflects current seismic design criteria (i.e. SDC 5 and LS D of ASCE 43). This is the base case;
- Select hazard curves for two sites for quantification of the event sequences;
- Select alternative SDC and LS categories to evaluate changes in the risk quantification;
- Revise fragilities of components to reflect the designs conducted to alternative selection of SDCs and LSs; and
- Quantify event sequence results and compare them to base case to evaluate how much change in introduced by using the LMP/ASCE 43 Integration Approach

Caveats

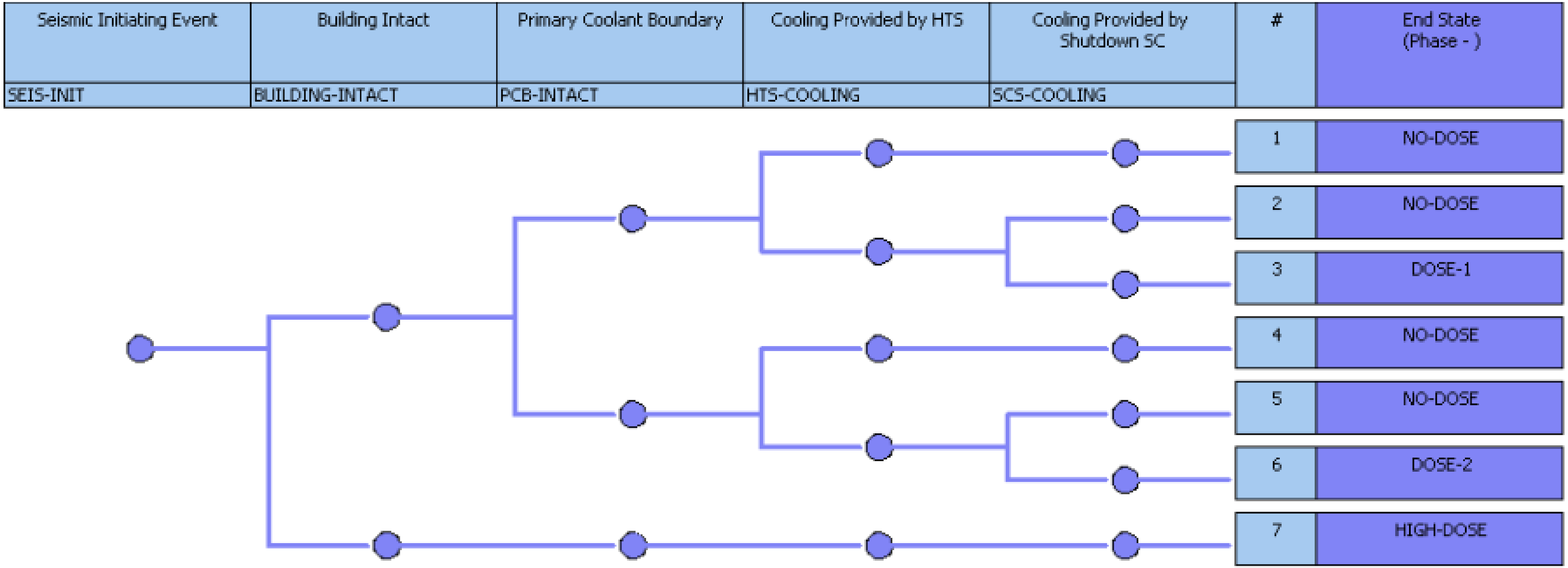
- These simple examples do not explore the following questions:
  - Generic vs. site-specific design;
  - Effect on cumulative risk;
  - Changes in risk insights, such as changes in dominant sequences, dominant contributors, and non-seismic failures;
  - Complex decision and implementation challenges; and
  - Impact of other regulatory and technical considerations.

- These questions will be explored in next phase

Hazard Curves for Site A and I



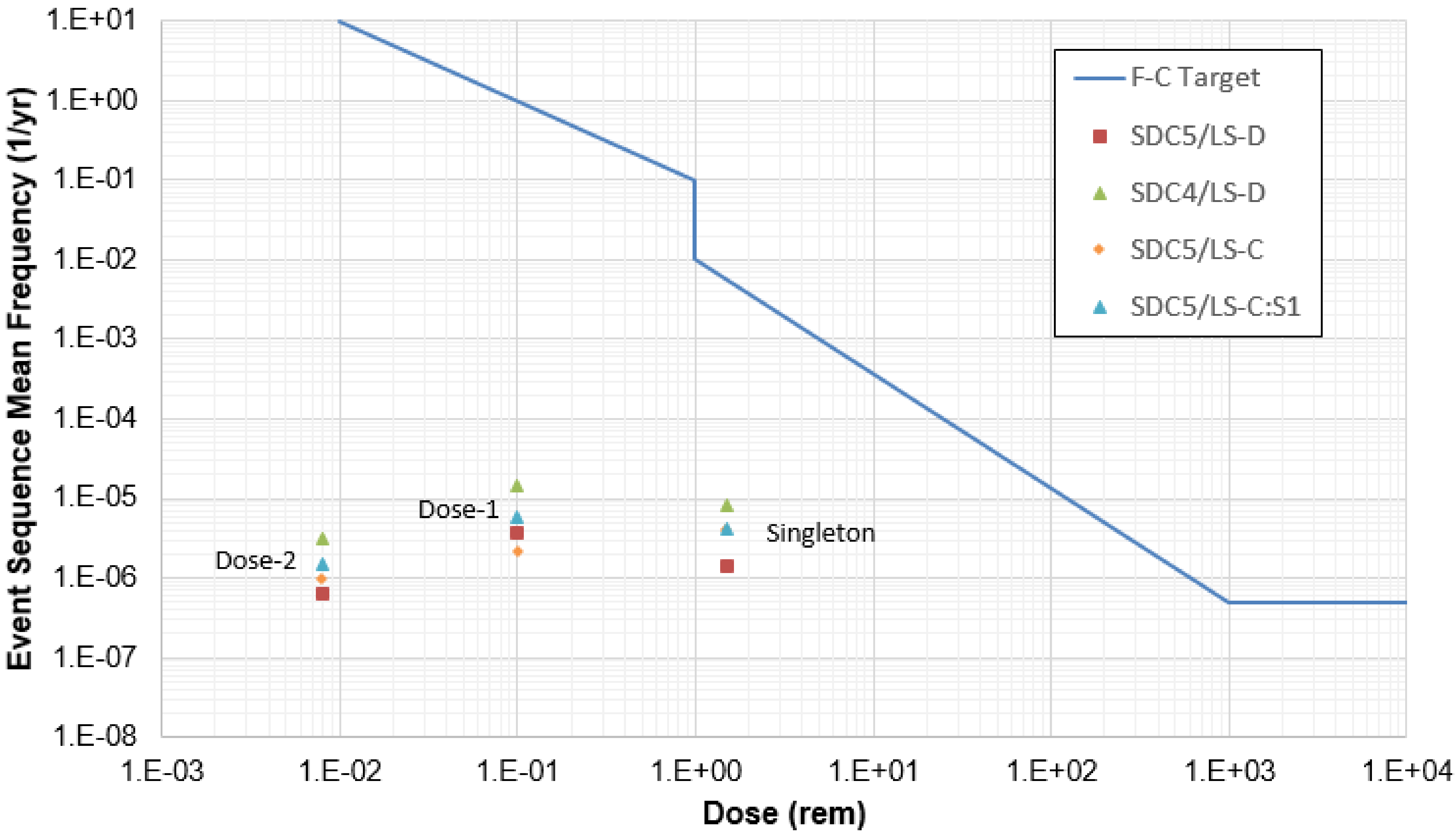
Simplified Event Tree for a Hypothetical Advanced Reactor



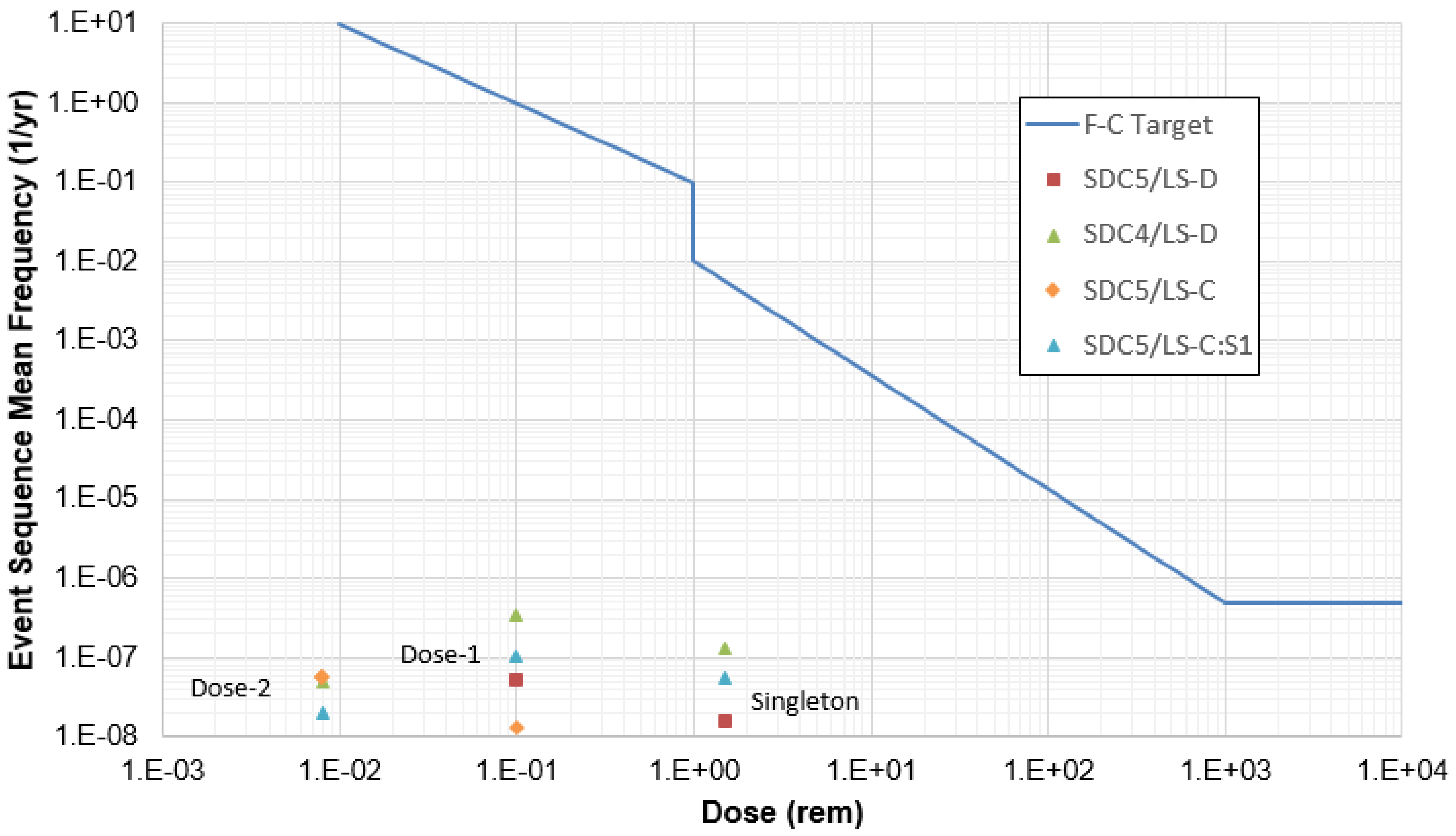
Fragilities for Three Design Options

Case A Hazard Data; Three Design Options								
	LMP Design 1 SDC5/LS-D		LMP Design 2 SDC4/LS-D		LMP Design 3 SDC5/LS-C		LMP Design 3 SDC5/LS-C Sensitivity S1	
	A_m	β_C	A_m	β_C	A_m	β_C	A_m	β_C
Shear Wall	2.9	0.43	1.45	0.46	1.93	0.43	1.93	0.43
Primary Boundary	2.9	0.43	1.45	0.46	1.93	0.43	1.93	0.43
HTS Cooling	1.24	0.40	0.62	0.4	1.24	0.4	0.93	0.4
SCS Cooling	1.24	0.40	0.62	0.4	1.24	0.4	0.93	0.4

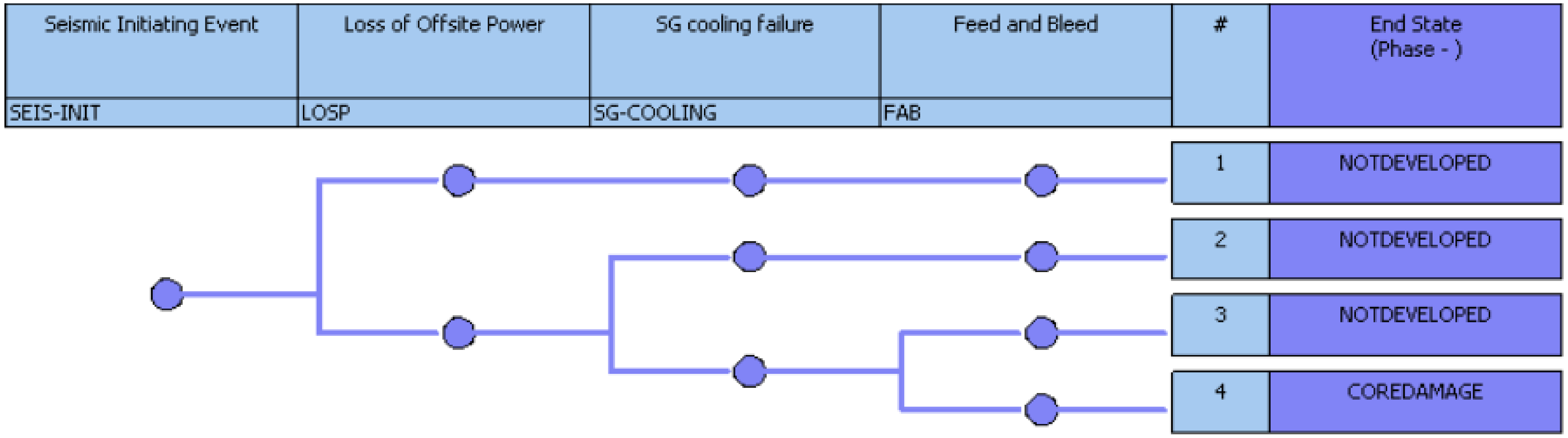
F-C Curve Generic Design and Site – A

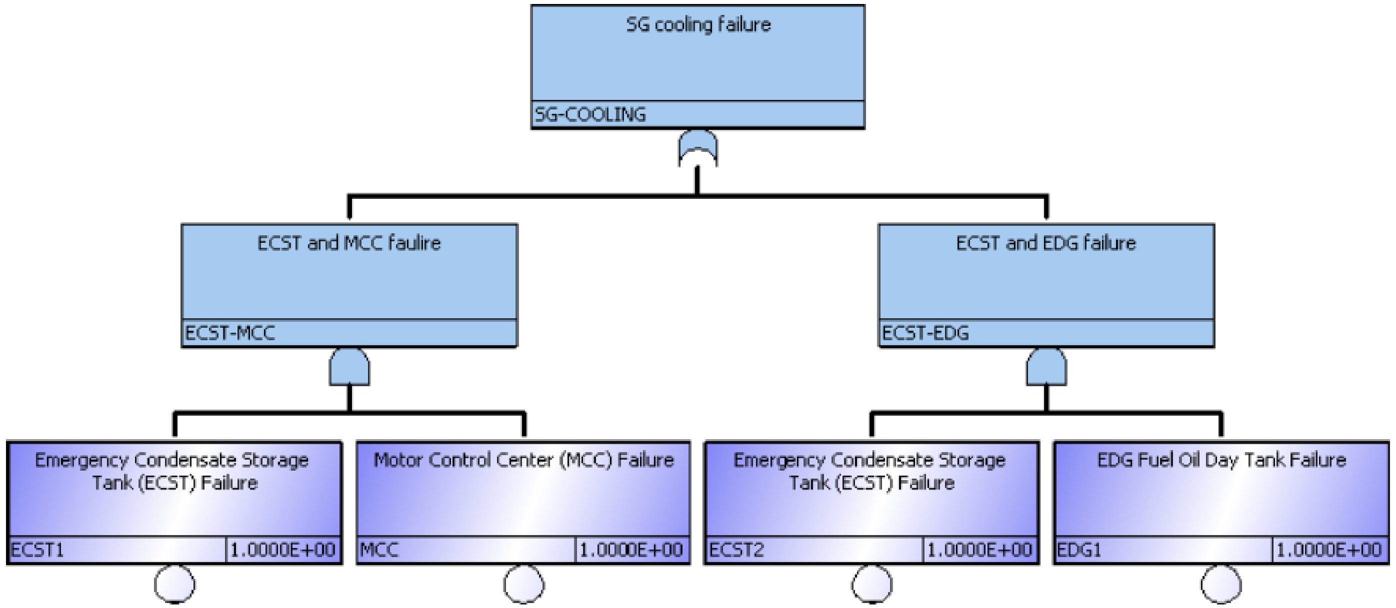


F-C Curve Generic Design and Site – I



Example of Simplified PWR Event SEQ-1





Results from Three sequence Site A –PWR Example

		Event Sequence Frequency		
Sequence	**Cut-Sets**	**SDC 5/LSD**	**SDC 4/LSD**	**SDC5/LSC**
SEQ-1	SEQ-1= $f_{LOSP}\ f_{MCC}\ f_{SG\text{-}COOLING}$ $f_{SG\text{-}COOLING}=(f_{ECST}\&f_{MCC})$ OR $(f_{ECST}\&f_{EDG})$	2.61×10^{-6}	1.33×10^{-5}	5.40×10^{-6}
SEQ-2	SEQ-2= $f_{LOSP}\ f_{EDG}\ (f_{ECST}$ OR $f_{TDAFW})$	2.18×10^{-6}	1.12×10^{-5}	2.25×10^{-6}
SEQ-3	SEQ-3= $f_{RVP}\ f_{ECST}$	6.06×10^{-6}	8.67×10^{-6}	6.05×10^{-6}
LOSP= Loss of Off-site Power MCC= Motor Control Center ECST= Emergency Condensate Storage Tank		EDG = Emergency Diesel Generator RVP = Reactor Vessel internal TDAFW = TD Auxiliary Feed Water pump		

Results from Three sequence Site I –PWR Example

		Event Sequence Frequency		
Sequence	**Cut-Sets**	**SDC 5/LSD**	**SDC 4/LSD**	**SDC5/LSC**
SEQ-1	SEQ-1= $f_{LOSP}\ f_{MCC}\ f_{SG\text{-}COOLING}$ $f_{SG\text{-}COOLING}$ =(f_{ECST}&f_{MCC}) OR (f_{ECST}&f_{EDG})	2.88×10^{-8}	2.16×10^{-7}	6.94×10^{-8}
SEQ-2	SEQ-2= $f_{LOSP}\ f_{EDG}$ (f_{ECST} OR f_{TDAFW})	2.50×10^{-8}	1.87×10^{-7}	2.60×10^{-8}
SEQ-3	SEQ-3= $f_{RVP}\ f_{ECST}$	8.83×10^{-8}	1.51×10^{-7}	8.82×10^{-8}
LOSP= Loss of Off-site Power MCC= Motor Control Center ECST= Emergency Condensate Storage Tank		EDG = Emergency Diesel Generator RVP = Reactor Vessel internal TDAFW = TD Auxiliary Feed Water pump		

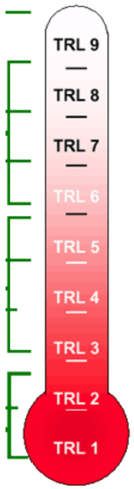
Preliminary Insights from Examples

- Other loads affect options for seismic design
- Response spectral shape may have significant impact on energy absorption factor
- Changes in probability of failure are relatively insensitive because of flat seismic hazard curves
- Reduction in the median capacity is not in direct proportion to the ratio of design ground motions
- Design for LS C will probably involve more iterative design work
- It is feasible to derive generic fragilities because of the ASCE 43 performance-based approach

Preliminary Insights from Examples

- The simple examination of selected event sequences shows that changes in frequencies of failure remain well within the acceptable range
- The frequency of individual sequences can go up and down and, therefore, it is crucial to analyze the entire SPRA model to evaluate impact on cumulative risk and risk insights (proposed Phase 2 activity)
- Changes in seismic design requirements are feasible
- Maturity of the SPRA methodology makes it feasible to demonstrate compliance with risk criteria.
- The proposed approach can be used both for a standard generic design and for a site-specific design

Risk-informed, performance-based design: past and present



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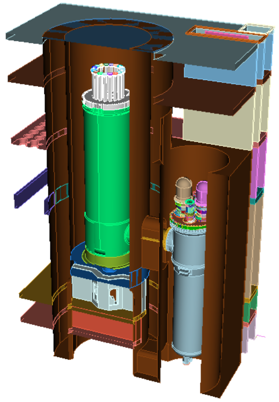


Outline

- ASCE/SEI Standards 4 and 43
- Key PBEE nuggets
- PBEE, Revision 0, 1981: Nuclear
- PBEE, Revision 1, 1992-1997: Buildings
- PBEE, Revision 2, 2009: Nuclear
- PBEE, Revision 3, 2000-2012: Buildings
- $RIPB_d$, 2020: nuclear
  - Seismic isolation, ARPA-E, nonlinear analysis, standards

ASCE/SEI Standards 4 and 43

- ASCE/SEI Standard 4
    - Analysis of safety-related nuclear structures
    - ASCE 4-16 superseded 4-98
- ASCE/SEI Standard 43
    - Design of safety-related nuclear structures
    - ASCE 43-19 will supersede 43-05
        - Scheduled for publication in Q4 of 2020
        - Chapter on seismic isolation
        - Seismic design categories and limit states

ASCE/SEI Standard 43

Table 1-1. Summary of Earthquake Design Provisions.

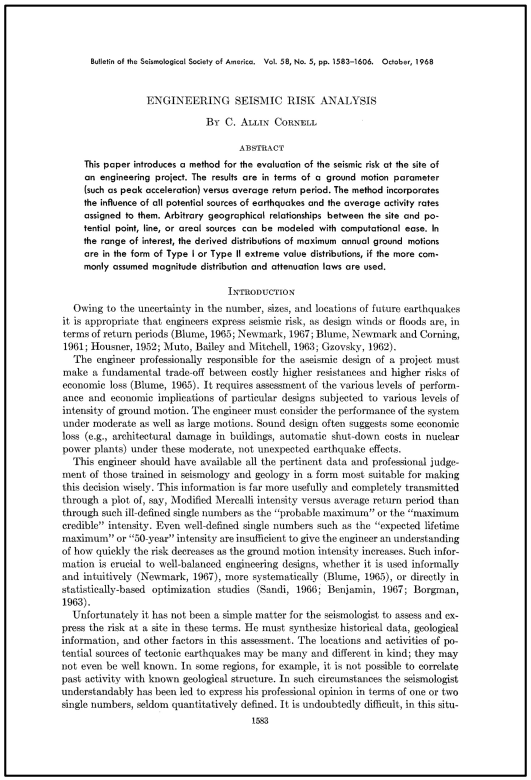
	Seismic Design Category			
	2	**3**	**4**	**5**
Target performance goal, P_F	4×10^{-4}	1×10^{-4}	4×10^{-5}	1×10^{-5}
DBE response spectrum or acceleration time series	SF $\times$ UHRS; Chapter 2			
Damping for structural evaluation	Section 3.3.3			
Analysis methods for structures	ASCE 4 and Chapter 3			
Analysis methods for systems and components	In-structure response spectra; ASCE 4 and Chapter 8 in this standard			
Load factor	1.0			
Inelastic energy absorption factors	Table 5-1 and/or Table 8-1 in this standard			
Material strength	Minimum specified value			
Component design strength	Design strength according to materials standards unless exceptions are made in this standard			
QA program	Chapter 10			
Independent peer review	Chapter 10			

Table 1-2. Deformation and Damage by Limit State.

Limit State	Expected Deformation	Expected Damage
A	Large permanent distortion, short of collapse	Significant damage
B	Moderate permanent distortion	Generally reparable
C	Limited permanent distortion	Minimal damage
D	Essentially elastic behavior	Negligible damage

Source: Adapted from ANS 2.26 (ANS 2017).

Key PBEE nuggets


Bulletin of the Seismological Society of America. Vol. 58, No. 5, pp. 1583–1606. October, 1968

ENGINEERING SEISMIC RISK ANALYSIS

By C. Allin Cornell

Abstract

This paper introduces a method for the evaluation of the seismic risk at the site of an engineering project. The results are in terms of a ground motion parameter (such as peak acceleration) versus average return period. The method incorporates the influence of all potential sources of earthquakes and the average activity rates assigned to them. Arbitrary geographical relationships between the site and potential point, line, or areal sources can be modeled with computational ease. In the range of interest, the derived distributions of maximum annual ground motions are in the form of Type I or Type II extreme value distributions, if the more commonly assumed magnitude distribution and attenuation laws are used.

Introduction

Owing to the uncertainty in the number, sizes, and locations of future earthquakes it is appropriate that engineers express seismic risk, as design winds or floods are, in terms of return periods (Blume, 1965; Newmark, 1967; Blume, Newmark and Corning, 1961; Housner, 1952; Muto, Bailey and Mitchell, 1963; Gzovsky, 1962).

The engineer professionally responsible for the aseismic design of a project must make a fundamental trade-off between costly higher resistances and higher risks of economic loss (Blume, 1965). It requires assessment of the various levels of performance and economic implications of particular designs subjected to various levels of intensity of ground motion. The engineer must consider the performance of the system under moderate as well as large motions. Sound design often suggests some economic loss (e.g., architectural damage in buildings, automatic shut-down costs in nuclear power plants) under these moderate, not unexpected earthquake effects.

This engineer should have available all the pertinent data and professional judgement of those trained in seismology and geology in a form most suitable for making this decision wisely. This information is far more usefully and completely transmitted through a plot of, say, Modified Mercalli intensity versus average return period than through such ill-defined single numbers as the "probable maximum" or the "maximum credible" intensity. Even well-defined single numbers such as the "expected lifetime maximum" or "50-year" intensity are insufficient to give the engineer an understanding of how quickly the risk decreases as the ground motion intensity increases. Such information is crucial to well-balanced engineering designs, whether it is used informally and intuitively (Newmark, 1967), more systematically (Blume, 1965), or directly in statistically-based optimization studies (Sandi, 1966; Benjamin, 1967; Borgman, 1963).

Unfortunately it has not been a simple matter for the seismologist to assess and express the risk at a site in these terms. He must synthesize historical data, geological information, and other factors in this assessment. The locations and activities of potential sources of tectonic earthquakes may be many and different in kind; they may not even be well known. In some regions, for example, it is not possible to correlate past activity with known geological structure. In such circumstances the seismologist understandably has been led to express his professional opinion in terms of one or two single numbers, seldom quantitatively defined. It is undoubtedly difficult, in this situ-

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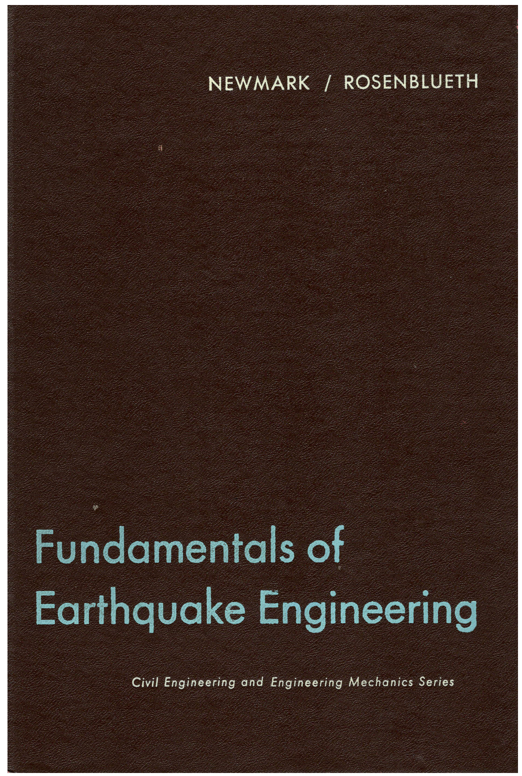

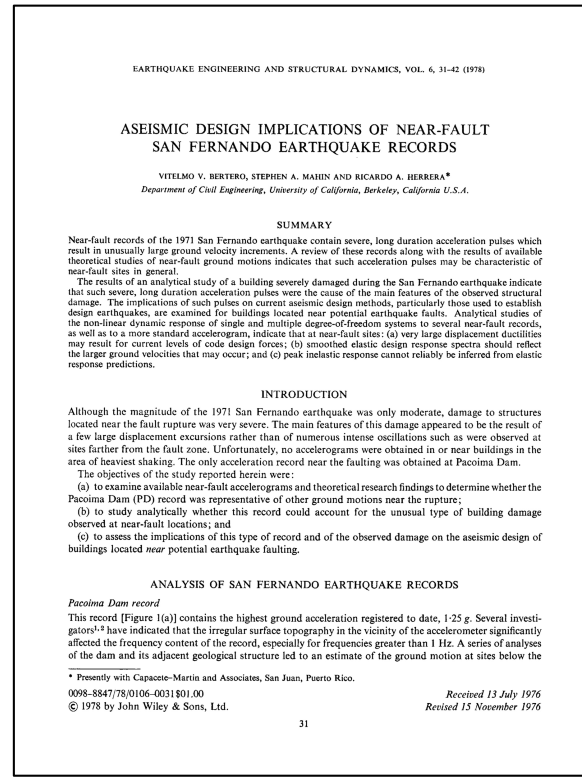
EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS, VOL. 6, 31–42 (1978)

ASEISMIC DESIGN IMPLICATIONS OF NEAR-FAULT SAN FERNANDO EARTHQUAKE RECORDS

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SUMMARY

Near-fault records of the 1971 San Fernando earthquake contain severe, long duration acceleration pulses which result in unusually large ground velocity increments. A review of these records along with the results of available theoretical studies of near-fault ground motions indicates that such acceleration pulses may be characteristic of near-fault sites in general.

The results of an analytical study of a building severely damaged during the San Fernando earthquake indicate that such severe, long duration acceleration pulses were the cause of the main features of the observed structural damage. The implications of such pulses on current aseismic design methods, particularly those used to establish design earthquakes, are examined for buildings located near potential earthquake faults. Analytical studies of the non-linear dynamic response of single and multiple degree-of-freedom systems to several near-fault records, as well as to a more standard accelerogram, indicate that at near-fault sites: (a) very large displacement ductilities may result for current levels of code design forces; (b) smoothed elastic design response spectra should reflect the larger ground velocities that may occur; and (c) peak inelastic response cannot reliably be inferred from elastic response predictions.

INTRODUCTION

Although the magnitude of the 1971 San Fernando earthquake was only moderate, damage to structures located near the fault rupture was very severe. The main features of this damage appeared to be the result of a few large displacement excursions rather than of numerous intense oscillations such as were observed at sites farther from the fault zone. Unfortunately, no accelerograms were obtained in or near buildings in the area of heaviest shaking. The only acceleration record near the faulting was obtained at Pacoima Dam.

The objectives of the study reported herein were:

(a) to examine available near-fault accelerograms and theoretical research findings to determine whether the Pacoima Dam (PD) record was representative of other ground motions near the rupture;

(b) to study analytically whether this record could account for the unusual type of building damage observed at near-fault locations; and

(c) to assess the implications of this type of record and of the observed damage on the aseismic design of buildings located *near* potential earthquake faulting.

ANALYSIS OF SAN FERNANDO EARTHQUAKE RECORDS

Pacoima Dam record

This record [Figure 1(a)] contains the highest ground acceleration registered to date, 1·25 *g*. Several investigators[1,2] have indicated that the irregular surface topography in the vicinity of the accelerometer significantly affected the frequency content of the record, especially for frequencies greater than 1 Hz. A series of analyses of the dam and its adjacent geological structure led to an estimate of the ground motion at sites below the

* Presently with Capacete-Martin and Associates, San Juan, Puerto Rico.

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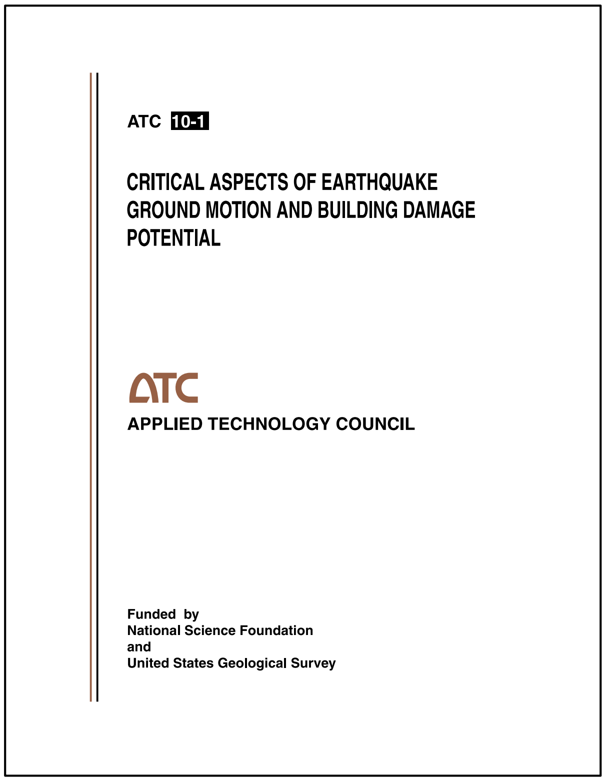
Key PBEE nuggets



ATC 10-1

CRITICAL ASPECTS OF EARTHQUAKE GROUND MOTION AND BUILDING DAMAGE POTENTIAL

ATC

APPLIED TECHNOLOGY COUNCIL

Funded by
National Science Foundation
and
United States Geological Survey

AN ALTERNATE SEISMIC DESIGN APPROACH

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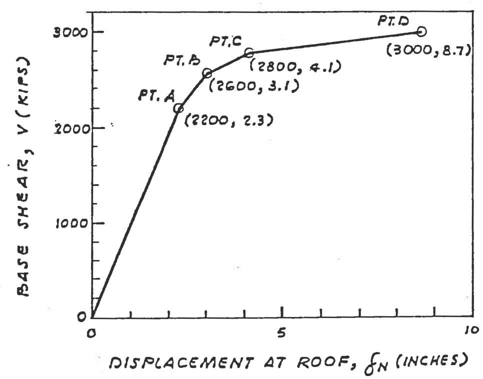
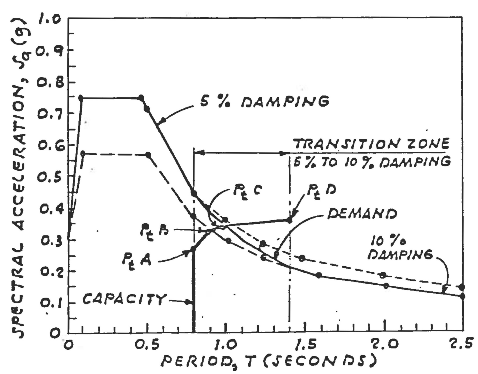
ALTERNATIVE DESIGN PROVISIONS

1. <u>General Provisions</u>

Buildings will be designed to resist two levels of earthquake motion. The first level of motion is designated EQ-1 and the second and larger amplitude of motion is designated EQ-II. The lateral force-resisting structural systems of these buildings will be designed to resist EQ-I by elastic, or nearly elastic, behavior as prescribed in the elastic design provisions. The buildings will be evaluated for their ability to resist EQ-II as prescribed in the post-yield analysis provisions.





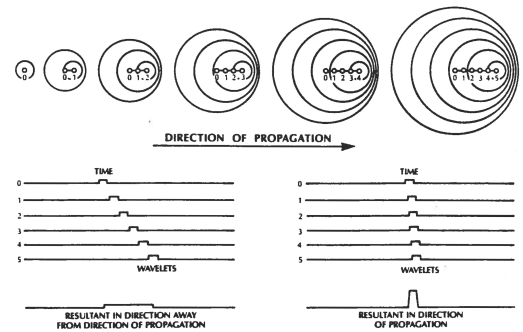
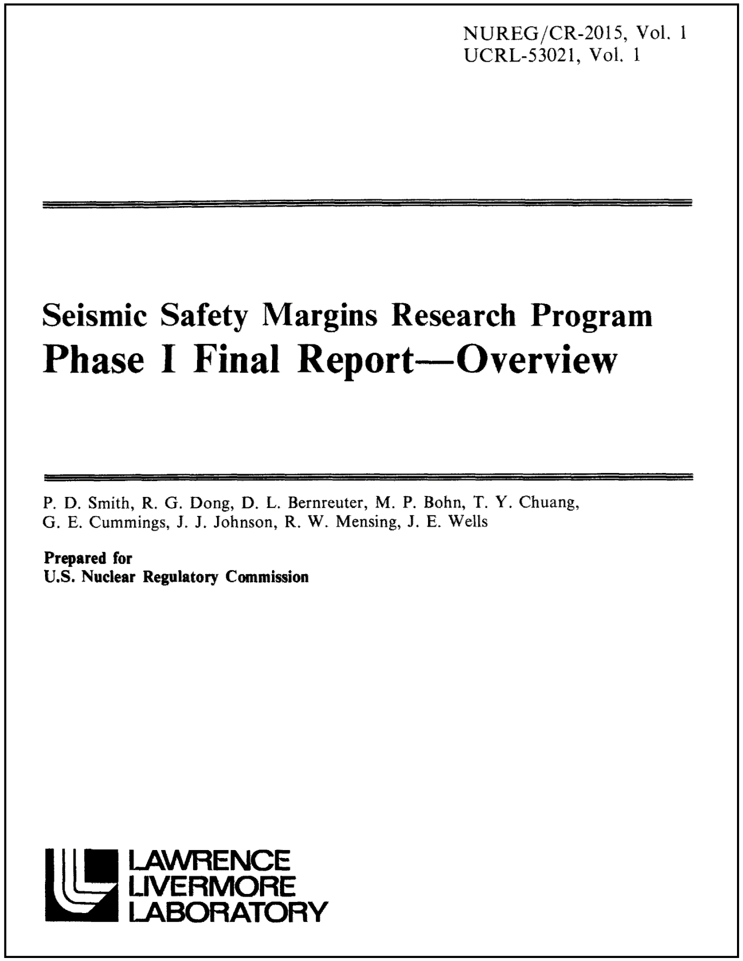


Figure 2. Schematic Representation of a Moving Radiating Source and Its Effect on Wave Amplitudes and Shapes (Modified from Benioff, 1955)

PBEE Rev 0, 1981: Nuclear

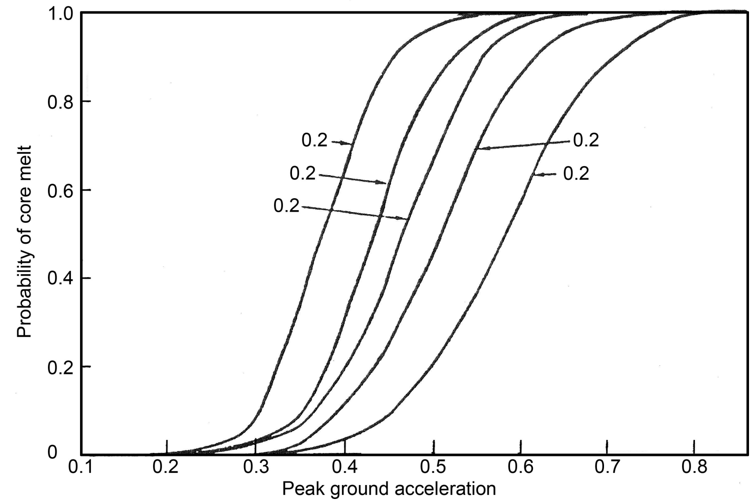

NUREG/CR-2015, Vol. 1
UCRL-53021, Vol. 1

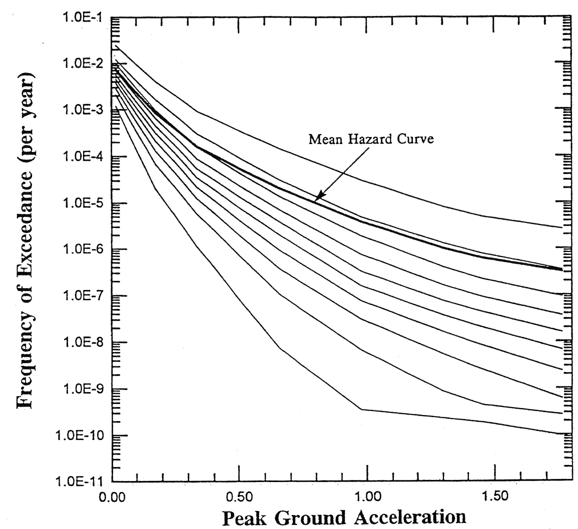
Seismic Safety Margins Research Program
Phase I Final Report—Overview

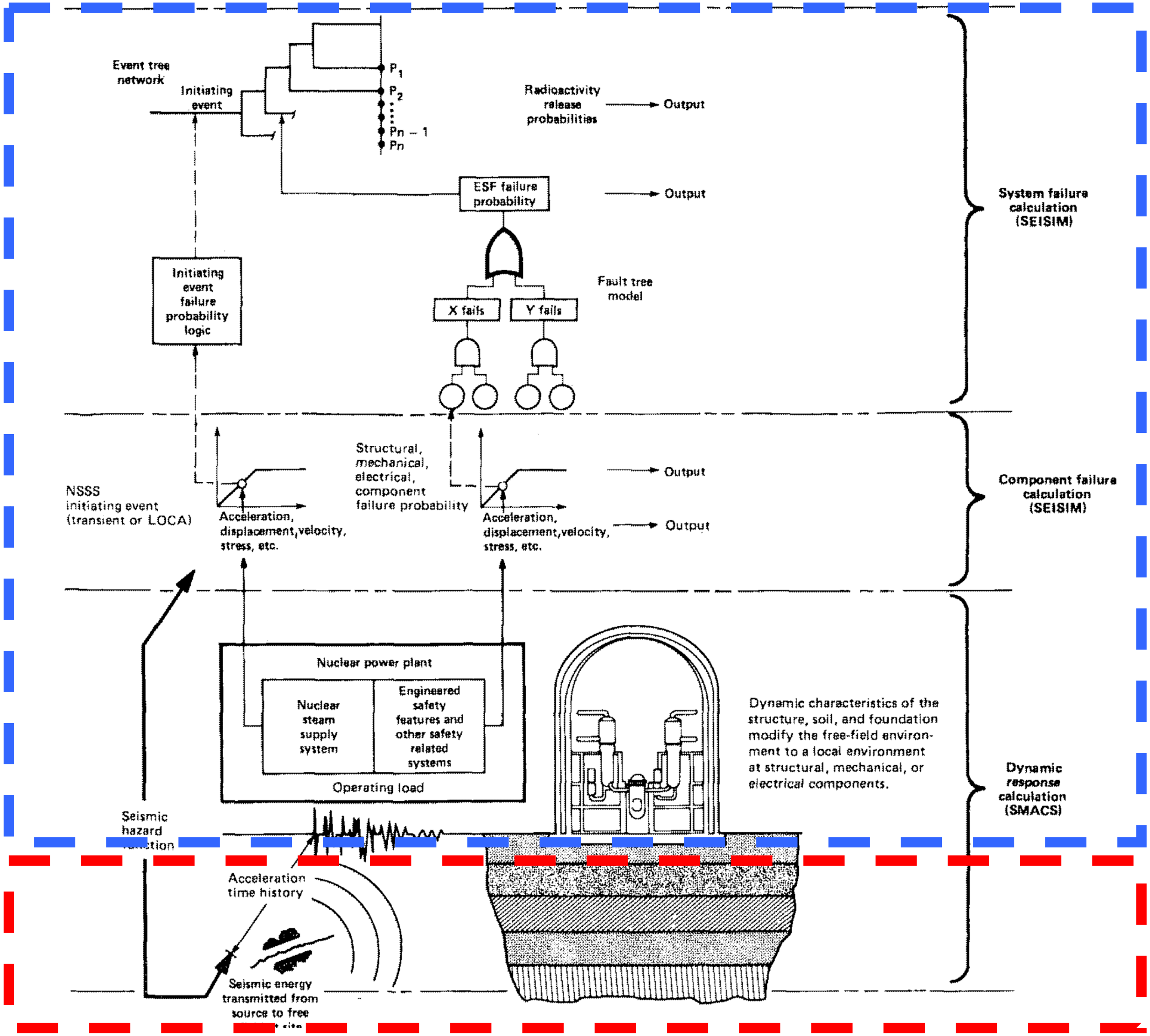
P. D. Smith, R. G. Dong, D. L. Bernreuter, M. P. Bohn, T. Y. Chuang, G. E. Cummings, J. J. Johnson, R. W. Mensing, J. E. Wells

Prepared for
U.S. Nuclear Regulatory Commission

LAWRENCE LIVERMORE LABORATORY







Enhancing risk-informed and performance-based seismic safety for advanced NLWRs, Washington, DC, 09/2020

PBEE Rev 1, 1992-1997

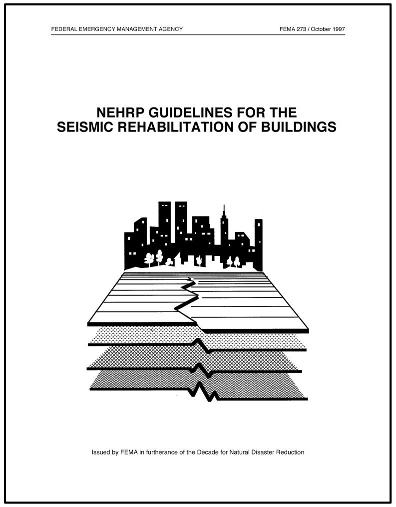


Table 2-2 *Rehabilitation Objectives*

		Building Performance Levels			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (~10%/50 year)	i	j	k	l
	BSE-2 (~2%/50 year)	m	n	o	p

Building Performance Levels and Ranges

Performance Level: the intended post-earthquake condition of a building; a well-defined point on a scale measuring how much loss is caused by earthquake damage. In addition to casualties, loss may be in terms of property and operational capability.

Performance Range: a range or band of performance, rather than a discrete level.

Designations of Performance Levels and Ranges: Performance is separated into descriptions of damage of structural and nonstructural systems; structural designations are S-1 through S-5 and nonstructural designations are N-A through N-D.

Building Performance Level: The combination of a Structural Performance Level and a Nonstructural Performance Level to form a complete description of an overall damage level.

Rehabilitation Objective: The combination of a Performance Level or Range with Seismic Demand Criteria.

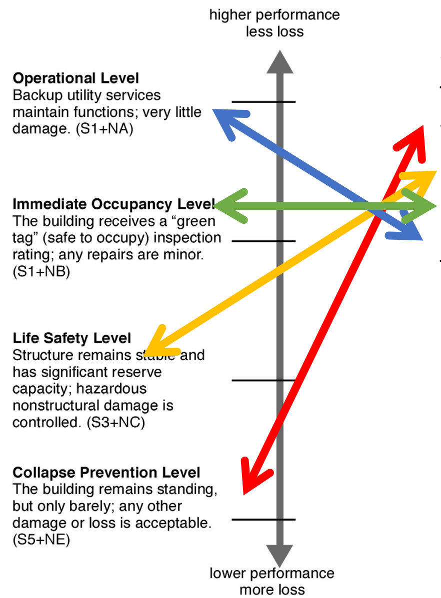


Table 1-2. Deformation and Damage by Limit State.

Limit State	Expected Deformation	Expected Damage
A	Large permanent distortion, short of collapse	Significant damage
B	Moderate permanent distortion	Generally reparable
C	Limited permanent distortion	Minimal damage
D	Essentially elastic behavior	Negligible damage

Source: Adapted from ANS 2.26 (ANS 2017).

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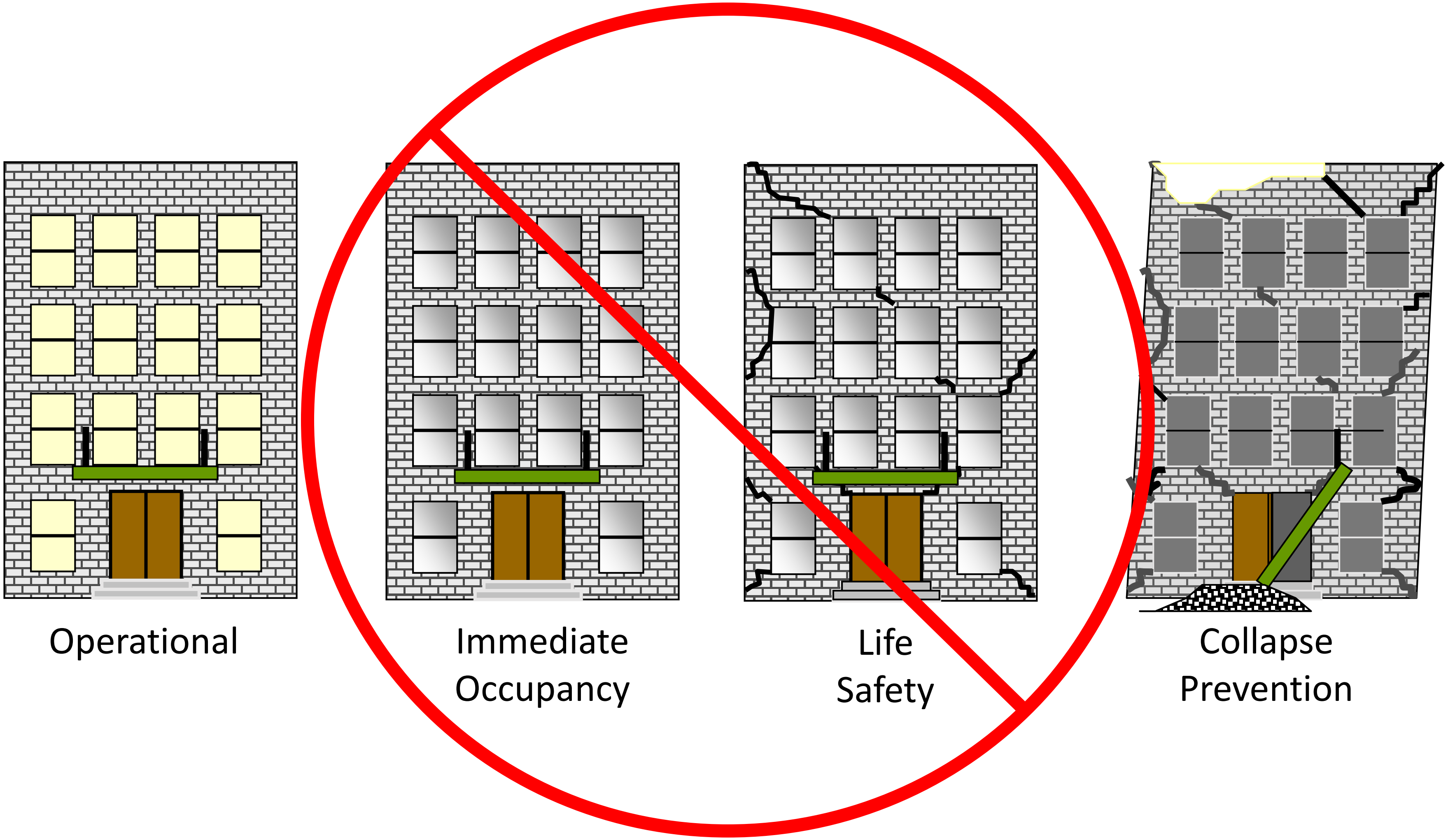

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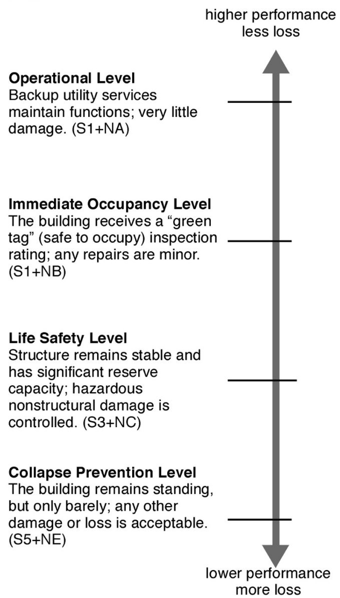

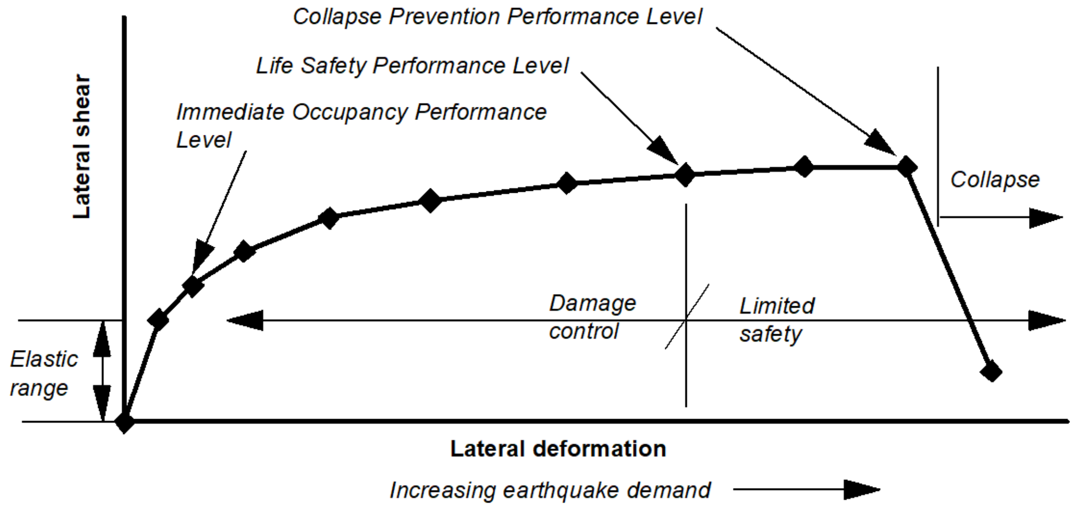

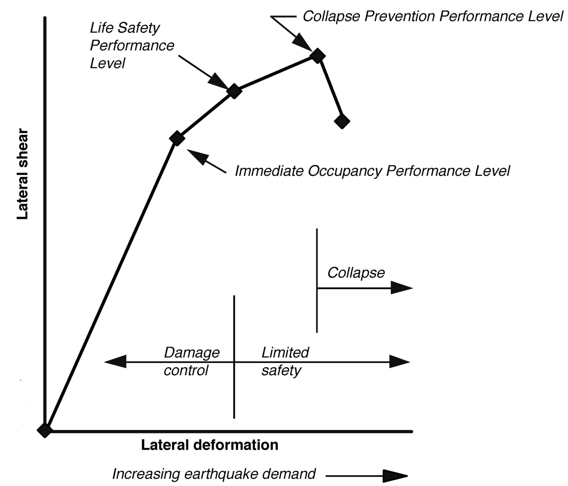

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- Damage = f (a, v, Δ)
- Methods of analysis
  - Linear static
  - Nonlinear static
  - Response-history
- Acceptance criteria
  - Component actions
    - Deformation, force
  - f (analysis method)
  - System performance
    - IO, LS, CP

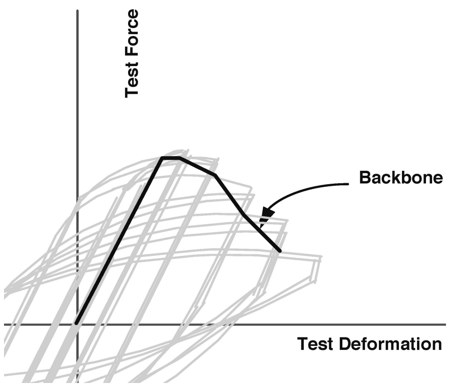
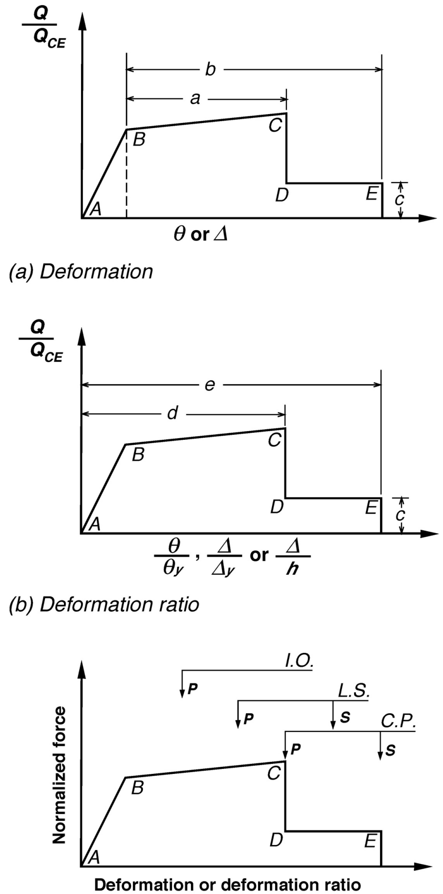


Table C3-1 ***Typical Deformation-Controlled and Force-Controlled Actions***

Component	Deformation-Controlled Action	Force-Controlled Action
Moment Frames • Beams • Columns • Joints	 Moment (M) M --	 Shear (V) Axial load (P), V V[1]
Shear Walls	M, V	P
Braced Frames • Braces • Beams • Columns • Shear Link	 P -- -- V	 -- P P P, M
Connections	--	P, V, M

1. Shear may be a deformation-controlled action in steel moment frame construction.



(a) Deformation

(b) Deformation ratio

(c) Component or element deformation limits

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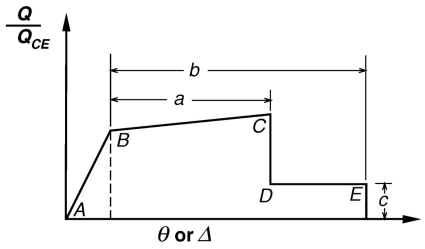
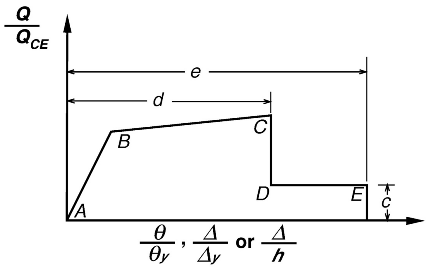
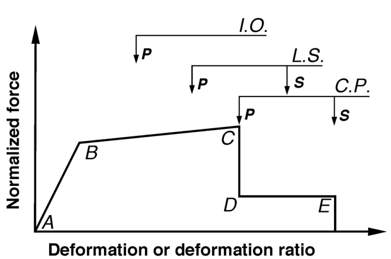


(a) Deformation



(b) Deformation ratio



(c) Component or element deformation limits

Table 6-6 ***Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams***

Conditions			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Component Type				
						Primary			Secondary	
			Plastic Rotation Angle, radians		Residual Strength Ratio	Performance Level				
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.005	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ *d*/2			0.0	0.02	0.2	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > *d*/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01

PBEE Rev 2, 2009: Nuclear

NUREG/CR-2015, Vol. 1
UCRL-53021, Vol. 1

Seismic Safety Margins Research Program
Phase I Final Report—Overview

P. D. Smith, R. G. Dong, D. L. Bernreuter, M. P. Bohn, T. Y. Chuang, G. E. Cummings, J. J. Johnson, R. W. Mensing, J. E. Wells

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A probabilistic seismic risk assessment procedure for nuclear power plants: (I) Methodology

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ABSTRACT

A new procedure for probabilistic seismic risk assessment of nuclear power plants (NPPs) is proposed. This procedure modifies the current procedures using tools developed recently for performance-based earthquake engineering of buildings. The proposed procedure uses (a) response-based fragility curves to represent the capacity of structural and nonstructural components of NPPs, (b) nonlinear response-history analysis to characterize the demands on those components, and (c) Monte Carlo simulations to determine the damage state of the components. The use of response-rather than ground-motion-based fragility curves enables the curves to be independent of seismic hazard and closely related to component capacity. The use of Monte Carlo procedure enables the correlation in the responses of components to be directly included in the risk assessment. An example of the methodology is presented in a companion paper to demonstrate its use and provide the technical basis for aspects of the methodology.

1. Introduction

In 1991, the United States Nuclear Regulatory Commission (NRC) issued Supplement 4 to Generic Letter No. 88-20 (USNRC, 1991) requiring nuclear power plant (NPP) utilities to perform an Individual Plant Examination of External Events (IPEEE) and also issued NUREG-1407 (Chen et al., 1991) to help guide the IPEEE. For an Individual Plant Examination (IPE) of seismic events, NUREG-1407 identified Seismic Margin Assessment (SMA) and Seismic Probabilistic Risk Assessment (SPRA) as acceptable methodologies for the examination of earthquake risk.

SMA seeks to identify critical components and systems in a NPP and determine the High-Confidence-Low-Probability-of-Failure (HCLPF) capacity of each critical NPP component and plant damage state, all in terms of ground-motion intensity. The HCLPF capacity of a NPP or its component represents the value associated with a 95% confidence of a 5% probability of failure. SMA procedures can be found in Budnitz et al. (1985), Prassinos et al. (1986) and Reed et al. (1991). SMA cannot be used to either (a) compute the seismic vulnerability or risk (annual frequency of unacceptable performance) of a NPP, or (b) identify the ground-motion intensity level and plant component that make the greatest contribution to the risk. These tasks can only be addressed using SPRA, which involves the integration of plant fragility data and a seismic hazard curves over a wide range of ground-shaking intensity and requires a full consideration of uncertainty in seismic hazard, structural response and properties and capacities of NPP components. The results of a SPRA can be used to determine the seismic margin of a NPP. This focus of this paper is SPRA.

SPRA determines the annual frequency of unacceptable performance, such as core melt and release of radiation. NUREG/CR-2300 (USNRC, 1983) provides general guidance for performing a SPRA. The guideline identifies two methods for SPRA: (1) Zion and (2) the Seismic Safety Margin (SSM). The Zion method was developed for the Oyster Creek probabilistic risk assessment and was later improved and applied for estimate seismic risk assessment at the Zion Plant (Pickard et al., 1981). The SSM method was developed in an NRC-funded project at the Lawrence Livermore National Laboratory (Smith et al., 1981). Although the procedures for computation of risk differ, both are based on the total probability theorem, which was also used by Cornell to develop probabilistic seismic hazard analysis (Cornell, 1968).

Recently developed procedures for the performance-based earthquake engineering (PBEE) of buildings (e.g., Moehle and Deierlein, 2004; Kiureghian, 2005; Yang et al., 2009) also utilize a probabilistic framework, which is similar in many regards to that developed by Smith et al. The ATC-58 project team developed procedures for seismic performance assessment of buildings using this framework (ATC, 2011). The ATC-58 methodology determines repair cost, downtime and casualties in a building subjected to seismic hazard characterized using a user-specified *intensity* of earthquake shaking, a user-specified *scenario* of earthquake

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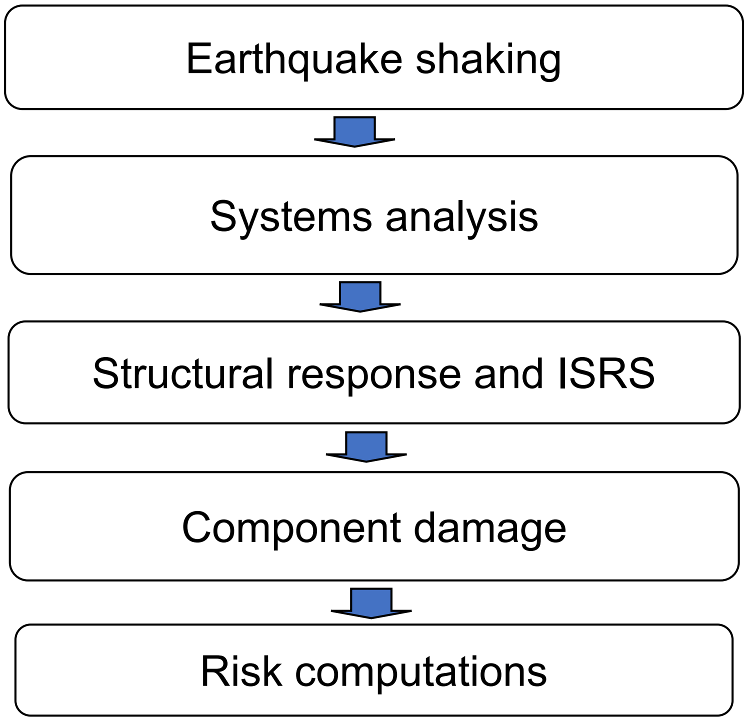

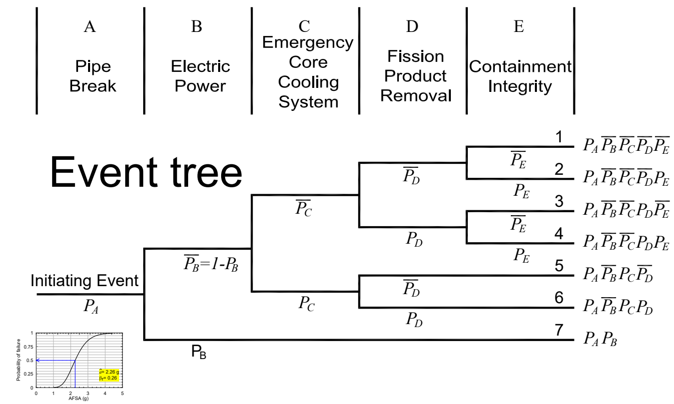

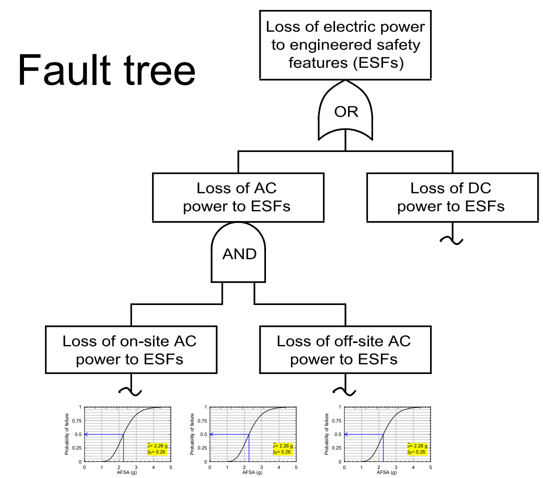

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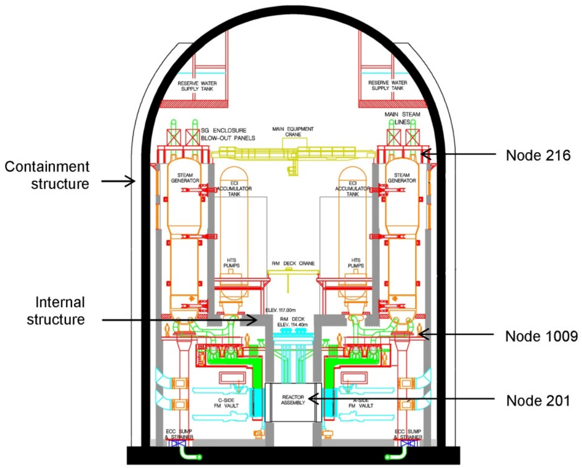

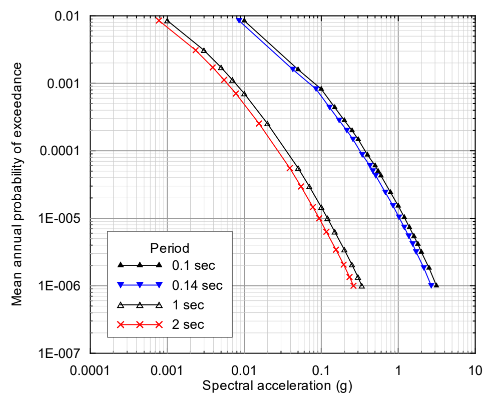

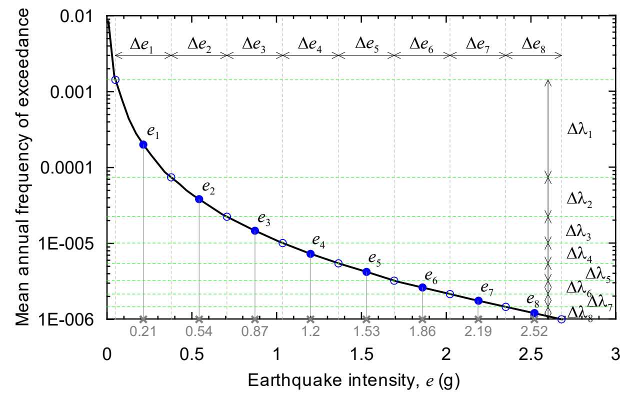

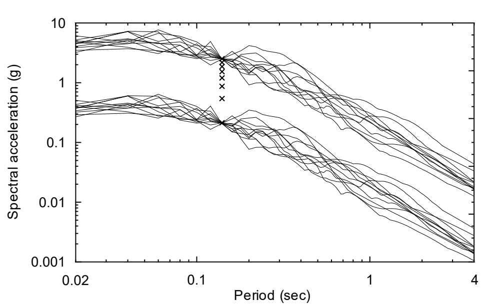

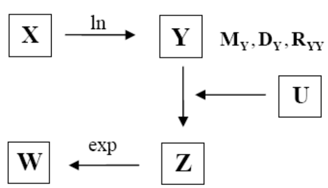

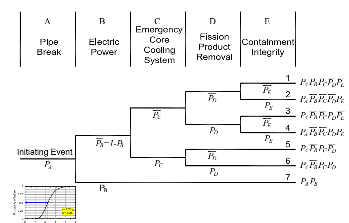

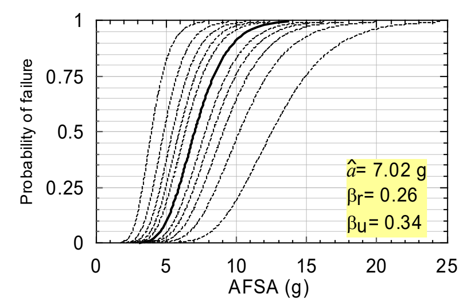

Yang, (2006, 2009)

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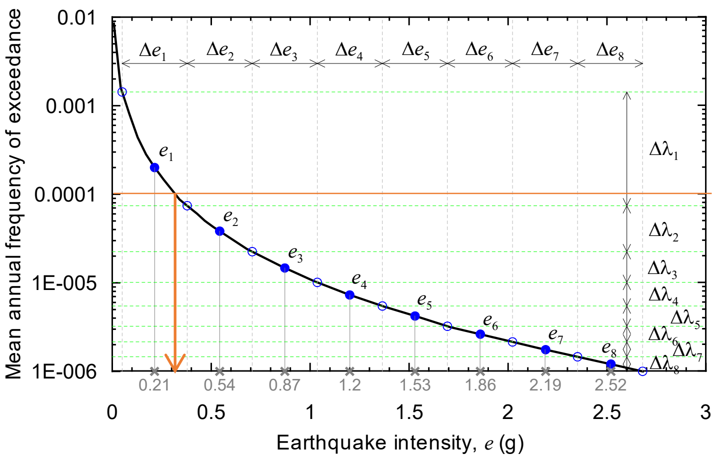
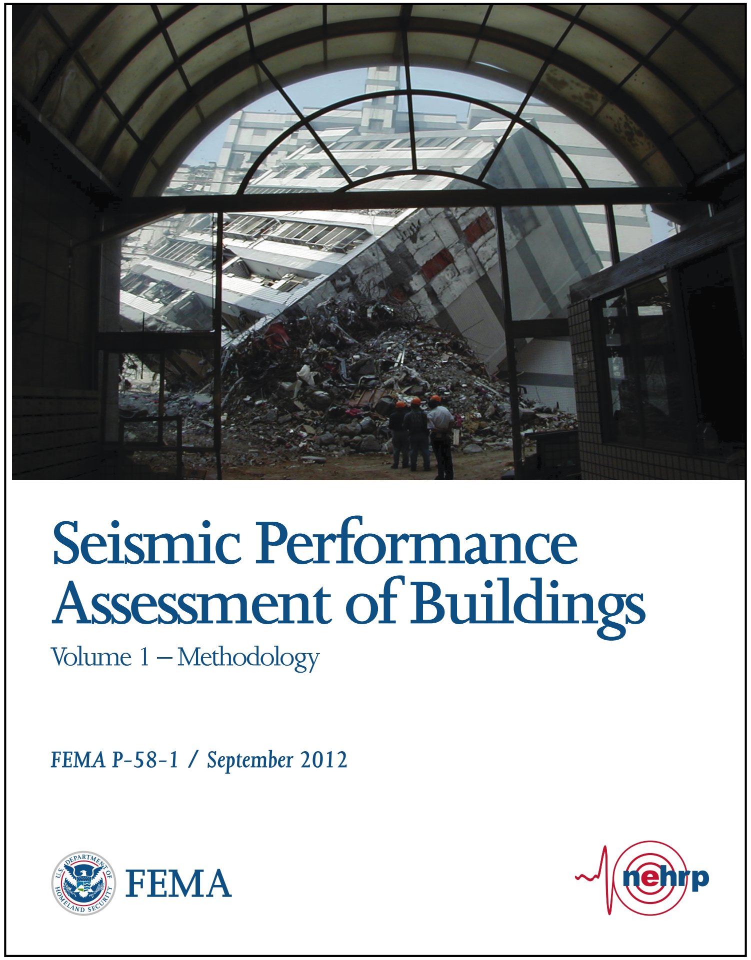


Ground-motion bin	$\Delta\lambda_i$	Probability of unacceptable performance, P_i	$\Delta\lambda_i \times P_i$	$\frac{\Delta\lambda_i \times P_i}{\sum_{i=1}^{8}\Delta\lambda_i \times P_i}$
TI1	1.35E-03	0	0	0.00
TI2	5.18E-05	0.16	8.29E-06	0.32
TI3	1.24E-05	0.68	8.43E-06	0.33
TI4	4.63E-06	0.94	4.35E-06	0.17
TI5	2.23E-06	0.99	2.21E-06	0.09
TI6	1.08E-06	1	1.08E-06	0.04
TI7	6.90E-07	1	6.90E-07	0.03
TI8	4.59E-07	1	4.59E-07	0.02
Annual freq of unacceptable performance			2.55E-05	—

PBEE Rev 3, 2000-2012

- Rev 1 shortcomings
  - Audience
  - Focus
    - Components
    - System level
  - Performance metrics
    - D/C, deformations
    - Deaths, dollars downtime, carbon
  - Asset allocation
- Risk focus
- Assessment types



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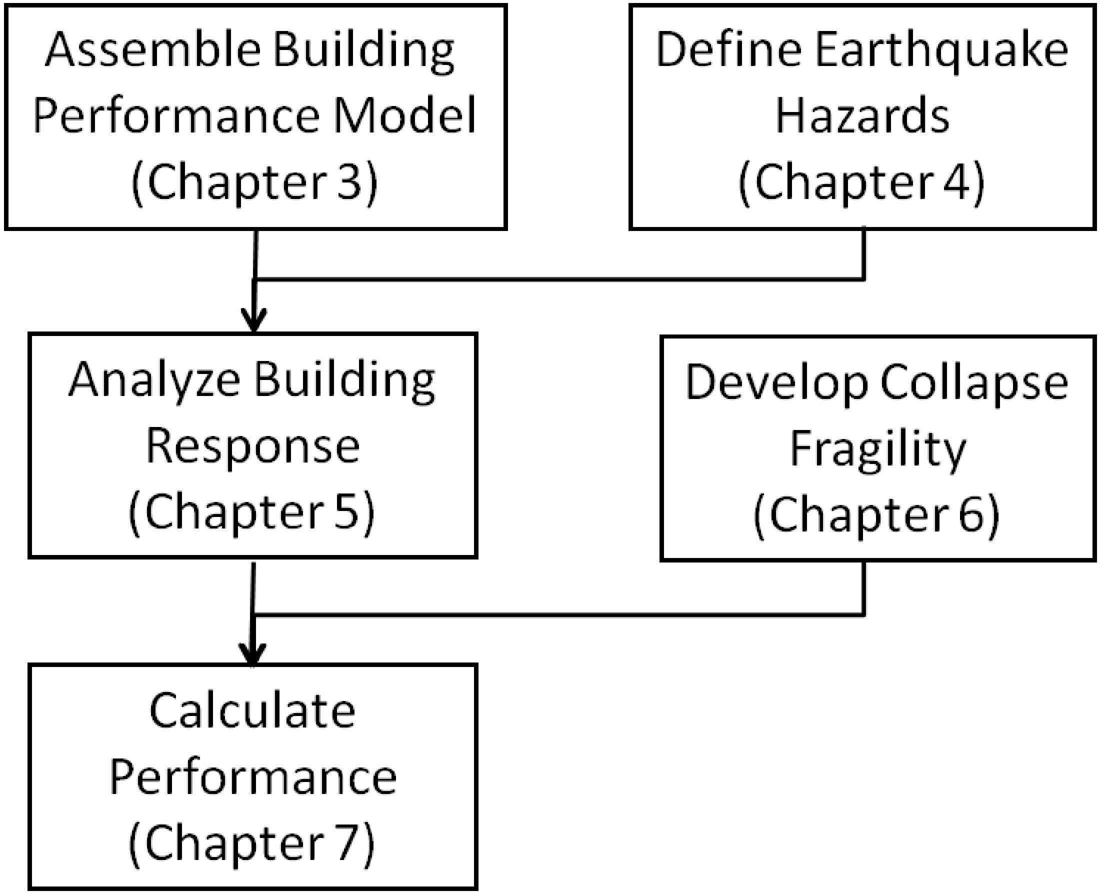
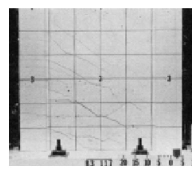
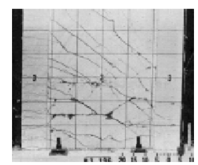
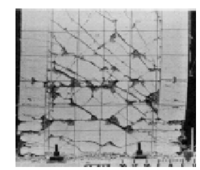


PBEE Rev 3, 2000-2012

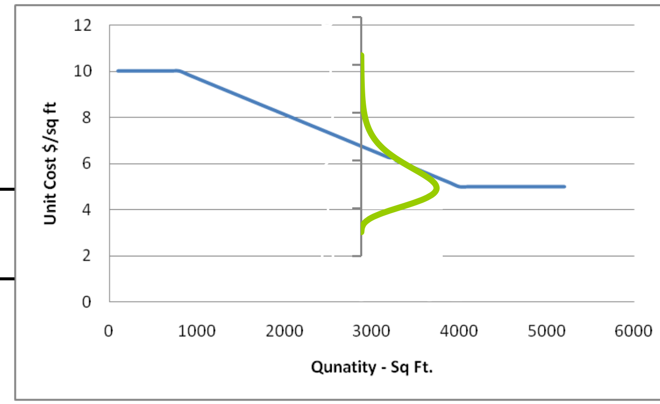
Fragility Specification **B1044.000 Reinforced Concrete Shearwalls**			
BASIC COMPOSITION	Reinforced concrete and finishes both sides		
Units for basic quantities	Square feet of wall area		
DAMAGES STATES, FRAGILIITES, AND CONSEQUENCE FUNCTIONS			
	DS1	DS2	DS3
DESCRIPTION	Flexural cracks < 3/16" Shear (diagonal) cracks < 1/16"	Flexural cracks > 1/4" Shear (diagonal) cracks > 1/8"	Max. crack widths >3/8" Significant spalling/ loose cover
ILLUSTRATION (example photo or drawing)			
MEDIAN DEMAND	1.5%	3.0%	5.0%
BETA	0.2	0.3	0.4
CORRELATION (%)	70%		
DAMAGE FUNCTIONS	Patch cracks each side with caulk Paint each side	Remove loose concrete Patch spalls with NS grout Patch cracks each side with caulk Paint each side	Shore Demo existing wall Replace Patch and paint
CONSEQUENCE FUNCTION			
Max. consequence up to lower quantity	$4.00 per sq ft up to 800 sq ft	$10.00 per sq ft up to 800 sq ft	$50.00 per sq ft up to 200 sq ft
Min consequence over upper quantity	$2.00 per sq ft over 4000 sq ft	$5.00 per sq ft over to 4000 sq ft	$30.00 per sq ft over 2000 sq ft
Beta (consequence)	0.2	0.3	0.3
TIMEFRAME TO ADDRESS CONSEQUENCES	days	weeks	months

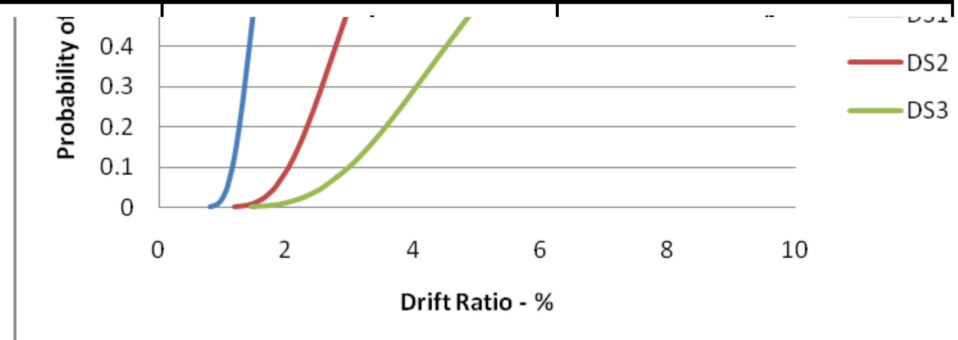
PBEE Rev 3, 2000-2012

Fragility Specification

B1044.000 Reinforced Concrete Shearwalls

BASIC COMPOSITION	Reinforced concrete and finishes both sides		
Units for basic quantities	Square feet of wall area		
DAMAGES STATES, FRAGILIITES, AND CONSEQUENCE FUNCTIONS			
	DS1	DS2	DS3
DESCRIPTION	Flexural cracks < 3/16" Shear (diagonal) cracks < 1/16"	Flexural cracks > 1/4" Shear (diagonal) cracks > 1/8"	Max. crack widths >3/8" Significant spalling/ loose cover
MEDIAN DEMAND	1.5%		
BETA	0.2		
CORRELATION (%)	70%		
DAMAGE FUNCTIONS	Patch cracks each side with caulk Paint each side	Remove loose concrete Patch spalls with NS grout Patch cracks each side with caulk Paint each side	Shore Demo existing wall Replace Patch and paint
CONSEQUENCE FUNCTION			
Max. consequence up to lower quantity	$4.00 per sq ft up to 800 sq ft	$10.00 per sq ft up to 800 sq ft	$50.00 per sq ft up to 200 sq ft
Min consequence over upper quantity	$2.00 per sq ft over 4000 sq ft	$5.00 per sq ft over to 4000 sq ft	$30.00 per sq ft over 2000 sq ft
Beta (consequence)	0.2	0.3	0.3





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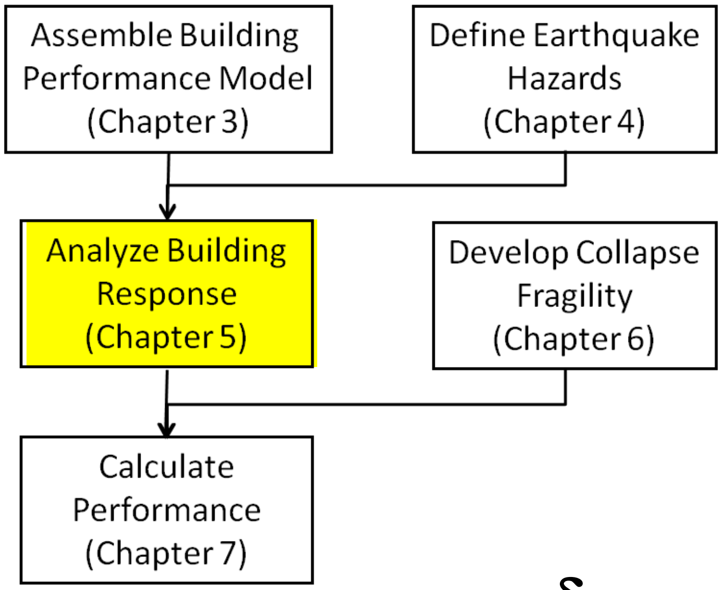

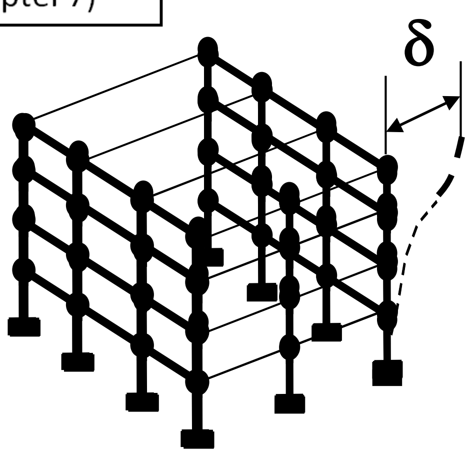

- Model structure
- Analysis for each stripe
  - Nonlinear response history
  - Simplified linear
- Predict median:
  - Story drifts
  - Floor accelerations
  - Floor velocities
  - Residual drifts
- Dispersions

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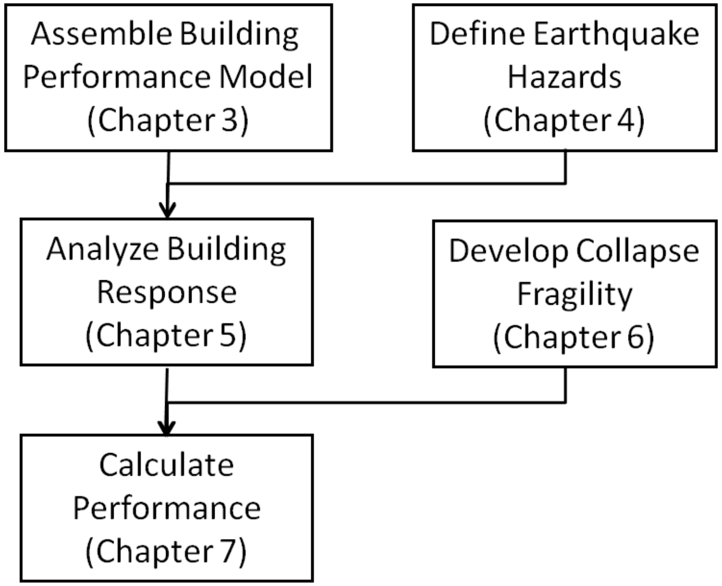
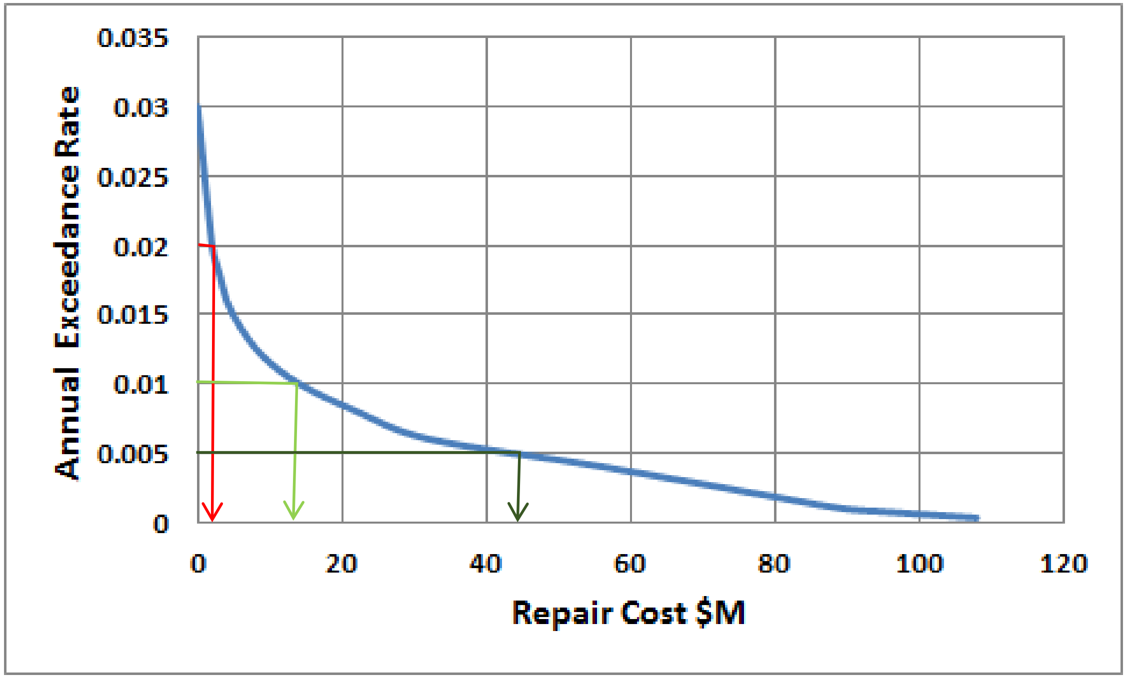


- Monte Carlo process
- 100s to 1000s of spins
  - Per intensity
  - 11 sets of base analyses
- Each spin a realization
- Unique
  - Demands
  - Damage
  - Consequences
- Generate loss curves

PBEE Rev 3, 2000-2012

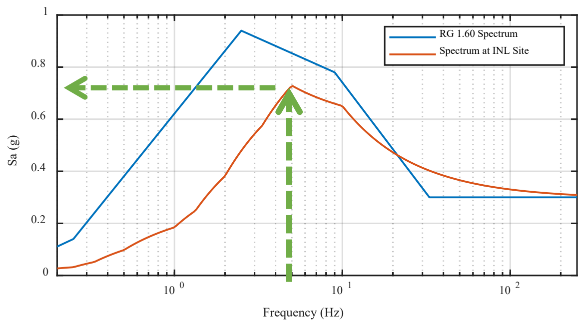
- 50-year loss $2,000K
- 100-year loss $14,000K
- 200-year loss $44,000K
- Ave annual loss $540K

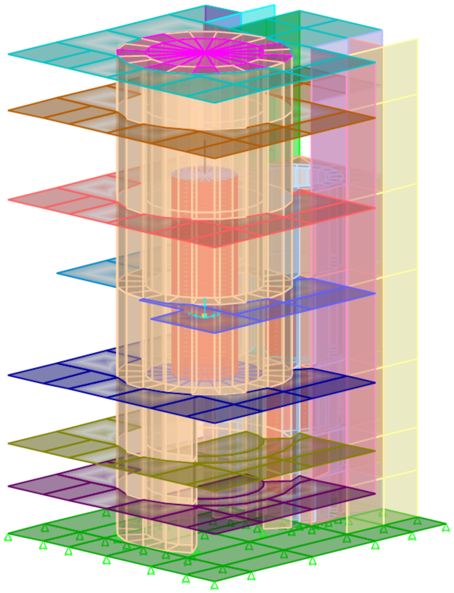


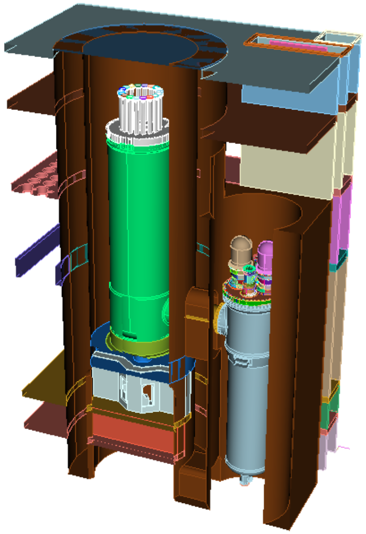
RIPB design—disciplinary silos

- Coupled soil-structure-equipment response
  - Earthquake shaking spectrum
  - Building amplification, equipment amplification
  - Cost
  - Infuse PRA into design

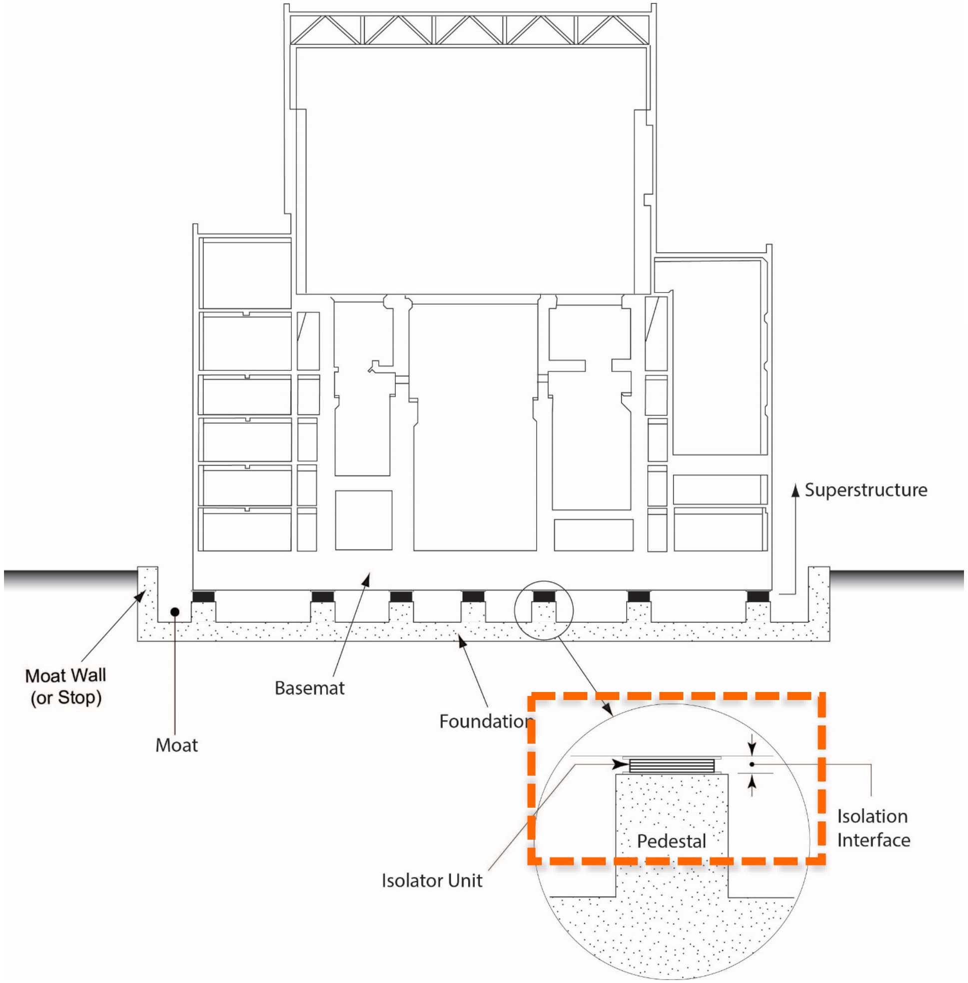
X-energy

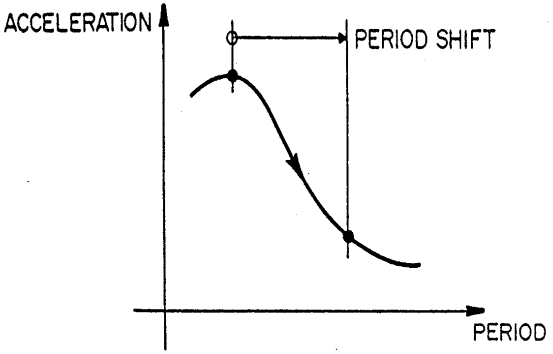


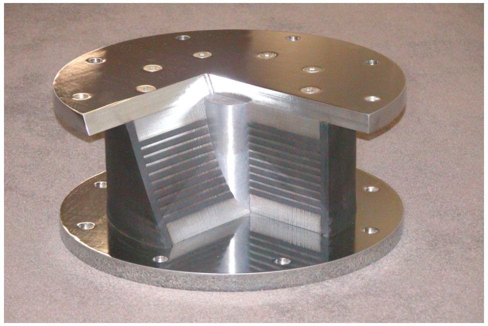


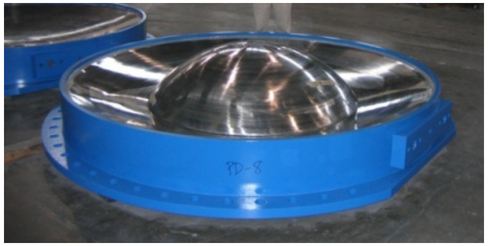


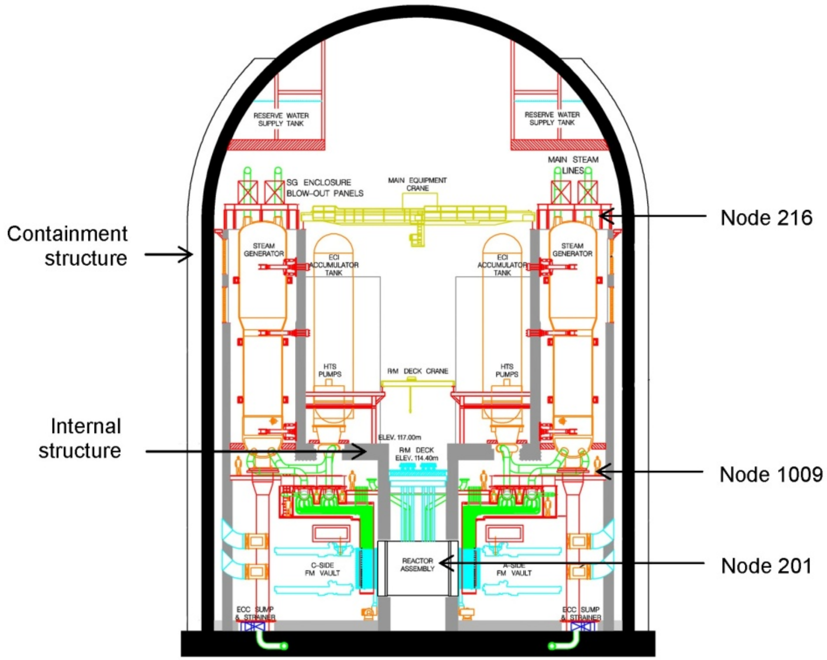
RIPB$_d$—isolation of LLWRs

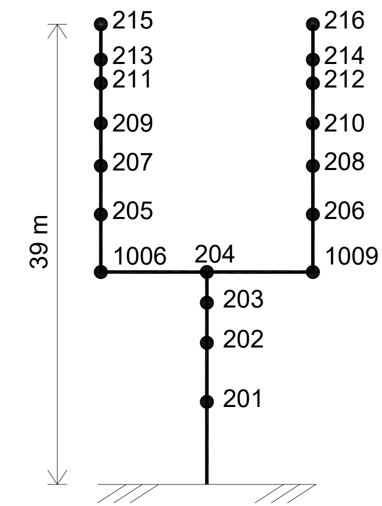









Containment structure
Internal structure
Node 216
Node 1009
Node 201
RESERVE WATER SUPPLY TANK
MAIN STEAM LINES
SG ENCLOSURE BLOW-OUT PANELS
MAIN EQUIPMENT CRANE
STEAM GENERATOR
ECI ACCUMULATOR TANK
HTS PUMPS
F/M DECK CRANE
REACTOR ASSEMBLY
C-SIDE F/M VAULT
A-SIDE F/M VAULT
ECC SUMP & STRAINER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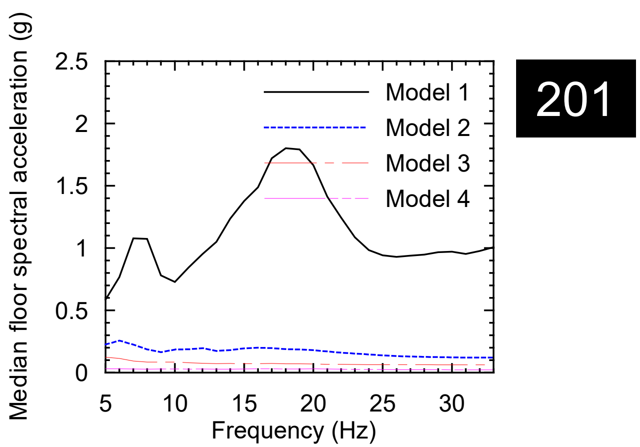
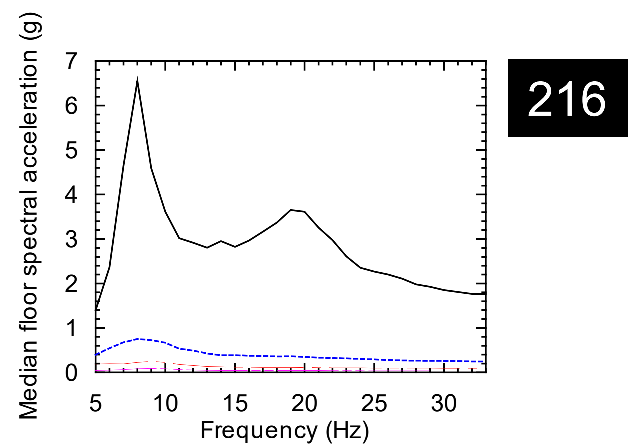
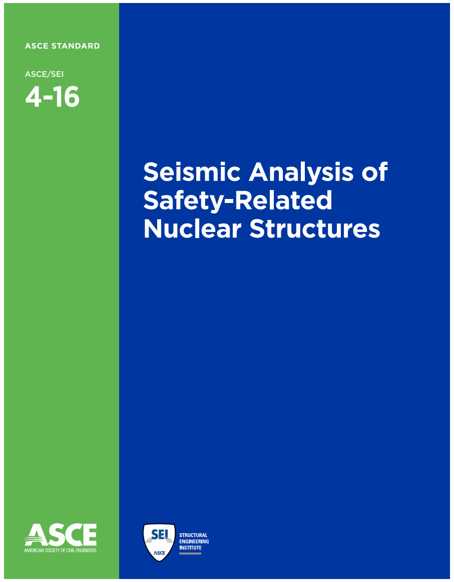

Median floor spectral acceleration (g)
Frequency (Hz)
Model 1
Model 2
Model 3
Model 4
201


Median floor spectral acceleration (g)
Frequency (Hz)
216

RIPB$_d$—isolation of LLWRs

- Regulatory guidance available
  - American Society of Civil Engineers
    - Chapter 12 of ASCE 4-16
    - Chapter 9 of ASCE 43-19
- NUREG/CRs
  - Technical considerations (7253)
  - Isolation of NPPs with sliding bearings (7254)
  - Isolation of NPPs with sliding bearings (7255)
- MCEER reports: 08-0019, 09-0008, 15-0006, 15-0008
- Numerical models for LDR, LR and FP isolators
- Seismic PRA procedures



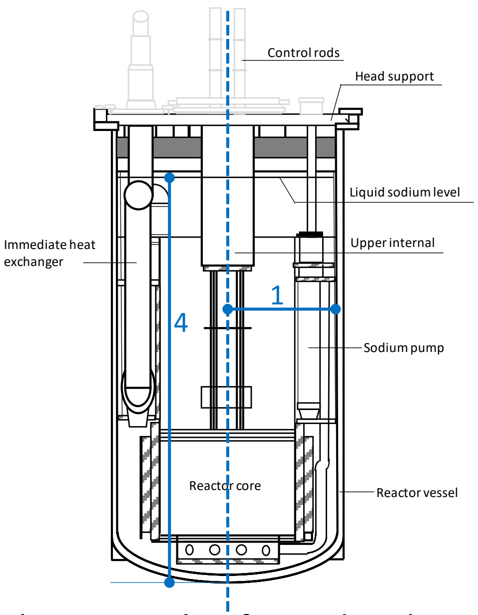
RIPB$_d$—isolation of LLWRs

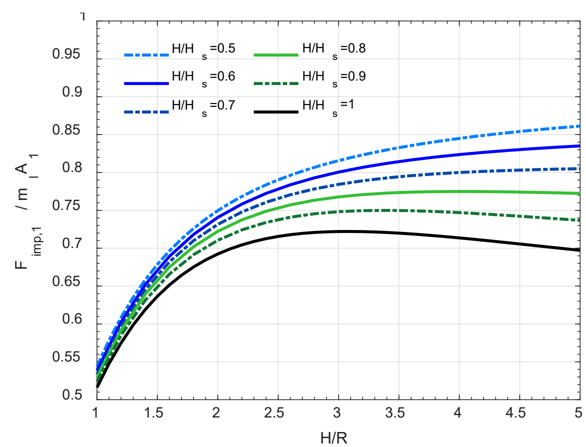
	Isolation system				
Ground motion levels	**Isolator unit and system design and performance criteria**	**Approach to demonstrating acceptable performance of an isolator unit**	**Superstructure design and performance**	**Umbilical line design and performance**	**Moat or stop design and performance**
GMRS+[2] Envelope of RG 1.208 GMRS and the minimum foundation input motion[3]	No long-term change in mechanical properties. Extremely high confidence of the isolation system surviving without damage when subjected to the mean displacement of the isolator system under the GMRS+ loading.	Perform production testing on each isolator for the mean system displacement under the GMRS+ loading and corresponding axial force.	Superstructure design and performance to conform to NUREG-0800 for GMRS+ loading.	Umbilical line design and performance to conform to NUREG-0800 for GMRS+ loading.	Moat gap sized such that there is less than 1% probability of the superstructure impacting the moat or stop for GMRS+ loading.
BDBE GMRS[4] Envelope of the UHRS at a MAFE of 1´10^{-5} and 167% of the GMRS+ per ISG 20	90% confidence of each isolator and the isolation system surviving without loss of gravity-load capacity at the mean displacement under BDBE GMRS loading.	Perform prototype testing must be performed on a sufficient number of isolators at the CS[5] displacement and the corresponding axial force to demonstrate acceptable performance with 90% confidence. Limited isolator unit damage is acceptable but load-carrying capacity must be maintained.	Less than a 10% probability of the superstructure contacting the moat or stop under BDBE GMRS loading.	Greater than 90% confidence that each type of safety-related umbilical line, together with its connections, shall remain functional for the CS displacement. Performance may be demonstrated by testing, analysis or a combination of both.[6]	Moat gap sized such that there is less than a 10% probability of the superstructure impacting the moat or stop for BDB GMRS loading. Stop designed to survive impact forces associated with isolation system displacement to 95th percentile BDBE isolation system displacement.[7] Limited damage to the moat or stop is acceptable but the moat/stop should perform its function.

1. Analysis and design of safety-related components and systems shall conform to NUREG-0800.
2. 10CFR50 Appendix S requires the use of an appropriate free-field spectrum (often the RG 1.60 spectral shape) with a peak ground acceleration of no less than 0.10g at the foundation level.
3. The analysis can be performed once using a composite spectrum or twice using the GMRS and the minimum spectrum separately.
4. The analysis can be performed once using a composite spectrum or twice using the 1×10^{-5} MAFE UHRS and the 167%GMRS+ separately.
5. CS=Clearance to the Stop
6. Seismic Category 2 SSCs whose failure could impact the functionality of umbilical lines shall also remain functional for the CS displacement.
7. Impact velocity calculated at the displacement equal to the CS assuming cyclic response of the isolation system for motions associated with the 95th percentile (or greater) BDB GMRS displacement.

RIPB$_d$—isolation of equipment

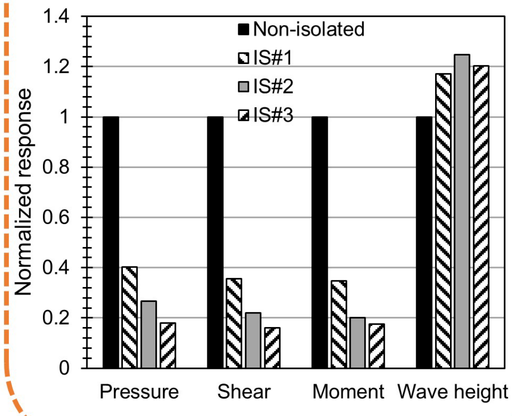
- ARPA-E focus is advanced reactors
- Pathway to seismically isolate equipment
  - Analysis, design, qualification, PRA, ASCE 4-21
  - Cost study, EPRI report 03002018345, August 2020



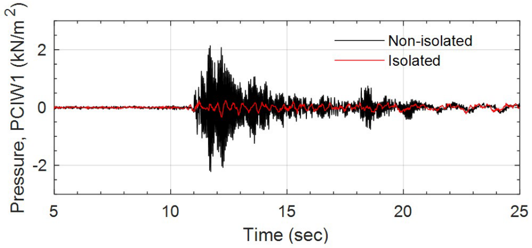


RIPB$_d$—isolation of equipment

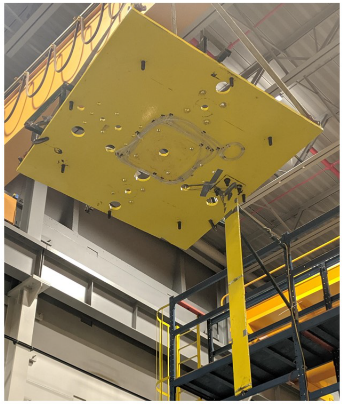
Test 1

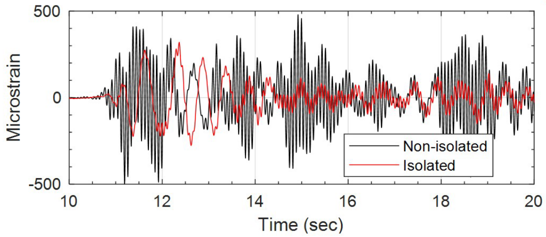


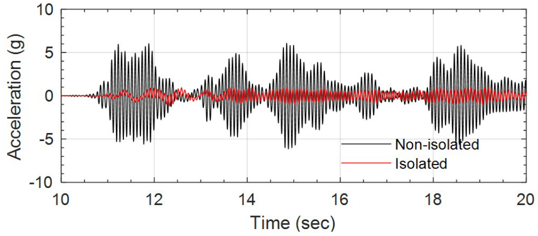
Test 2



Test 3

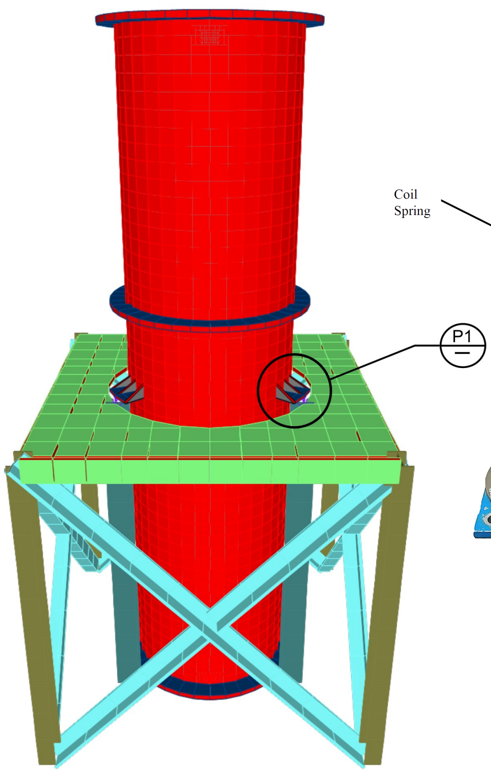


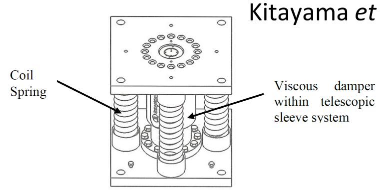




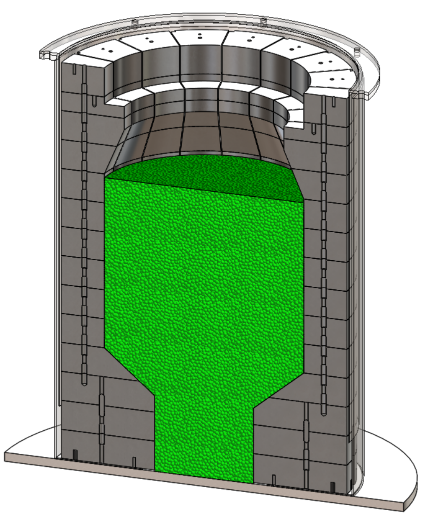
RIPB$_d$—isolation of equipment

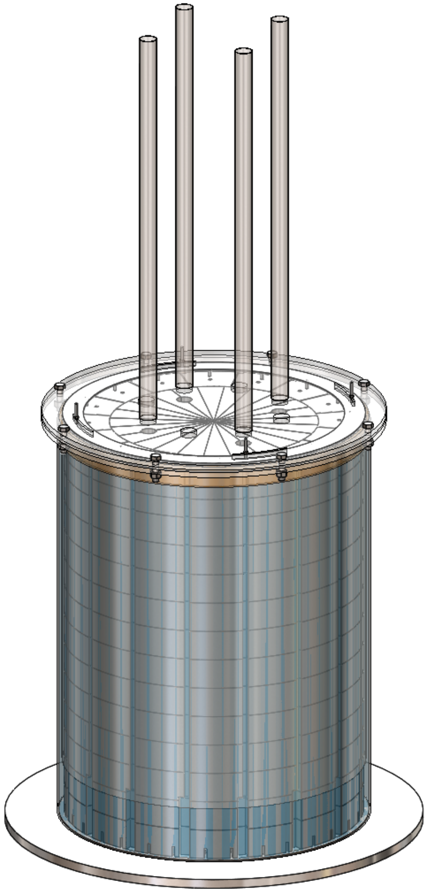
- Steam generator
- Molten salt pebble bed reactor





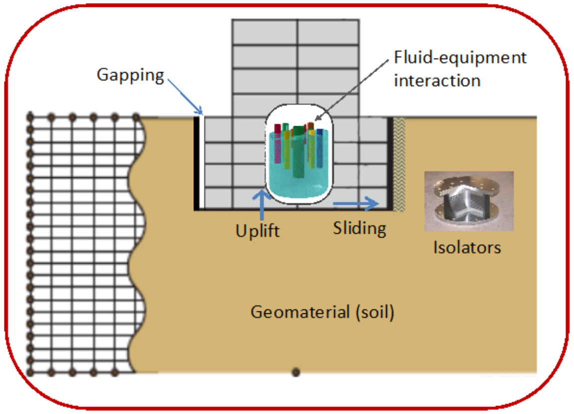
Kitayama *et al.*, 2016





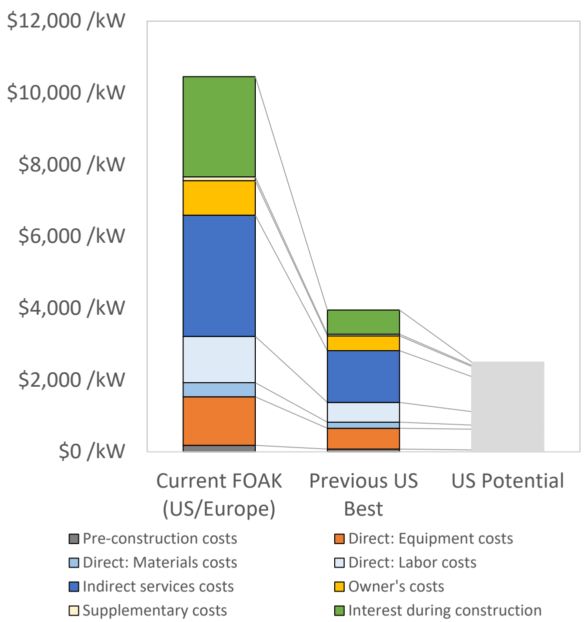
RIPB$_d$—nonlinear analysis

- ASCE 4 and 43
  - Soil, isolators, dampers, soil-foundation interface, structure, equipment, anchorages
  - Advanced reactors different from LLWRs
- DOE-funded Pathway 3 FOA to SC Solutions
  - Integrated nonlinear analysis of systems
  - Software CGD
    - Guidance and test cases
    - Agnostic to software platform
    - Agnostic to reactor-type
  - Build regulatory confidence



RIPB$_d$—codes and standards

- Integrated codes and standards
  - ASCE 4 and 43
    - PBEE circa 1995, FEMA 273
  - AISC N690
  - ACI 349
  - ASME
  - IEEE
  - ANS
    - SPRA suitable for LLWRS
    - AR and microreactors: SDC 3 and 4, LS C
  - Perhaps the biggest challenge to RIPB$_d$



LucidCatalyst, 2019

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- Robert Kennedy, RPK Structural Mechanics
- Manish Kumar ^2, IIT Bombay and IIT Gandhinagar
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- Mete Sozen, UIUC
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- MCEER, incl. Dr. Michael Constantinou, Ching-Ching Yu, Faizan Mir, Kaivalya Lal, Sharath Parsi,
- Pacific Earthquake Engineering Research (PEER) Center, incl. Dr. Steve Mahin
- US Department of Energy, incl. Dr. Justin Coleman
- US Nuclear Regulatory Commission, incl. Dr. Jose Pires

awhittak@buffalo.edu

GE-Hitachi Nuclear Energy Risk Informed Performance Based (RIPB) Seismic Design Experience and Perspectives

NRC/RES Virtual Public Workshop

September 2-3, 2020

Luben Todorovski, Ph.D. P.E.

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GEH RIPB Seismic Design Experience and Perspectives

Presentation Objective

- Describe the experience GE-Hitachi Nuclear Energy (GEH) has gained with Risk Informed Performance Based (RIPB) seismic design while working on ta recent DOE U.S. Department of Energy (DOE) test reactor project
- Describe the current approach for the seismic design of the new GEH's BWRX-300 Small Modular Reactor (SMR)
  - ASCE 43-05 Limit State concept is introduced to consider limited permanent deformations for II/I seismic interaction evaluations
- Present GEH's perspectives for using RIPB for:
  - the development of new advanced sodium-cooled reactor technologies
  - the development of the design-for-construction BWRX-300 SMR

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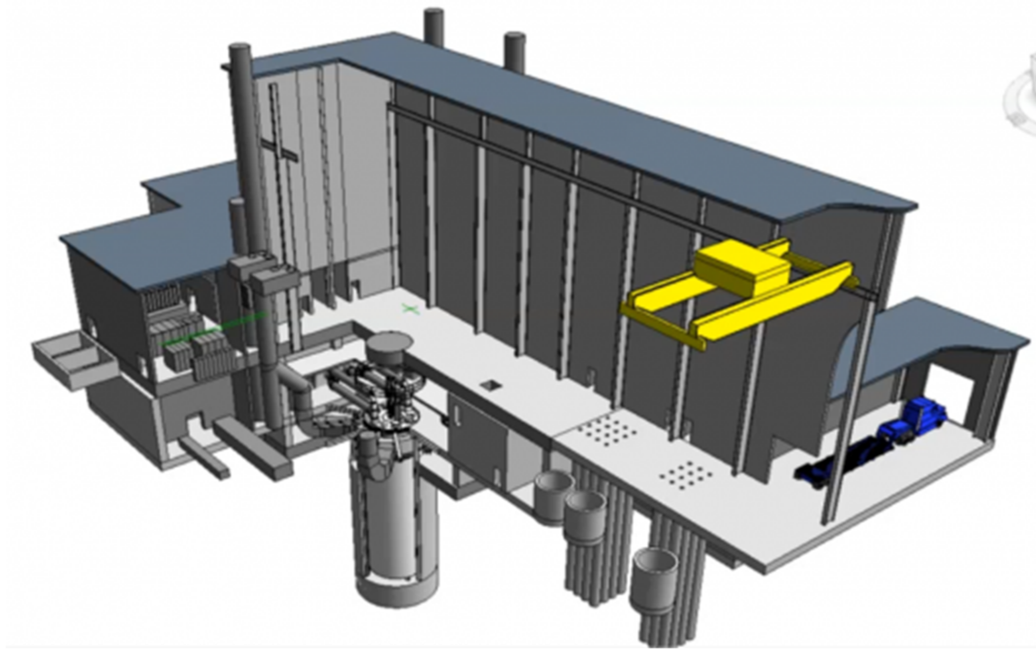
GEH RIPB Seismic Design Experience and Perspectives

VTR Project

- GEH is part of the team working on the development of a fast-neutron testing facility for the U.S. Department of Energy (DOE) VTR project
  - The likely design will be a 300 MWt sodium-cooled reactor based on GEH's PRISM reactor design
  - The primary functions of the VTR are to provide reliable testing of fuels and materials for advanced reactors
  - Plant heat rejected to the atmosphere primarily via sodium-air heat exchangers
  - Conceptual seismic design is being performed to advance the VTR design



GEH RIPB Seismic Design Experience and Perspectives

VTR Seismic Design Basis

- The RIPB approach is being implemented for the VTR design to:
  - meet the plant safety requirements of U.S. DOE Order 420.1C, Chapter IV
  - achieve an acceptable and balanced risk to the facility workers and public by applying seismic design requirements commensurate with the severity of consequences from SSC failure
- The graded approach of DOE-STD-1020-2016 is implemented for the seismic design and evaluation of VTR Structures, Systems and Components (SSCs)
- Performance category and an approximate annual probability of unacceptable performance (PF) is assigned to all:
  - VTR SSCs required to perform its safety function for protection of the public and co-located workers during and after a design basis seismic event
  - Other VTR SSCs used to protect these safety SSCs or to prevent or mitigate two-over-one common-cause failures and systems interaction effects during and after design basis seismic event

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GEH RIPB Seismic Design Experience and Perspectives

VTR Seismic Design Categories

- VTR safety SSCs are categorized in the five ASCE 43-05 Seismic Design Categories (SDC)
- The seismic design of VTR SDC-3, SDC-4, and SDC-5 SSCs is based on Design Basis Earthquakes (DBEs), defined in Table 1-2 of ASCE 43-05 by seismic hazard exceedance probabilities (H_D) that meet the required performance (P_F) goals:
  - SDC-3 DBE with $H_D = 4 \times 10^{-4}$ to meet target performance goal $P_F \approx 1 \times 10^{-4}$
  - SDC-4 DBE with $H_D = 4 \times 10^{-4}$ to meet target performance goal $P_F \approx 4 \times 10^{-5}$
  - SDC-5 DBE with $H_D = 1 \times 10^{-4}$ to meet target performance goal $P_F \approx 1 \times 10^{-5}$
- The seismic design of VTR SDC-1 and SDC-2 SSCs is based on the DBE developed per provisions of IBC-2015 and ASCE 7-10
  - The VTR SDC-1 DBE is developed considering ASCE 7-10 Risk Category II approximately corresponding to $P_F < 10^{-3}$
  - The SDC-2 DBE is developed considering ASCE 7-10 Risk Category IV approximately corresponding to $P_F < 4 \times 10^{-3}$

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VTR Limit States

Four Limit States (LS) are assigned to VTR seismically categorized SSCs based on critical threshold value(s) of stress, strain, or deformation at which the SSC fails its safety function or compromises the safety function of another SSC during and/or after a design earthquake

Limit State	Condition
LS-D	Representing no damage corresponding to an essentially elastic behavior without permanent deformations
LS-C	minimal damage accompanied with limited permanent deformations
LS-B	generally repairable damage accompanied with moderate permanent deformations
LS-A	significant damage short of collapse accompanied with large permanent deformations

LS are assigned to different types of SSC using the examples provided in Appendix B of ANSI/ANS-2.26-2004

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VTR Seismic Categorization

- The selection of SDC and LS is based on results of integrated safety analyses perfomed per guidelines of ANSI/ANS-2.26-2004, Section 6 considering the following concepts:
  - Defense-in-depth to design the facility with layers of defense against adverse consequences of the SSC failure
  - Redundancy of the considered SSC safety function that is performed by another SSCs or can be replaced by administrative or control measure
  - Common cause failure of multiple SSCs resulting from a certain licensing basis event, unless SSCs is robust or incorporates redundancy with low probability of failure during earthquake
  - Robustness achieved by providing assured margins in the resistance to seismically induced damage typically by using LS C or D levels for the seismic design
  - II/I system interaction of safety and non-safety SSCs which failure may impair the safety function of another SSCs
- SDC and LS are assigned to the VTR SSCs based on the level of unmitigated consequences of earthquake induced failure
  - Unmitigated consequences are estimated without taking credit for mitigating effects of any SSC or procedure
  - Unmitigated consequence tresholds specified by DOE-STD-1020-2016, Section 2.3.3 are used

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VTR Conceptual Design Seismic Design Categories

- In the absence of safety analysis results and site design inputs, a conservative categorization of VTR safety SSCs was used for purposes of conceptual seismic design evaluations

• Reactor Vessel (RV), reactor containment SSCs and their supporting structures	SDC-5
• Reactor shutdown SSCs	SDC-5
• RV Cooling System and primary coolant boundary SSCs	SDC-5
• SSCs for handling and storage of experiments or fuel outside of confinement	SDC-3
• Cover gas and sodium cleanup systems and their supporting structures	SDC-3

- All VTR structures and foundations that support and/or protect safety SSCs are categorized in the SDC that corresponds to the highest SDC of the SSCs they support or protect
- Conceptual design of SDC-5 and SDC-3 SSCs defaults to LS-D (essentially elastic behavior and no damage), with a few exceptions:
  - The seismic design of RV auxiliary cooling system stacks (RVACS) may consider LS-C because it's safety performance is not affected by the RVACS limited permanent deformations

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VTR II/I System Interaction Considerations

- Section 6.3.2.4 of ANSI/ANS-2.26-2004 specifies the following approaches for addressing II/I system interaction and categorization of SSCs which failure may affect the safety function of nearby safety SSC
  1) upgrade the non-safety or lower SDC or Limit State SSC to the necessary extent to preclude its adverse interaction with the target safety SSC
  2) place the non-safety or lower SSCs in the same or higher SDC and modify its LS to preclude interaction with the target SSCs
  3) configure the facility layout or SSCs design to preclude adverse II/I interaction with safety SSCs
  4) designing the target safety SSCs to withstand the imposed interaction load
- For the conceptual design, VTR SSCs which failure may affect the safety function of nearby safety SSC:
  - are designed based on their seismic categorization
  - separate II/I interaction evaluations are performed for these SSCs using the SDC DBE of the target safety SSC and considering LS-C limited permanent deformations and minimal damage

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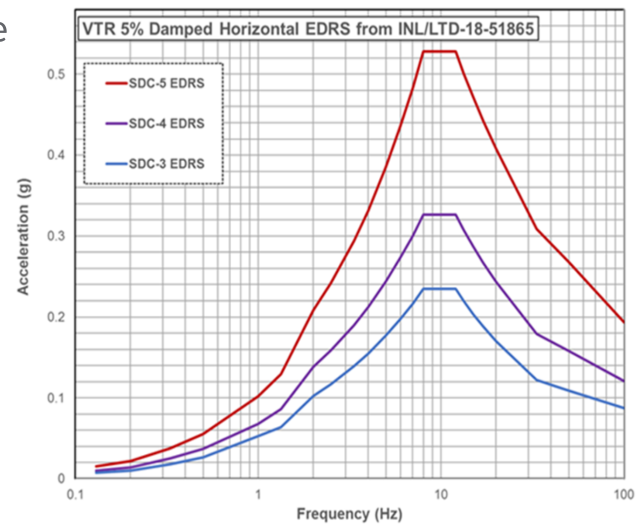
GEH RIPB Seismic Design Experience and Perspectives

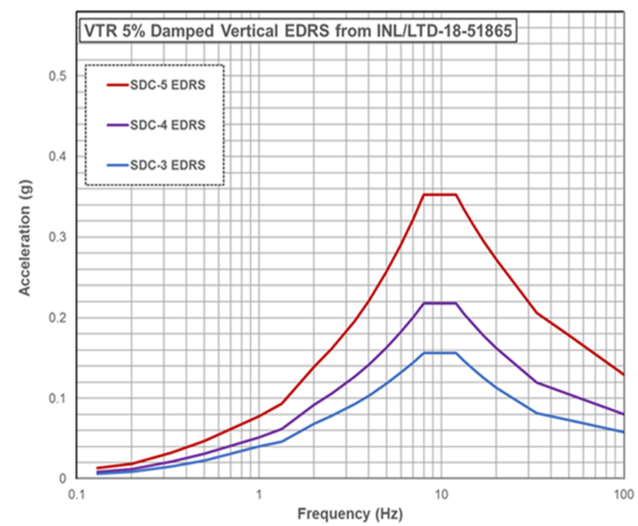
VTR Design Response Spectra

- 5% damped performance-based Design Response Spectra (DRS) are developed for seismic design of VTR SDC-3, SDC-4 and SDC-5 SSCs following the approach of ASCE 43-05, Section 2.1 to achieve relatively consistent annual probability of earthquake induced failure across the whole range of structural frequencies and locations

- In the absence of specific information regarding the seismological and subgrade site conditions Early Design Response Spectra (EDRS) are used for the conceptual design
- SDC-3, SDC-4 and SDC-5 DBE EDRS are developed from PSHA of nearby site using 84th% fractile level and an appropriate peak frequency broadening to address potential increases to the mean hazard
- SDC-3 DRS are 40% to 52% of SDC-5 DRS
- SDC-4 DRS are 58% to 67% of SDC-5 DRS

VTR 5% Damped SDC-3, SDC-4 and SDC-5 DBE Conceptual Design Early Design Response Spectra





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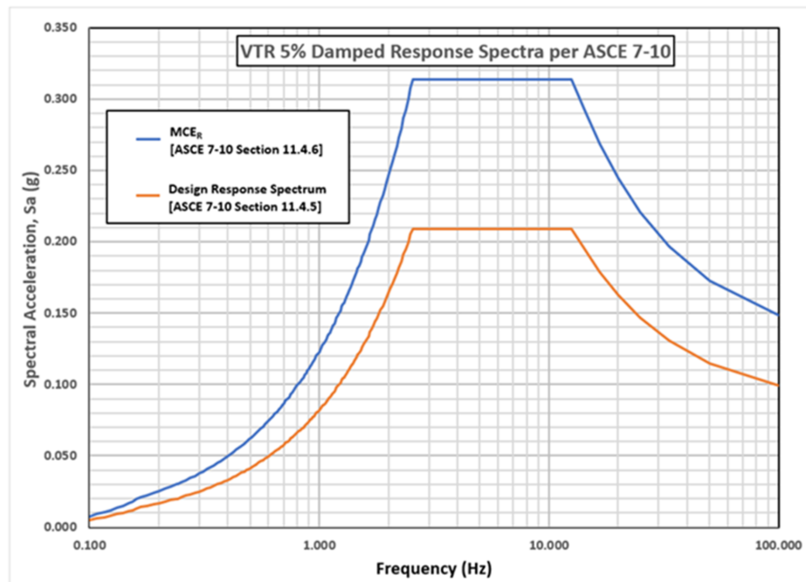
GEH RIPB Seismic Design Experience and Perspectives

VTR Design Response Spectra

- 5% damped DRS for seismic design of SDC-1 and SDC-2 SSCs is developed following the provisions of ASCE 7-10 (17), Section 11.4 that achieves anticipated reliability against total or partial structural collapse of:
  - 10% for SDC-1 (Risk Category II) structures; and
  - 3% for SDC-2 (Risk Category IV) structures

VTR 5% Damped DRS
Conceptual Design of SDC-1 and SDC-2 SSCs

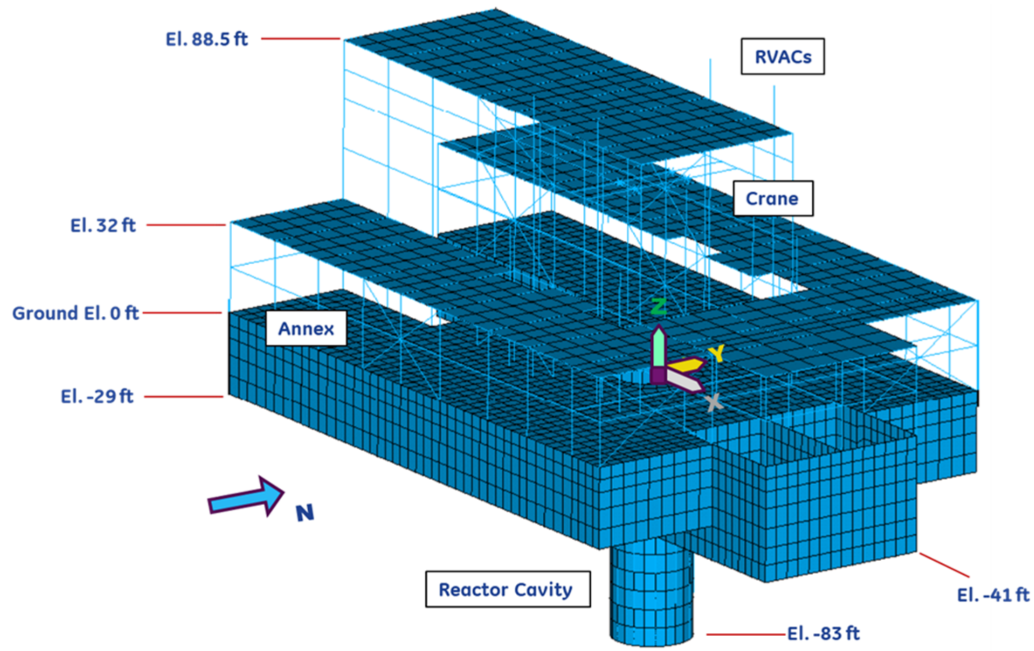


GEH RIPB Seismic Design Experience and Perspectives

VTR Seismic Response Analysis

- Soil-structure interaction analysis were performed for the VTR U11 Facility to develop seismic demands for conceptual design of VTR safety SSCs
  - Input motion compatible to SDC-5 EDRS was used to calculate structural demands and in-structure seismic responses
  - Seismic demands for conceptual design of SDC-3 SSCs are obtained by scaling down the calculated SDC-5 DBE responses by half

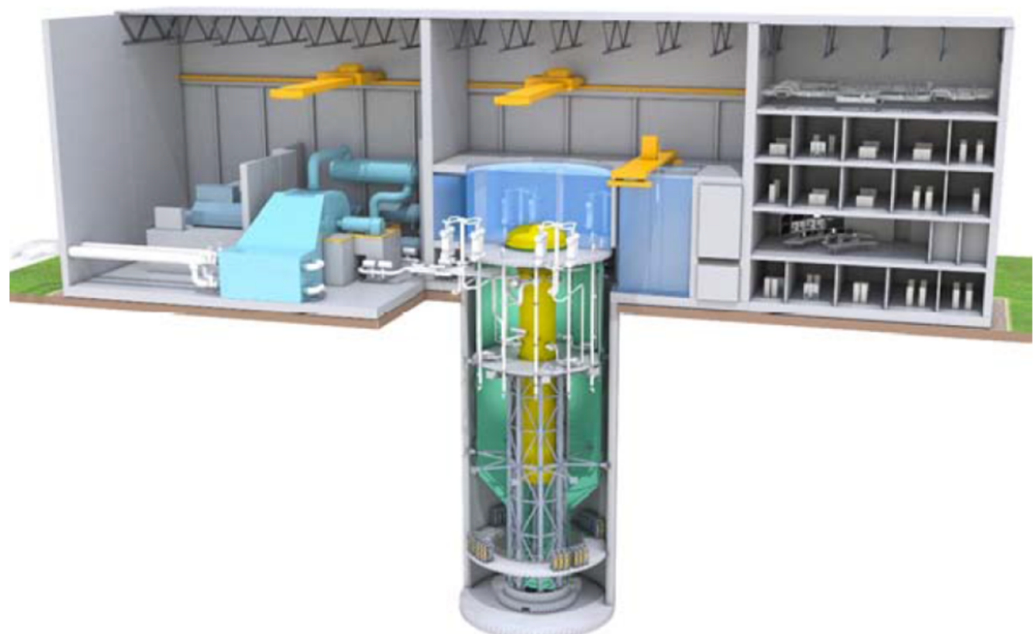
VTR U11 Facility Seismic Model



GEH RIPB Seismic Design Experience and Perspectives

BWRX-300 SMR

- GEH's new BWRX-300 Small Modular Reactor (SMR)
  - is the tenth generation of the Boiling Water Reactor (BWR) and represents the simplest, yet most innovative BWR design
  - uses simple, natural phenomena driven safety systems that mitigate Loss-of-Coolant Accidents (LOCA)
  - is designed to optimize the cost of construction, operation, maintenance, staffing and decommissioning



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GEH RIPB Seismic Design Experience and Perspectives

BWRX-300 SMR Seismic Design

- BWRX-300 design follows the 10CFR50 requirements and regulatory guidance of Light Water Reactors (LWR) edition of NUREG-0800 Standard Review Plan (SRP) and relevant U.S. NRC Regulatory Guides (RGs)
  - All SSCs providing safety function during and after a design level earthquake are categorized in Seismic Category (SC-I) and designed for Safety Shutdown Earthquake (SSE)
  - SSE DRS are developed following the ASCE 43-05 performance-based approach for development of SDC-5 DRS
  - Design of SC-I SSCs considers only essentially elastic behavior without permanent deformations equivalent to ASCE 43-05 LS-D
- Seismic design of BWRX-300 SC-I SSCs ensures a consistent level of safety from earthquake induced failures (defined by level of response resulting in an onset of significant inelastic deformations) with a probability of unacceptable performance:
  - less than about 1% if the ground motion is equal to the SSE
  - less than about a 10% if the ground motion is equal to 1.5 x SSE

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GEH RIPB Seismic Design Experience and Perspectives

BWRX-300 SMR Seismic Design

- Per US NRC RG 1.143, RW-IIa seismic category is assigned to SSCs containing radioactive material which failure results in a total design basis unmitigated radiological release:
  - greater than 500 millirem per year at the boundary of the unprotected area, or
  - maximum unmitigated exposure to site personnel within the protected area greater than 5 rem per year
- RW-IIa SSCs are designed for one half SSE considering LS-D essentially linear response
- The seismic design of all other SSCs is in accordance with IBC-2015 and ASCE 7-10
- II/I interaction evaluations are performed for BWRX-300 SSCs which structural failure may adversely affect the safety function of SC-I SSC to ensure
  - These SSCs can withstand an SSE design level event with limited permanent deformations and minimal damage corresponding to ASCE 43-05 LS-C
  - The design considers increased seismic drifts to account for the limited inelastic response and ensure these SSCs will not collide with nearby SC-I SSC during an SSE design level event

HITACHI

GEH RIPB Seismic Design Experience and Perspectives

GEH RIPB Seismic Design Perspectives

- The VTR seismic design experience following the DOE RIPB approach is invaluable for the development of the new GE advanced sodium-cooled reactor technologies
  - Technology specific graded seismic design approach will be followed with an ASCE 43 SDC and LS seismic categorization based on results of safety analyses
  - Experience gained on the VTR project will help simplify the RIPB seismic categorization, analysis and design process and make them more effective
- GEH is closely following the prospects for implementation of RIPB approach for design of new LWR SMR technologies
  - The RIPB seismic design approach can take advantage of the improved safety features and benefit the BWRX-300 SMR design for safety and cost
- Updates in the performance based seismic design method are being evaluated for their use in the GEH designs after the issuance of the next ASCE 43 code revision

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Closing Remarks and Questions

OKLO INC

Oklo analysis: A proposed alternative risk-informed and performance-based regulatory framework for seismic safety at NRC regulated facilities

About us

Raised the first-ever, modern, venture-led, series A for a fission company

Granted an INL site use permit from Department of Energy

Selected to demonstrate recycle of spent fuel at Idaho National Laboratory

Became the first advanced fission company in the country to have a license application accepted by the U.S. Nuclear Regulatory Commission

Oklo develops clean energy generation sources with advanced fission to mitigate the social and environmental impacts of pollution as well as energy poverty.

Fast facts

20 year fuel life

1-2 MWe output

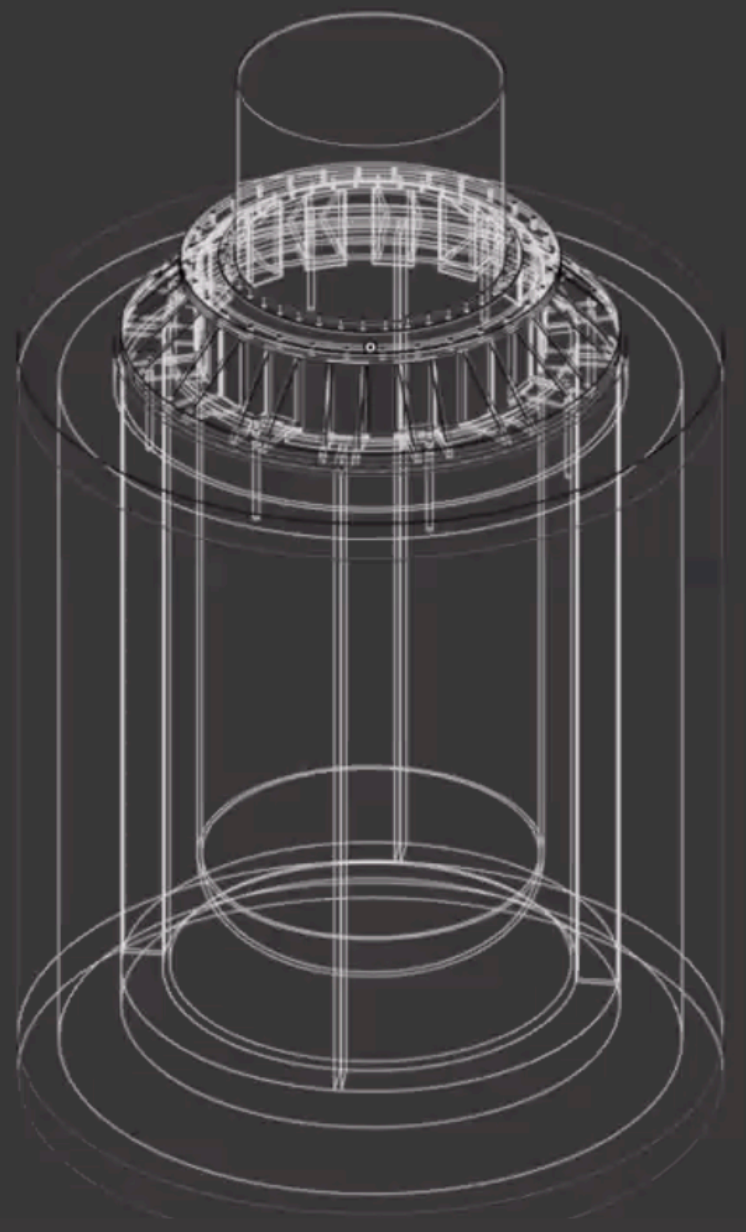
Integrated with solar

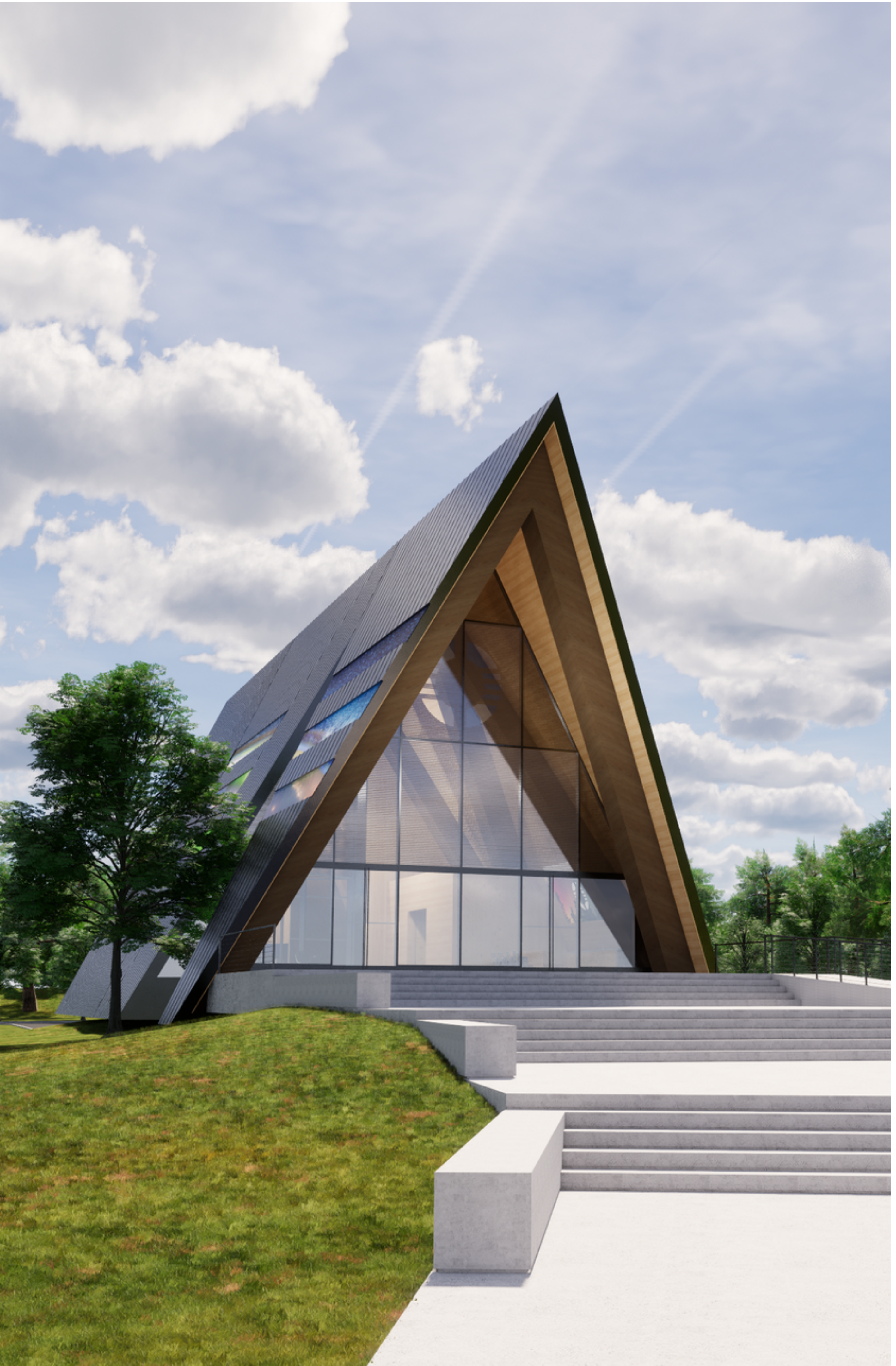
No water use

Cooled by natural forces

Recycles nuclear waste

Saves 1,000,000 tons of CO_2





COLA Approach

Submitted March 2020, Accepted June 2020

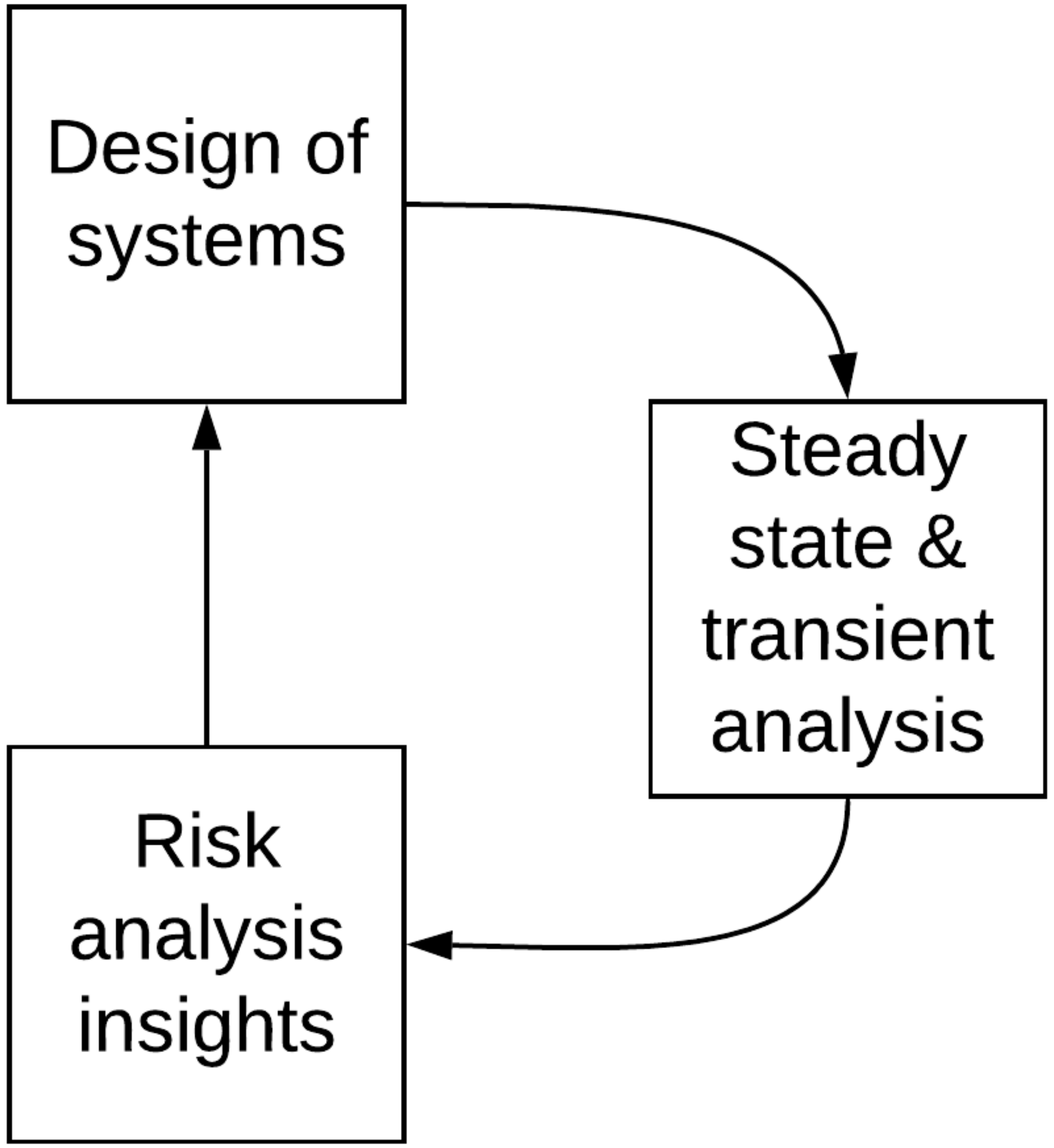
Approach overview

Pursuing custom COL for the INL Site

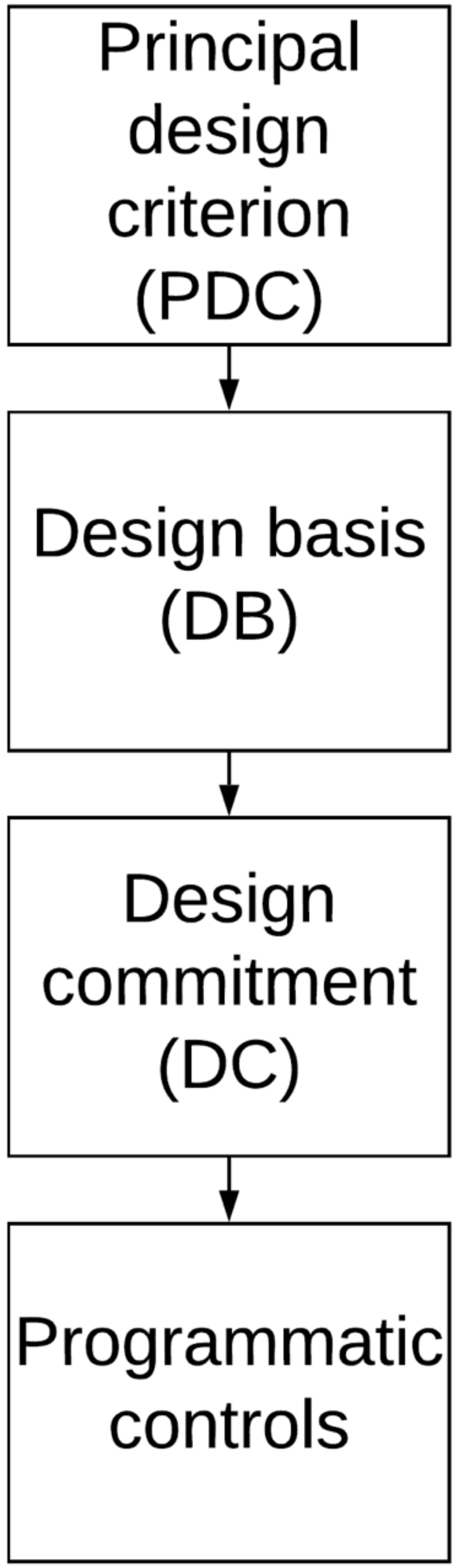
- Largely deterministic approach
- Coupled with risk analysis iterations
- Confirmed through performance-based programmatic controls

Changing the paradigm:

- QAPD – approved by NRC
- Continuity of staff – core team
- Few to no "safety-related" components



Summary



The design process is iterative with insight from risk and external hazards, and PDC allow for a functional derivation of DBs

QAPD, ITP, TS, and ITAAC provide the programmatic controls that ensure the DBs and DCs are met starting from manufacturing, initial testing, and on an ongoing basis.

Ultimately, the Aurora-INL COLA is performance based.

Seismic Considerations for the Aurora

What does not matter for the Aurora…

Electricity	• Does not rely on electricity to achieve a shutdown state • Does not require electricity for removal of decay heat • Passive heat removal from the fuel via heat pipes (no pumps, etc.)
Differential displacement	• Single building with no inlet or outlet pipes required for cooling • Does not rely on the power conversion system for heat removal
Reactivity oscillations	• No reactor coolant sloshing since heat pipe cooled • No rod oscillations since the rods are outside of the reactor

What matters for the Aurora…

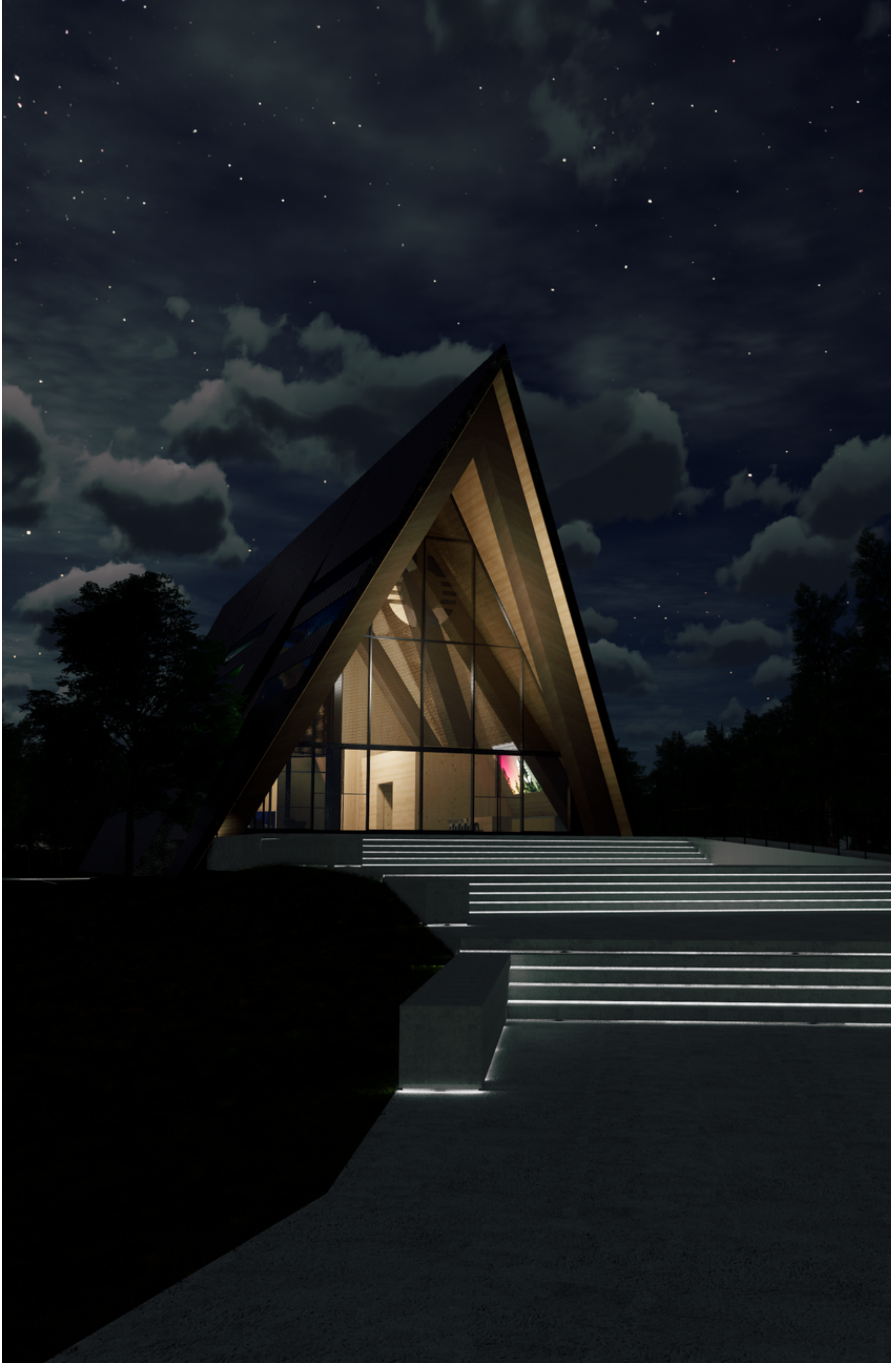
The shutdown rods insert into the core.

The integrity of the shutdown rods is protected.

The reactor module maintains its integrity.

Aurora seismic analysis conclusions

1	The shutdown rods insert into the core.	→	Large tolerances allow for the insertion of shutdown rods.
2	The integrity of the shutdown rods is protected.	→	Structural analysis found the reactor module is robust during an extreme earthquake.
3	The reactor module maintains its integrity.	→	Structural analysis found the reactor module is robust during an extreme earthquake.



RIPB Seismic Standard

LMP/ASCE 43

Benefits of LMP/ASCE43

- ✓ Performance-based approach
- ✓ Reduces overdesigning of systems
- ✓ Appropriate scaling SDC of components proportionally to the seismic risk

Concerns with LMP/ASCE43

Suggests that the current framework is not risk-informed

Assumes highest risk category and most fragile category for components

Use of CDF and LERF to compare seismic PRA end states

Requires the use of a seismic PRA

These aspects of the LMP/ASCE43 methodology could be an undue burden to the Aurora and potentially other advanced reactor designs.

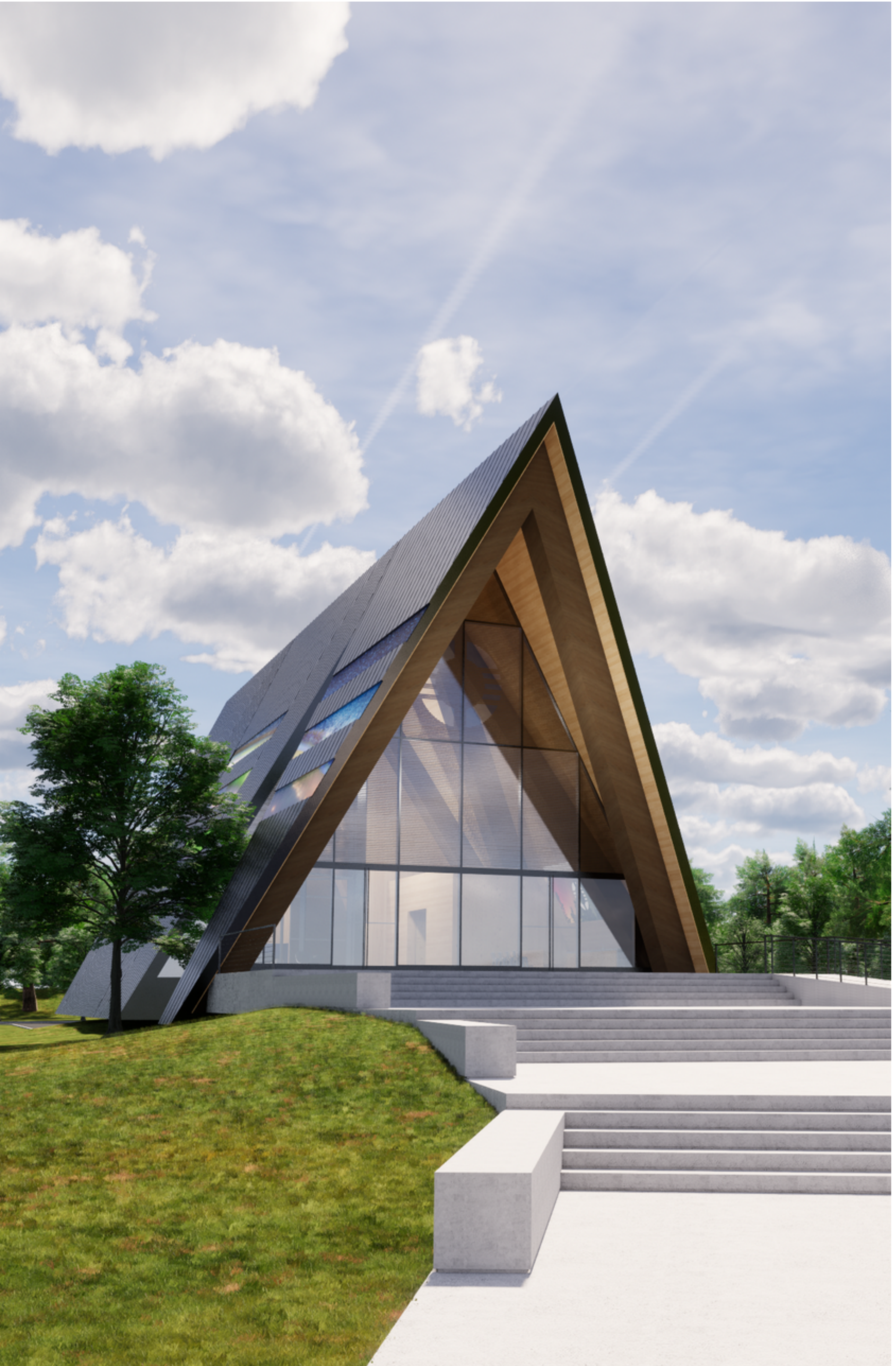
LMP/ASCE 43 methodology

Step	Description	Applied to the Aurora
Step 1	Select the initial ASCE 43 SDC and LS categories for each SSC and use the LBEs identified in the internal-events analysis, including the internal-events-based safety classification of various SSCs	Assume the SSCs for the Aurora are SDC-5 and LS-D
Step 2	Design SSCs according to applicable codes for the chosen SDC/LS	Design the SSCs to an SDC-5 and LS-D level
Step 3	Determine the fragility of SSCs	Determine the fragilities and design criteria for Aurora SSCs in consultation with the LMP Component Group
Step 4	Perform the SPRA in accordance with applicable codes and guidance	Perform an SPRA, then compare it to a CDF or LERF.
Step 5	Check SPRA results against the F-C Target and cumulative risk criteria, as well as defense-in-depth, reliability, and other risk-informed decision-making factors. Revise SDC and LS for SSCs as appropriate.	If an SPRA was performed, an F-C curve developed, and an LMP safety class initially assigned to all SSCs, there may be the ability to reduce the SDC for the component.
Step 6	Repeat Steps 2 to 5, as needed	Possibly perform another complete design and iterate.
Step 7	Finalize the selection of ASCE 43 SDC and LS categories for the licensing basis seismic design	Perform another, final SPRA.

Overall conclusion for the Aurora

This proposed standard could be burdensome for advanced reactor designs and, therefore, might not meet the intent of NEIMA.

For advanced reactors like the Aurora, the same level of safety could be achieved through less burdensome deterministic methodologies that are widely already accepted in the seismic analysis community.



Seismic Analysis for the Aurora

From the Aurora-INL COLA

Aurora seismic analysis steps

1. Determine and describe the appropriately bounding earthquake for the desired region. In the case of the Aurora, the desired region bound the United States.
2. Determine the relevant structures, systems, or components (SSCs) that are potentially vulnerable to a seismic event and that require further analysis.
3. Analyze the relevant SSCs to determine impact of the appropriately bounding earthquake for the U.S.
4. Summarize the results of the seismic analyses to determine if the overall safety of the facility is impacted.

Aurora seismic site commitment

Seismic basis: The proposed site will not damage the Aurora reactor by a large ground acceleration.

Seismic event commitment: The largest recorded PGA for the proposed site will be determined under ASCE 7. If the PGA of the proposed site exceeds 0.50 g*, additional analyses must be performed.

*As a conservative measure, the Aurora seismic site commitment amplifies the ASCE 7 value to the the UHRS frequency associated with an SDC-5 facility. The risk level associated with the Aurora meets the ANSI/ANS-2.26 definition of an SDC-2 facility.

Benefits of the Aurora seismic methodology

- ✓ Generic to the region of interest
- ✓ Generic to the reactor design
- ✓ Provides a bounding analysis, reducing PRA reliance
- ✓ Does not require site specific investigations, unless the proposed site location is not bound by the analysis
- ✓ Uses the ASCE 7, which is publicly available data for the entire United States

Conclusions

1. The LMP/ASCE43 standard might not be technology-inclusive
2. A seismic PRA is not always necessary to determine the design criteria for SSCs
3. For low-risk reactor designs, the SDC can be determined early in the design process due to a small source term
4. Language should be added to the standard to incentivize simple, easy to analyze, robust reactor designs
5. There needs to be additional language in the standard to reinforce that this is a voluntary approach that is valuable only in the case that a sufficient safety case cannot be made with a bounding analysis


Thank you

NuScale Experiences and Perspectives for Using RIPB Methods for Seismic Design

September 2, 2020

Luke McSweeney
Probabilistic Risk Assessment

NuScale Nonproprietary

NuScale Current Status of Seismic Risk Modeling

- NuScale performed a seismic margin assessment (SMA) as part of its design certification application
  - Seismic PRA deferred to after COLA stage
  - Hazard not quantified without a site
- **Main finding:** Seismic risk profile is very different from operating fleet
  - No relay chattering
  - Power supply (diesels, cabinets) failures are not risk significant
  - Structural failures and valve mechanical failures dominate

Civil Structural Design/Analysis – PRA interface

- PRA uses SSE (0.5g PGA) analysis inputs
  - In-structure response spectra (ISRS) for component excitation
  - RXB analysis for structural forces & moments
- Analysis scope limited to SSCs that could cause core damage if failed
- Capacity is derived from structural codes (ACI 349, ASCE 43, AISC N690) and material properties
- Conservatisms are replaced with median-centered values and uncertainties
- Demand-to-capacity ratios translated into earthquake scale factors
- Credit for ductility, when allowed

To date, no SMA results or insights have required revision of civil structural design or analysis (HCLPF > 0.84g)

NuScale risk-informed decision making process

- PRA is kept up to date to support risk-informed decisions
- Every engineering change request is evaluated for risk impacts
- PRA membership on engineering change board and DRAP panel
- Additional risk evaluations are conducted to potential design alternatives
- RIPB activities are conducted by trained personnel on a project-by-project basis following best practices

Examples of Potential Risk-Informed Decisions in Seismic Design/Analysis

- Demand/capacity margins for wall shear/flexure could be adjusted to account for reinforcement ductility (when allowed)
- Separate criteria for component performance – operability vs. safety function
- Indirect, non-conservative effects of conservative design criteria, e.g. concrete compressive strength affects stiffness, modal response
  - Complicates direct scaling of seismic loads

- For advanced reactor designs, seismic risk is likely to be dominated by severe, very rare earthquakes
  - Incorporation of nonlinear soil-structure interaction effects (EPRI suggestion)
  - Consideration of catastrophic failure modes (e.g. wall pushover) in structural analysis methods

Challenges in applying RIPB methods

- Without a site hazard profile, performance goals (e.g. 1E-5/y) cannot be defined
  - Wide hazard uncertainty bands and unclear acceptance criteria
- SMAs and SPRAs already use best-estimate limit states and corresponding consequences
- SMAs do not currently allow for frequency weighting by limit state, i.e. a binary definition of success or failure is typically required
  - Possible future application for limit states in seismic PRAs, provided accident sequences can incorporate relevant consequences
- Fragility results not always sensitive to seismic design improvements
  - In an SMA, a small number of seismic failures drive the top-level results
  - Benefit of relaxed failure criteria may be negated by higher uncertainties

Summary – Towards a Workable RIBP Seismic Design

- Two facets of seismic risk require further development

1. Use of hazard curves
   - Hazard uncertainty bands are very wide, making it difficult to define frequency of occurrence thresholds that incorporate uncertainty
   - Severe diminishing returns when applied to robust structures
2. Need for staggered failure criteria for structures according to their severity and safety impact e.g.
   - ASCE 43 limit states are a good start, but engineering judgment is required in their application
   - Systematic screening for structural failures and their potential to lead to core damage and large release
   - Need for categorical assumptions about consequences of failure beyond yield/crack propagation

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GENERAL DISCUSSION AND FEEDBACK ON RIPB APPROACH

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