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DOE-HDBK-1220-2017

DOE HANDBOOK

NATURAL PHENOMENA HAZARDS ANALYSIS AND DESIGN HANDBOOK FOR DOE FACILITIES

U.S. Department of Energy
Washington, D.C. 20585

FOREWORD

This handbook is approved for use by all Department of Energy (DOE) components and their contractors.

This handbook provides advice and recommended good practices for implementing DOE-Standard (STD)-1020-2016, *Natural Phenomena Hazards Analysis and Design Criteria for Department of Energy Facilities*. The handbook has been developed primarily for experts in each Natural Phenomena Hazards (NPH) field.

The handbook does not add any new requirements to those stated in DOE-STD-1020-2016, nor does it alter in any respect that standard's requirements or recommended practices.

Beneficial comments (recommendations, additions, and deletions), as well as any pertinent data that may be of use in improving this document, should be e-mailed to: nuclearsafety@hq.doe.gov or sent to:

Office of Nuclear Safety (AU-30)
Office of Environment, Health, Safety and Security
U.S. Department of Energy
19901 Germantown Road
Germantown, MD 20874

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[1] Gaps in the numbering scheme below relates to the fact that not all sections of DOE-STD-1020-2016 are discussed in this handbook. Entries correspond to parallel sections of that standard.

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1.0 INTRODUCTION

1.1 Background

This handbook has been developed to capture and update useful and applicable technical information previously found in Department Of Energy (DOE)-Standard (STD)-1020-2002, DOE-STD-1020-2012, and DOE Guide 420.1-2.

1.2 Purpose

This handbook is a companion document to DOE-STD-1020-2016, *Natural Phenomena Hazard Analysis and Design Criteria* (referred to hereafter as "the Standard"). It identifies good practices that can be used to meet the Standard's requirements and guidance; it also offers general technical advice on topics related to natural phenomena hazard (NPH) mitigation. The handbook provides clarification of the rationale for some provisions in the Standard and cites references to assist all DOE components and their contractors in applying the Standard. With respect to the Standard's requirement statements, the handbook should be viewed as an implementation aid, not as an interpretation of the requirements.

1.3 Applicability

The applicability of the handbook is the same as that of DOE-STD-1020-2016.

1.4 Organization

The handbook follows the format and organization of the Standard.

1.5 Acronyms

ANS	American Nuclear Society
ANSI	American National Standards Institute
APC	Atmospheric Pressure Change
APE	Annual Probability of Exceedance
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
DBE	Design Basis Earthquake
DBFL	Design Basis Flood Level
DBPL	Design Basis Precipitation Level
DBRP	Design Basis Return Period
DOE	Department of Energy
DRS	Design Response Spectra
FEMA	Federal Emergency Management Agency
FDC	Flood Design Category
HEC	Hydrologic Engineering Center
HMS	Hydrologic Monitoring Center
IBC	International Building Code
LS	Limit State
NDC	NPH Design Category
NPH	Natural Phenomena Hazard
NQA	Nuclear Quality Assurance
NRC	Nuclear Regulatory Commission
NWS	National Weather Service
ORP	Office of River Protection
PC	Performance Category
PDC	Precipitation Design Category
PFHA	Probabilistic Flood Hazard Assessment
PISA	Potential Inadequacy in the Safety Analysis
PPHA	Probabilistic Precipitation Hazard Assessment
PSHA	Probabilistic Seismic Hazard Assessment
PTHA	Probabilistic Tsunami Hazard Analysis

PWHA	Probabilistic Wind Hazard Assessment
QA	Quality Assurance
RAS	River Analysis System
SASSI	System for Analysis of Soil-Structure Interaction
SDC	Seismic Design Category
SEI	Structural Engineering Institute
SME	Subject Matter Expert
SSCs	Structures, Systems and Components
SSHAC	Senior Seismic Hazard Analysis Committee
STD	Standard
TPG	Target Performance Goal
USGS	United States Geological Survey
VDC	Volcanic Design Category
VHA	Volcanic Hazard Analysis
WDC	Wind Design Category

2.0 GENERAL CRITERIA AND GUIDANCE FOR NPH DESIGN

2.1 Overview

In accordance with the Standard, this section provides an overview of DOE's design approach for structures, systems, and components (SSCs) required to withstand the effects of NPHs. Section 2.1 addresses all facilities; Section 2.2 addresses non-nuclear facilities; and Section 2.3 addresses Hazard Category 1, 2, and 3 nuclear facilities.

2.1.1

Public Law 101-614 and Executive Order 13717 can be met for DOE non-nuclear facilities by performing seismic safety evaluations according to "Standards of Seismic Safety for Existing Federally Owned and Leased Buildings"—contained in Interagency Committee on Seismic Safety in Construction Recommended Practice RP8, NIST GCR 11-917-12—and International Building Code (IBC)-2015, as specified in the Standard. Public Law 101-614 and Executive Order 13717 can be met for DOE Hazard Category 1, 2, and 3 nuclear facilities by performing seismic safety evaluations according to American Society of Civil Engineers (ASCE)/Structural Engineering Institute (SEI) 43-05 or IBC-2015, as specified in the Standard.

2.1.2

The flood design parameters of Executive Orders 11988 and 13690 are applicable to all DOE facilities. Nuclear facilities that comply with Sections 5 and 7 of the Standard also comply with these executive orders. When flood requirements in the executive orders are more stringent than the IBC requirements for non-nuclear facilities, the executive order requirements should be applied.

2.2.1 Facilities without Chemical or Toxicological Hazards

Facilities without chemical or biological hazards are designed in accordance with IBC-2015. Chapter 16 of IBC-2015 is referenced in the Standard to highlight "Risk Categorization" in Table 1604.5.

2.2.3 Radiological Facilities

The Standard requires that the NPH design of SSCs in radiological facilities follow the criteria of IBC-2015. However, IBC 2015 does not explicitly address radiological facilities. The facility should be designated as IBC Risk Category IV unless the technical basis exists for a lower risk category.

2.2.5 Facilities Containing Explosives

Additional information on best practices may be obtained from DOE's Explosives Safety Committee and from the Department of Defense Explosives Safety Board (http://www.denix.osd.mil/ddesb/home).

2.2.6 Criteria for Evaluating Volcanic Eruption Hazards

Section 2.2.6 of the Standard does not require a volcanic hazards assessment for non-nuclear facilities. However, Section 8 of the Standard may be used if desired to determine potential ash loads on roofs of non-nuclear facilities.

2.3 Hazard Categories 1, 2, and 3 Nuclear Facilities

2.3.2 Scope of SSCs for NPH Design

SSCs whose failure could adversely affect the safety functions of DOE Hazard Category 1, 2, or 3 nuclear facilities should be categorized in accordance with Section 2.3.3 and Section 2.4 of the Standard. The use of these provisions is needed to meet the system interaction mitigation criteria of American National Standards Institute (ANSI)/American Nuclear Society (ANS) 2.26-2004, *Categorization of Nuclear Facility Structures, Systems, and Components for Seismic Design* (Revision 2010). In general, NPH design categorization is not carried out for non-nuclear facilities.

2.3.3 Criteria and Guidance for Establishing NPH Design Categories for Safety SSCs

2.3.3.1

As stated in Section 2.3.3.1 of the Standard, the NPH Design Category (NDC) of an SSC is determined based on the severity of unmitigated consequences from radiological and/or chemical releases of SSC failure using the categorization criteria given in Table 2-1. The NPH design categorization methodology has been detailed in ANSI/ANS-2.26-2004 (R2010). In DOE-STD-1020-2012, DOE adopted seismic hazard design categorization criteria and methodology for use with other NPH hazards. Thus, when applying Table 2-1 and the categorization methodology of ANSI/ANS-2.26-2004 (R2010), seismic design category (SDC) means NDC.

Since the NDC is based on unmitigated consequences of SSC failure, credit cannot be taken for the mitigating effects of any SSC or procedure when estimating the consequences for the purpose of categorization.[2] Therefore, the design category of an SSC for all NPH types can be the same as the NDC. However, the mitigating effects of other SSCs should be considered when evaluating the demand resulting from an NPH. The examples below illustrate this point:

1. ***Scenario***: A glovebox situated inside a concrete building will need to be designed to perform a safety-related confinement function during and after an NPH event. The building has already been designed as NDC-3 based on its failure consequences, and it has a safety-related confinement function. The building also can act as a barrier against inundation of the glovebox.

2. ***NDC of the Glovebox:*** For determining the NDC of the glovebox, the consequences of glovebox failure are determined without taking credit for any mitigating effects of other SSCs including the building. In this scenario, glovebox failure could result in a dose to a member of the public above 25 rem, so the glovebox is determined to be NDC-3, because no credit is taken for the building's mitigating confinement function.

3. ***Extreme Wind Design Load for the Glovebox***: Since the building has been designed as NDC-3 or wind design category (WDC)-3 to withstand the extreme wind load, in designing the glovebox, credit for the mitigating shielding effects of the building enclosure can be taken to determine the design basis extreme wind pressure on the glovebox. As a result, the design basis wind pressure on the glovebox is likely to be negligible.

[2] Unless "robustness of the mitigating SSC can be demonstrated," see Section 6.3.2.5 of ANS/ANS-2.26-2004 (R2010).

4. ***Seismic Design Load for the Glovebox***: The building cannot mitigate the seismic loads on the glovebox, rather it may amplify the loads. Hence, the glovebox is required to be designed for seismic loads. However, since the building is NDC-3 or SDC-3, and is designed to perform confinement function, the glovebox does not have to be designed to withstand the impact loads from collapsing building structural components, because the building would not be postulated to collapse for an NDC-3 level earthquake.

5. ***Flood Design Load for the Glovebox***: Since the building and any required SSCs (e.g., active ventilation), have been designed as NDC-3 or Flood Design Category (FDC)-3 to withstand the extreme flood load, in designing the glovebox, credit for the mitigating effects of the building enclosure to prevent inundation of the glovebox can be taken. Also, in determining the design basis hydrostatic and hydrodynamic pressure on the glovebox, credit can be taken for the building enclosure acting as a barrier, resulting in a negligible pressure.

2.3.3.4

The purpose of Section 2.3.3.4 is to reduce the number of design iterations, and thereby achieve some cost efficiency. Design basis seismic motion is needed to design SSCs, and to determine it, one needs to know what would be the highest SDC of the SSCs in the facility, because the level of Probabilistic Seismic Hazard Assessment (PSHA) to be performed depends on the highest SDC of the SSCs in the facility. Even when a site-specific PSHA is available, it becomes necessary to assume the anticipated highest SDC of the SSCs in the facility to determine the design basis seismic return period and the associated seismic load used for preliminary design. SDC-3 has been recommended as the default SDC because past experience has shown that most of Hazard Category 2 facilities have some SDC-3 SSCs and very few or no SDC-4 SSCs.

2.4 General Criteria and Guidance for Defining Limit States

2.4.1

As a general matter, NPH-caused failure of an SSC may be defined in terms of maximum deformation level, structural damage (i.e. concrete cracking, yielding, permanent deformation), level of water intrusion or submergence, or amount of leakage that it can sustain without compromising its safety function.

2.4.4

Section 2.4.4 provides an additional design requirement that may be controlling in some circumstances. For example, if the safety function of a building is restricted to life safety of its occupants, the limit state (LS) is deformation short of collapse, or LS-A per ASCE 43-05 and ANS-2.26. On the other hand, if the safety function of a building is confinement permitting little or no leakage, the limit state is LS-C or LS-D, depending on the ventilation system per ASCE 43-05 and ANS-2.26. Note that LS-C allows limited inelastic deformation while LS-D has negligible damage and cracking.

3.0 CRITERIA AND GUIDELINES FOR SEISMIC DESIGN

This section of the handbook discusses the best practice methods for designing SSCs to meet the criteria and guidance given in Section 3 of the Standard.

3.1 Seismic Design Categorization and Limit States

In ASCE/SEI 7-10, 'R' factors are independent of Risk Category, and its use corresponds to a near-collapse of the seismic-force resisting system. The development of Actual Response Modification Coefficients (R_a) factors assumed that this level of damage is consistent with ASCE/SEI 43-05 as Limit State A. Thus, when an 'R' factor is used to reduce the elastically-computed seismic load of a Risk Category II structure (i.e., SDC-1), for which the Importance factor is 1.0, the structure is assumed to reach a deformation level close to what would cause incipient collapse. Using the same 'R' factor for a Risk Category IV structure (i.e., SDC-2), for which the Importance factor is 1.5, the structure is assumed to reach a lower deformation level equivalent to Limit State B. Thus, the annual probability for the Risk Category IV/SDC-2 structure to reach a Limit State A is lower than that for a Risk Category II/SDC-1 structure.

3.1.2

ASCE/SEI 43-05 and ASCE 4-98 should be used in designing SDC-3 through SDC-5 SSCs, but DOE-STD-1020-2016 takes precedence if any conflict arises.

3.1.4

Section 3.1.4 of the Standard requires that SDC-1 SSCs whose failure is defined by Limit State A be designed based using IBC-2015 requirements for Risk Category II facilities, while SDC-2 SSCs whose failure is defined by Limit State B be designed using IBC-2015 requirements applicable to Risk Category IV facilities. For SDC-1 and 2 SSCs not having these respective limit states, Section 3.1.4 of the Standard requires the use of Table 3-1 which provides R_a to be used instead of the 'R' factors given in ASCE/SEI 7-10. An accurate determination of these factors is complicated because 'R' factors in ASCE/SEI 7-10 include the effects of inelastic energy absorption (ductility), system overstrength and damping, while the inelastic energy absorption factor in ASCE/SEI 43-05 intentionally omits the beneficial effects of system overstrength. In addition, the magnitude of deformation allowed by ASCE/SEI 7-10 may exceed Limit State A for some structural systems.[3]

The Ra values given in Appendix A of DOE-STD-1189-2008[4] to SDC-2C (SDC-2, Limit State C) and SDC-2D (SDC-2, Limit State D) were found to be excessively conservative. This anomaly occurs because SDC-3 SSCs are designed following ASCE/SEI 43-05 requirements, whereas SDC-1 and SDC-2 SSCs are designed following ASCE/SEI 7-10 requirements. The limit states in ASCE 7-10 and ASCE 43-05 are not equivalent and the Response Modification Coefficients given in Table 3-1 of the Standard that were intended to

[3] The complexities related to selecting R_a values are under study by an ASCE/SEI 43 working group. Topics being addressed by the committee include deformation limits, system overstrength, equivalency of analytical methodologies, consistency in levels of achieved safety, etc. Until this committee provides new or additional guidance, the R_a values in Table 3-1 of the Standard, as discussed and modified in the text above, should be used, noting that the intent of Table 3-1 is to *reduce* the R factors, not to increase them.

[4] DOE-STD-1189-2008 has been superseded by DOE-STD-1189-2016, which does not contain this technical material.

compensate for the differences in these two sets of requirements, although they do not always compensate adequately. For this reason, the Response Modification Coefficients given in Table 3-1 of the Standard for Limit State D can be further adjusted by placing a lower limit $R_a \geq 1.2$ for Limit State C and Ra=1.2 for Limit State D.

3.2 Selection of Design Basis Earthquake (DBE) Return Period to Approximately Meet Target Performance Goal (TPG)

3.2.1

The design basis seismic loads or demands for SDC-3 through SDC-5 SSCs in nuclear facilities are determined using a three-step, PG-based method:

Step 1: Determine the SDC and limit states for SSCs, using the methodology and criteria provided in ANSI/ANS-2.26 and Table 2-1 of the Standard, which group SSCs into SDCs.

Step 2: Select TPGs for seismic design of SDC-3, SDC-4, and SDC-5 SSCs using the methodology given in the Standard for seismic design and evaluation of SSCs. Recommended values of TPGs for SDC-3, SDC-4, and SDC-5 SSCs are provided in Table A.1 of ANSI/ANS-2.26 and in Table 1-3 of ASCE/SEI 43-05.

Step 3: Develop a Design Response Spectra (DRS) in accordance with ASCE/SEI 43-05.

Safety analysts, NPH design engineers, and SSC design engineers should work together to implement the above steps. The criteria and methodology given in DOE-STD-1020-2016 are based on a combination of deterministic and probabilistic approaches. These were developed to achieve a set of probabilistic TPGs given in ANSI/ANS-2.26 and ASCE/SEI 43-05. Note that the term DBE in the Standard should be interpreted as the DRS to be consistent with ASCE/SEI 43-05.[5]

3.2.2

IBC-2015 limits the permissible reduction in seismic force when a site specific hazard study is used, and this limitation should be maintained.

3.3 Site Characterization for Seismic-Related Hazards

Section 3.3 of the Standard requires the use of ANSI/ANS-2.27-2008 (R2016), *Criteria for Investigation of Nuclear Facility Sites for Seismic Hazard Assessments*. Note that the criteria of ANSI/ANS-2.27 also contain guidance on conducting geotechnical investigations, site response analyses, liquefaction, ground settlement and slope failure analyses that are directly applicable to SDC-3, to SDC-5 SSC and provide useful information for SDC-1 and SDC-2 SSC evaluations.

ANS 2.30-2015, *Criteria for Assessing Tectonic Surface Fault Rupture and Deformation at Nuclear Facilities,* provides useful guidance on assessing the surface rupture hazard at a site and should be used as appropriate in site characterization.

Section 4.3.2, “Site Investigations,” of ANSI/ANS-2.27-2008 (R2016) provides criteria for geotechnical site investigations. Prior development of an investigation program plan for the site may reduce the time

[5] See Appendix C for further details on TPGs.

and effort needed to evaluate SDC-3, 4, and 5 facilities. The program plan is to (a) provide a documented basis for the results of the review of the available information, data, and available existing site investigations described in Section 4.3.1, and (b) document the plans for the surface/subsurface investigation, including investigation methods; obtaining, handling, tracking soil and rock samples; and laboratory testing. In addition, in accordance with the general quality assurance requirements of Section 10 of the Standard, the site investigation program plan should include a quality assurance plan meeting the intent of American Society of Mechanical Engineers (ASME) Nuclear Quality Assurance (NQA)-1, Subpart 2.20, *Quality Assurance Requirements for Subsurface Investigations for Nuclear Facilities.*

Section 4.3.2.2, "Laboratory Testing," of ANSI/ANS-2.27-2008 (R2016) briefly discusses the importance of minimizing disturbance of soil during sample removal from tubes and specimen preparation. Whenever the in-situ strength of the soil should be determined, "in-tube" testing is preferred to conventional methods involving sample removal to minimize the strength degradation effects from sample disturbance.

3.4 Probabilistic Seismic Hazard Analysis (PSHA)

3.4.1

Section 3.4.1 of the Standard states "In specifying a lower-bound magnitude as required by Section 5.1.1 of ANSI/ANS-2.29-2008, the guidance in EPRI Technical Report 1012965, *Use of CAV in Determining Effects of Small Magnitude Earthquakes on Seismic Hazard Analyses*, may be used if site-specific sensitivity analyses demonstrate that site hazard at return periods of interest is not unduly reduced." The phrase "unduly reduced" corresponds to a 10% reduction in the response spectra. NUREG/CR-6728 was used to develop the site response provisions in Section 2 of ASCE/SEI 43-05 and Section 5.4 of ANSI/ANS-2.29-2008.

3.4.2

Section 3.4.2 applies to SDC-3 through SDC-5 SSC. The PSHA should develop strain-compatible soil properties for use in SSI analyses consisting of the median properties and the variability in those properties. ASCE 43-05 provides requirements for the development of foundation input motions.

3.5 Building and Equipment Response Analysis to Determine Seismic Demand

3.5.1

The seismic wave incoherence provision in Section 3.3.1.10 in ASCE 4-98 is obsolete and thus cannot be used. However, incoherent seismic motions may be used if justified on a case-by-case basis.

Section 3.5.1 of the Standard requires that soil-structure interaction be considered in the determination of seismic demand on structures. The System for Analysis of Soil Structure Interaction, a computer program referred to as "SASSI," (Lysmer, 1999) is often used to solve seismic SSI problems. The subtraction method in SASSI is an approximation and its use beyond the frequency range of its applicability may lead to spurious results in computed responses (see U.S. DOE 2011 and OE-3, Report 2011-2, "SASSI Software Problem"). The extended (modified) subtraction method will improve the solution and will reach the same solution as the direct method when the number of interaction nodes is sufficiently extended. Engineers using other (non-SASSI) computer codes should learn from the "SASSI Software Problem" report and ensure that their solutions are within the bounds of their validation set.

The use of approximations to the flexible volume method implemented in SASSI (i.e., the subtraction or extended subtraction methods) should be validated for the model plan dimensions and depth, range of soil properties and the excavated soil volume. An acceptable validation method for large problems is to approximate the plan dimensions of the building footprint with a symmetric shape and analyze the excavated soil volume using quarter-symmetry with the flexible volume method. Similarity of flexible volume and subtraction transfer functions in the bottom of the excavation over the entire frequency range of interest would indicate that the subtraction method does not create spurious results for the project geometry and soil properties (U.S. DOE 2011 and OE-3).

3.6 Building and Equipment Capacity Evaluation

3.6.1

The seismic load path should be documented in the engineering calculation. Redundant structures should be incorporated into design whenever practical. Meeting the ductile detailing requirements of IBC-2015 or ASCE/SEI 43-05 provides adequate ductile elements and connections. Mechanical equipment except for movable equipment (such as forklifts) should be anchored to resist seismic loads, including loads on roofs, walls, floors, and platforms.

3.6.4

Seismic qualification of equipment may be performed by: analysis, testing, actual earthquake experience data, or generic shake table test data. Combinations of these methods may be used to qualify different attributes of a component considering its safety significance.

4.0 CRITERIA AND GUIDELINES FOR EXTREME STRAIGHT-LINE WIND, TORNADO, AND HURRICANE DESIGN

4.1 Wind Design Categorization

Section 4 of the Standard provides criteria and guidelines for designing SSCs subject to extreme straight-line winds, tornados, hurricanes, tornado atmospheric pressure change (APC), tornado-generated missiles, and hurricane-generated missiles. The major steps in the design and evaluation process of SSCs subject to wind hazards are described and illustrated in Figure 4-1 below.

4.1.1 General Approach

Step 1. Determine Wind Design Category (WDC) for each SSC

As shown in Figure 4-1, the first step in wind design and evaluation is the determination of WDC by applying the provisions given in Sections 2.3 and Section 4.1.2 of the Standard. Accordingly, SSCs would be placed in WDC-1 through WDC-5 categories based on the SSC failure consequences. WDC-1 and WDC-2 SSCs are designed for extreme wind-related hazards using the criteria given in IBC-2015 for Risk Category II and Risk Category IV facilities, respectively, whereas WDC-3 through WDC-5 SDCs are designed using the guidelines and criteria provided in Section 4 of the Standard or the guidelines and criteria provided in ANSI/ANS-2.3-2011 (R2016).

Step 2. Perform Wind Parameter Characterization

For WDC-3 through WDC-5 SSCs, design basis wind speeds given in Figures 1 through 4 and in Table 2 of ANSI/ANS-2.3-2011 (R2016) can be used. The design basis wind speeds in ANSI/ANS-2.3-2011 (R2016) are based on a generalized regionalization of wind speeds. Based on this generalized regionalization, some sites may choose to perform site-specific wind hazards assessments, and if so, then acquisition of local representative meteorological data for a site-specific wind parameter characterization study will be needed. For WDC-1 and WDC-2 SSCs, site-specific wind characterization is not needed, because design basis wind speeds applicable for Risk Category II and Risk Category IV facilities can be obtained directly from IBC-2015.

Step 3. Perform Probabilistic Wind Hazard Assessment

For WDC-3 through WDC-5 SSCs, if design basis wind speeds given in Figures 2 through 4 and Table 2 of ANSI/ANS-2.3-2011 (R2016) are applicable, no site-specific Probabilistic Wind Hazard Assessment (PWHA) is required, unless the facility is located in a site with extreme local topography and/or in close proximity to large bodies of water. These special conditions should be considered when performing a site-specific PWHA, pursuant to Section 4.3.2 of the Standard. For WDC-1 and WDC-2 SSCs, no site-specific PWHA is required.

Step 4. Design SSCs to Mitigate Wind-Related Hazards

The primary parameter used in designing SSCs to withstand wind hazards is wind speed. The design basis wind speed is selected based on the WDC of the SSC. Once the appropriate WDC is determined for each of the facility SSCs (see Step 1, above), the Design Basis Mean Return Period (DBMRP) for the extreme wind hazards applicable for the site and the facility are selected from Table 4-1 in the Standard for WDC-3 through WDC-5. The design basis wind speed, APC from a tornado, and types of missiles are then selected as a function of the WDC from ANSI/ANS-2.3-2011 (R2016) for use in designing or evaluating the SSCs in the facility. Design criteria for WDC-3 through WDC-5 are provided in Section 4.4.2 of the Standard. WDC-1 and WDC-2 SSCs are designed using IBC-2015 criteria (see Step 1 above) Even though wind speeds for WDC-3 through WDC-5 SSCs are taken from ANSI/ANS-2.3-2011 (R2016), these are designed following ASCE/SEI 7-10 criteria, as provided in the Standard.

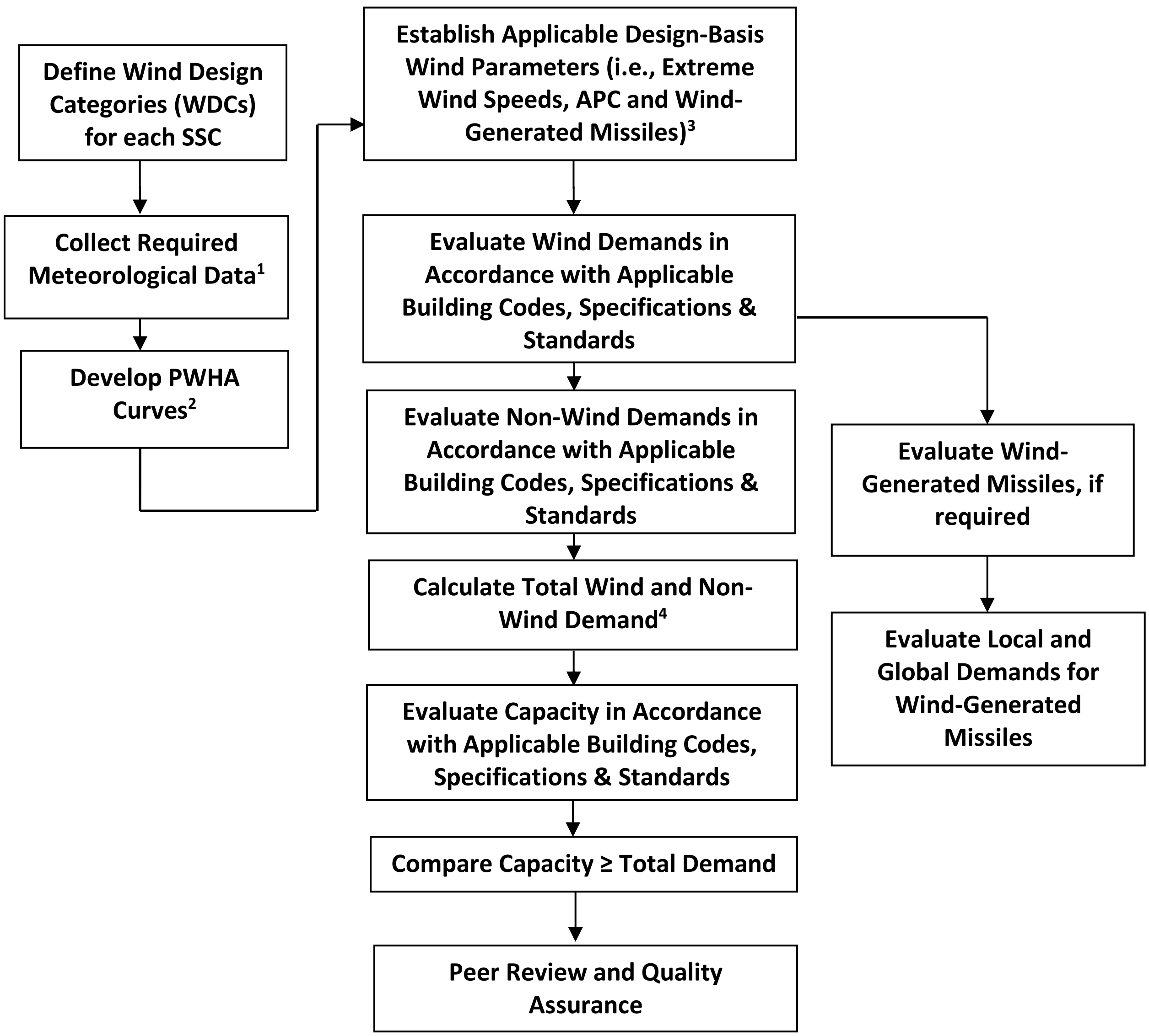

Figure 4-1. Wind Design and Evaluation Procedure

Notes:

1. Site-specific wind-related hazard data collection and the PWHA, as noted in Section 4.3 of the standard, should be based on the highest WDC of the SSCs at the DOE site.
2. For DOE sites that only have WDC-1 and WDC-2 SSCs, criteria given in IBC-2015 for Risk Categories II and IV, respectively, should be used.
3. For DOE sites that have WDC-3, WDC4 and WDC5 SSCs, ANSI/ANS-2.3-2011 (R2016) wind speeds may be used for developing the site-specific PWHA.
4. For WDC-3, WDC-4, and WDC-5 SSCs, nuclear industry codes ANSI/AISC N690-2012, *Specification for Safety-Related Steel Structures for Nuclear Facilities,* and ACI 349-13, *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary*, should be used for design basis load combination and acceptance criteria. However, ASCE/SEI 7-10 should still be used for methods of analysis, for example, conversion of wind speeds to loads on structures.

4.2 Site Characterization for Wind-Related Hazard Design

4.2.1 General Requirements

Wind design categorization is based on the severity of unmitigated failure consequences resulting from all types of extreme wind-related hazards. Table 2-1 of the Standard provides design categories for various levels of unmitigated failure consequences in DOE nuclear facilities. However, for SSCs in facilities other than DOE Hazard Category 1, 2, and 3 nuclear facilities, only facility risk categorization following IBC-2015 is required (see Sections 2.2 and 4.1.2 of the Standard).

Engineered barriers can be used to provide protection of systems and components from extreme wind forces and from wind-generated missiles, but the barriers will not lower the WDC of the protected systems and components. For example, a barrier could be used to protect a tank located outside a building structure from extreme wind forces and wind-generated missiles. A barrier of this kind would have to be designed at an appropriate WDC level commensurate with the WDC of the tank assuming that the tank was not protected by the barrier. If the barrier is designed to protect the tank from the effects of the extreme wind forces and wind-generated missiles, then the tank would not be required to be designed for the extreme wind forces and wind-generated missiles. In most cases, the building structure serves as an engineered barrier for the systems and components in the building.

4.3 Probabilistic Wind Hazard Assessment and Determination of Wind Design Parameters

4.3.2 Development of PWHA

Data Needs for PWHA: The extreme wind hazards defined in ANSI/ANS-2.3-2011 (R2016) are based on the regionalization of wind speeds for the continental United States (see Figure 1 of the ANSI/ANS standard). This regionalization results in the inherent smoothing of isotachs (lines of equal wind speed intensity). For a smaller-scale evaluation, especially when local topography and close proximity to large bodies of water may have significant effects on the regional winds structure, it may be prudent to perform a site-specific PWHA to account for these effects. Requirements and guidance on data needed for a site-specific PWHA can be found in:

- Section 4.2 of DOE-STD-1020-2016,
- ANSI/ANS-2.3-2011 (R2016),
- ASCE/SEI 7-10,

- Nuclear Regulatory Commission (NRC) Reg. Guide 1.76, *Design Basis Tornado for Nuclear Power Plants*,
- NRC Reg. Guide 1.221, *Design Basis Hurricane and Hurricane Missiles for Nuclear Power Plants*,
- NUREG/CR-4461, *Tornado Climatology of the Contiguous United States*,
- NUREG/CR-7005, *Technical Basis for Regulatory Guidance on Design-Basis Hurricane Wind Speeds for Nuclear Power Plants*, and
- UCRL-ID-140922, *Development of a Probabilistic Tornado Wind Hazard Model for the Continental United States*.

Design Basis Extreme Wind: The determination of a design basis extreme wind speed is based on historical wind statistics representative of the site. The longer the data period that is used, the better the chance that extreme climatology is embedded within. The winds used in the PWHA should be gleaned from National Weather Service (NWS) records, supplemented by meteorological data collected by the site's meteorological program, provided that all data relied on meet the quality assurance (QA) requirements of ANSI/ANS-3.11-2015, *Determining Meteorological Information at Nuclear Facilities*. A temporally representative database of maximum three-second gust wind speeds should be used if available.

Wind Measurement and Satellite Imagery: Straight-line and hurricane wind speeds are usually obtained from in situ measurements with bi-vane or sonic anemometers. Aircraft may be sent into hurricanes to determine a wind speed profile from the outer radius to the eye wall. Satellite imagery may be used to further define the three-dimensional wind field where in situ measurements are not available.

Anemometer Height: The anemometer height above the ground varies at monitoring locations and may also vary from time to time at the same location. For this reason, it may be necessary to extrapolate or interpolate the wind data to correspond to the standard measurement height of 33 feet (10 meters). ANSI/ANS-3.11-2015 provides a methodology for extrapolating wind data to the standard measurement height. For DOE sites that monitor wind speed at various heights on a meteorological tower and take wind speed measurements at many locations, the most spatially representative wind speed measurement should be used for analysis. A professional with sufficient education and training in atmospheric sciences should provide guidance on this topic.

Frame of Reference for Wind Speeds: Wind speeds should be cited within a consistent frame of reference. In ANSI/ANS-2.3-2011 (R2016) and ASCE/SEI 7-10, the frame of reference commonly used is "peak gust" wind speed, which is the speed of air passing over the instrument, averaged over a three-second period. Such measurements are commonly monitored at 33 feet above ground level in flat, open terrain. Wind speeds measured relative to one frame of reference can be converted to another frame of reference through the use of logarithmic wind speed profiles and various relationships between averaging times (see ASCE/SEI 7-10). Wind speeds are also affected by the roughness of the terrain, as this factor determines the magnitude of frictional forces on air movement in the planetary boundary layer (PBL) (See ASCE/SEI 7-10). However, logarithmic wind speed profiles or terrain roughness effects do not apply to tornadoes, which have a more complex wind field structure.

Converting Peak Gusts to Mean Wind Speeds: The Durst curve (Durst, 1960) has been developed for non-hurricane winds and the Krayer-Marshall curve (Krayer, 1992) has been established for hurricane winds. Both of these curves relate the peak wind speed and the hourly peak wind speed and both can be used to convert wind speeds to various averaging times. These curves can also be used for converting the hourly mean speed and the fastest-mile wind speed to the three-second gust wind speed, and vice-versa. These techniques do not apply to tornadoes.

Wind Loading: Winds associated with meteorological NPH other than frontal passages and derechos have both a translational component (i.e., movement from one geographic location to another) and a rotational component along the path of movement. The scale of tornadoes and Mesoscale Convective Vortices (MCVs) is much smaller than the massive scale of a hurricane; the diameter of the rotating winds in a small hurricane exceeds the diameter of a very large tornado. The significant damage path width of a tornado is usually less than one mile in diameter, the largest being about 2.6 miles in diameter. The rotational wind diameter of these extreme meteorological phenomena is large compared to the typical dimensions of a building or structure. Although tornadoes, hurricanes, and extreme straight-line winds are produced by distinctly different meteorological phenomena, building structure research has shown that their effects on SSCs are essentially the same with the exception of APC effects. Table 2 of ANSI/ANS-2.3-2011(R2016) provides APC magnitudes for various maximum tornado wind speeds. In accordance with Section 4.4.2 of the standard, tornado loading should account for the combined dynamic effects of these pressure changes and the lateral wind loadings.

Tornado Characteristics: Tornadoes can be characterized by (a) maximum total wind speed, (b) radius of maximum tangential wind speed, (c) tangential, vertical, radial, and translational wind speeds, and (d) associated APCs within the tornado wind field. In order to account for the effects of all of these variables, tornado hazard probability models should consider (a) gradations of velocity along and across the tornado path, and (b) biases in tornado occurrence reporting that can affect velocity estimation. Tornadoes usually overwhelm any pre-existing ambient meteorological conditions at a site. Topographic characteristics do have some effect on tornado formation and vertical extent, but present models cannot quantitatively account for them. Tornadoes rarely touch down in mountainous regions due to the extreme friction from rough terrain.

Tornado Wind Scale: In general, wind speeds in tornadoes cannot be measured by conventional anemometers due to destructive forces on such instruments. The method typically used to measure tornado wind speeds is the apparent mechanical damage within the storm path or other indirect parameters such as penetration of straw into trees or movement of an automobile from its point of origin. The Enhanced Fujita (EF) scale classification uses this method and is the accepted standard for estimating tornado wind speeds. (See http://glossary.ametsoc.org/wiki/ Enhanced_Fujita_Scale.)

APC Effects: In addition to wind effects, tornadoes produce APC effects which can result in facility explosions if confinement design does not compensate for the abrupt pressure changes. Since the APC affects only sealed structures, substantial breaches in a building caused by wind-generated missiles may prevent significant differential pressure loads. Openings of one square foot per 1,000 cubic feet volume are sufficiently large to permit equalization of inside and outside pressure as the tornado passes over the structure. Consideration should be given for some dynamic structural responses between the maximum external wind forces and the development of the APC internal forces. SSCs that are purposely sealed will experience the net pressure difference caused by an APC. The APC, when present, acts outwardly and combines with external negative (i.e., outward) wind pressures. The magnitude of the APC is a function of the tangential wind speed of the tornado.

Determining Local Wind Speeds: Site representative wind speeds may be determined by interpolating values found in NWS wind data. NWS data can be acquired from the National Climatic Data Center in

Asheville, NC. Whether such a method is valid depends on site topography, the proximity of the site to nearby water bodies, and local meteorological conditions.

Complex Terrain: For sites located in complex terrain such as a valley between mountains, physical modeling may be needed to relate wind data available at: existing wind stations to expected wind characteristics at the site.

Hurricane-Tornado Comparison: Although hurricanes and tornadoes are of different scales and origins, they have very high sustained wind speeds as a commonality. Both phenomena have maximum three-second gust wind speeds that can exceed 200 mph. In most regions, tornado wind speeds exceed hurricane wind speeds. (See Figures 2 through 4 in ANSI/ANS-2.3-2011(R2016)) However, hurricane missile speeds, as a function of wind speed, can exceed tornado missile speeds. Hurricane characteristics are much the same as for straight-line wind effects from derechos and frontal passages in that they do not include an APC component. Only tornadoes, due to the large rotational component of a tightly packed storm with relatively rapid translation, create an APC.

Missiles: Sustained three-second gust wind speeds in excess of 75 mph may generate missiles from unsecured objects and debris from building damage. Such low-mass missiles, even when driven by wind speeds of up to 110 mph, rarely cause structural damage to reinforced concrete and structural steel industrial-type facilities. The proper selection of wind-generated missiles for the PWHA is dependent on (a) intensity of the wind speed at the site, (b) types of missiles present and their location relative to the facilities, (c) the missile position relative to the projected wind path, and (d) physical properties (such as hardness) of the missiles. Two basic approaches in the characterization of missiles are generally recognized as acceptable: (a) a standard spectrum of missiles as defined in Table 4 of ANSI/ANS-2.3-2011 (R2016), and (b) a site-specific probabilistic assessment of the missile hazard. ANSI/ANS-2.3-2011 (R2016) should be used unless sufficient data is available to support a site-specific probabilistic assessment of the missile hazard.

4.4 SSC Design to Mitigate Wind-Related Hazards

4.4.1 General Design Criteria

The PWHA yields the wind speeds for various meteorological phenomena as a function of mean return period. In addition, wind-generated missiles are also required to be considered as part of the PWHA.

The methodologies for performing the PWHA for extreme straight-line wind speeds and hurricane wind speeds are defined in ASCE/SEI 7-10 and ANSI/ANS-2.3-2011 (R2016). Methodologies for performing the PWHA for tornadoes and various approaches to define tornado-generated missiles can be found in ANSI/ANS 2.3-2011, NRC Regulatory Guide 1.76, NUREG/CR-4461, and UCRL-ID-140922. ANSI/ANS-2.3-2011 (R2016) states that there are two acceptable methodologies for developing a family of tornado wind hazard curves: NUREG/CR-4461 and Boissonnade et al. If the methodology defined in Boissonnade et al. is used, adjustments should be made to account for the changeover from the Fujita F-scale to the Enhanced Fujita EF-scale.

4.4.2 Design of SSCs in Categories WDC-3, WDC-4, and WDC-5

As already stated, ANSI/ANS-2.3-2011 (R2016) is used to determine extreme straight-line, hurricane, and tornado wind speeds affecting WDC-3, WDC-4 and WDC-5 SSCs, unless local topography and local meteorological conditions require a site-specific PWHA. Determination and distribution of applied forces on SSCs in a facility are determined using the procedures described in Chapters 26-30 of ASCE/SEI 7-10.

Various degrees of conservatism are introduced in the design process by means of load combinations. The load combinations to be used for WDC-3, WDC-4 or WDC-5 SSCs are defined in Section 4.4.2 of the Standard, and are based on strength design, ANSI/AISC N690-2012, *Specification for Safety-Related Steel Structures for Nuclear Facilities*, and ACI 349-13. Criteria defined in IBC-2015 should be used for WDC-1 and WDC-2 SSCs.

The methodology in ASCE/SEI 7-10 is used for determining the wind pressures and net forces on an SSC as a function of wind speeds. The APC differential pressure on structures caused by a tornado is defined in Table 2 of ANSI/ANS-2.3-2011 (R2016). The outward load effects of the tornado APC on a building are exerted on its sides, roof, and leeward walls and windows.

Clarifications of the procedures and additional considerations for designing SSCs subjected to extreme wind hazards are provided in the paragraphs below.

1. **Wind Pressure Equations**. Sections 27.3.2 and 27.4.1 – 27.4.5 of ASCE/SEI 7-10 provide equations for determining the wind pressures using the design basis wind speeds and the WDC of the building structure. The equations in ASCE/SEI 7-10 can be used for all WDCs as long as the design basis wind speed for the applicable WDC category is used in the equations.[6] Chapter 28 of this document provides a simplified conservative procedure for low-rise buildings. Chapter 29 gives the equations for determining wind pressures on other structures and building appurtenances. Chapter 30 provides equations for the wind pressures on components and cladding.

2. **Mechanics of Wind Pressures on Structures**. Wind pressures on structures can be classified as external or internal. External pressures develop from aerodynamic effects such as incompressible air flows over and around enclosed structures. The air particles change speed and direction, producing a pressure field on the external surfaces of the structure. At sharp edges, the air particles separate from contact with the building surface (a phenomenon called flow separation), with an attendant energy loss. These particles produce outward-acting pressures near the location where the flow separation takes place. External pressures act outwardly on all surfaces of an enclosed structure except windward walls and steep windward roofs. Internal pressures develop when air flows into or out of an enclosed structure through existing openings or openings created by airborne missiles. Internal pressures act either inward or outward, depending on the location of the opening and the wind direction. If air flows into the structure through an opening in the windward wall, pressure inside the building increases relative to the outside pressure. This pressure change produces additional net outward-acting pressures on all interior surfaces. Openings in any wall or roof area where the external pressures are outward acting allow air to flow from inside the structure, which causes the pressure inside the structure to decrease relative to the outside pressure. The pressure change produces net inward-acting pressure on all interior surfaces. Internal pressures combine with external pressures acting on a structure's surface.

3. **Effects of Terrain**. The roughness of surrounding terrain significantly affects wind speed. Terrain roughness is typically defined in four classes: urban, suburban, open, and smooth. Each terrain roughness class has a roughness length associated with it, defined as the height where the wind

[6] Use of these equations is also subject to the "Conditions" stated in Section 27.1.2 and the "Limitations" stated in Section 27.1.3.

speed is effectively zero. For engineering purposes, wind speed profiles, as a function of height above ground, are represented by a logarithmic power law relationship. The wind speed increases with height, as the frictional effects of the earth diminish with height, reaching a maximum at the top of the PBL where the frictional effects of the earth's surface are no longer present and the wind speed at that level and above are termed geostrophic (arising from the rotation of the earth).

4. **APC and Maximum Winds**. The maximum tornado wind speed and the maximum APC do not occur at the same location within the tornado vortex. The lowest APC occurs at the center of the tornado vortex, whereas the maximum wind pressure occurs at the radius of the maximum wind, which ranges from 150 to 500 feet from the tornado center. Since the maximum APC pressure occurs at the center of the tornado vortex where the rotational wind speed is theoretically zero, a more severe loading condition occurs at the radius of maximum tornado wind speed, which is a finite distance from the vortex center. The APC is approximately one-half of its maximum value at the radius of maximum wind speed. With APC acting on a sealed building, the internal pressure need not be considered.

5. **Missiles**. Hurricane winds and tornado winds pick up and transport various pieces of debris, including roof gravel, pieces of sheet metal siding, girts, I-beams, timber planks, pipes, vehicles and other objects that have high surface to weight ratios. These objects can be carried to heights up to 200 feet and beyond in EF4 and EF5 tornadoes. Steel pipes, posts, lightweight beam sections and open web steel joists having smaller area-to-weight ratios can also be transported by hurricane and tornado winds, but this occurs less frequently and such missiles do not normally reach heights above 100 feet. Forces due to missiles and tornado/hurricane wind effects should be combined appropriately unless it can be justified as not necessary.

6. **Barrier Design for Missiles**. Section 4.4.2.3 of the Standard provides criteria for designing barriers to protect WDC-3, WDC-4, and WDC-5 SSCs against a tornado APC load or missiles generated by tornadoes and hurricanes. The barrier design practice for tornado missile and hurricane missile consists of (a) preventing excessive local damage (i.e., penetration, perforation, scabbing, and spalling), and (b) preventing failure of the barrier caused by its inability to withstand the absorbed energy without excessive bending and shear. Preventing excessive local damage by a missile requires either that (a) the wall or barrier is thick enough to prevent perforation and excessive penetration, scabbing, or spalling of the concrete, or (b) a properly designed plate is attached to the front and/or to the rear surface of the wall or barrier. Overall wall failure due to bending or shear is prevented by designing the wall to have reserve strain energy greater than the total absorbed energy to which it is subjected.

7. **Barrier Design for Pressure**. Barriers may also be provided to protect SSCs from wind pressure loadings. A typical example of such a barrier is a building enclosure consisting of walls and a roof designed to withstand extreme pressures caused by extreme straight-line, tornado, APC, or hurricane winds corresponding to the highest WDC of the SSCs inside the building.

5.0 CRITERIA AND GUIDELINES FOR FLOOD, SEICHE, AND TSUNAMI DESIGN

5.1 Flood Design Categorization

Section 5 of the Standard provides the criteria and guidelines for designing SSCs subjected to flood hazards that include hazards from seiche and tsunami. The five major steps in the design and evaluation process of SSCs subject to flood hazards are described and illustrated in Figure 5-1 below.

Step 1. Determine Flood Design Categories

As shown in Figure 5-1, the first step in flood design and evaluation is the determination of FDC by applying the provisions given in Sections 2.3 and 5.1 of the Standard.

Step 2. Perform Flood Parameter Site Characterization

The primary purpose of site characterization for flood hazards is to determine the elevation of the highest water level that can result at the site from the potential sources of flood. When a site-specific flood characterization is performed using Section 5.2 of the Standard, the activities should be integrated with characterization of localized flooding resulting from extreme precipitation.

Site characterization depends upon thorough investigation of off-site precipitation data and hydrologic characteristics of a site and its surroundings. For this purpose, off-site precipitation includes runoff indirectly affecting site conditions through downstream effects on the drainage capacities of stormwater structures. Site characterization is best facilitated by the development of a topographic map showing surface drainage patterns, contours, elevations, peak, and valleys. With such data in hand, drainage divides may be delineated and times of concentration and peak runoff discharges estimated.

Step 3. Determine Flood-Related Hazards

Typical adverse effects of a flood are: (i) SSC failure from water intrusion or submergence, and (ii) structural failure from hydrostatic and hydrodynamic loads. Hence, the elevation of the water level that can result from a flood is the most important flood design parameter. The highest water level that can result at the site is compared with the Design Basis Flood Level (DBFL) for the SSC.

Step 4. Perform Probabilistic Flood Hazard Assessment (PFHA)

In accordance to Section 5.4 of the Standard, to determine DBFL, a site-specific PFHA is required only for FDC-3 through FDC-5 SSCs; for FDC-1 and FDC-2 SSCs, DBFL is determined based on the return periods in Tables 5-2 and 5-3 of the standard. When a new facility is planned for construction at a DOE site with an existing PFHA, the PFHA used to design the facility will follow the Standard's requirements. If it does not conform, a new PFHA, or an update to an existing one, is normally undertaken. Guidance on performing PFHAs for the five most common flood initiators can be located in Appendix B, Sections B.5 through B.9.

Step 5. Design SSCs to Mitigate Flood-Related Hazards

Section 5.5 of the Standard provides design and evaluation criteria for SSCs to mitigate flood-related hazards.

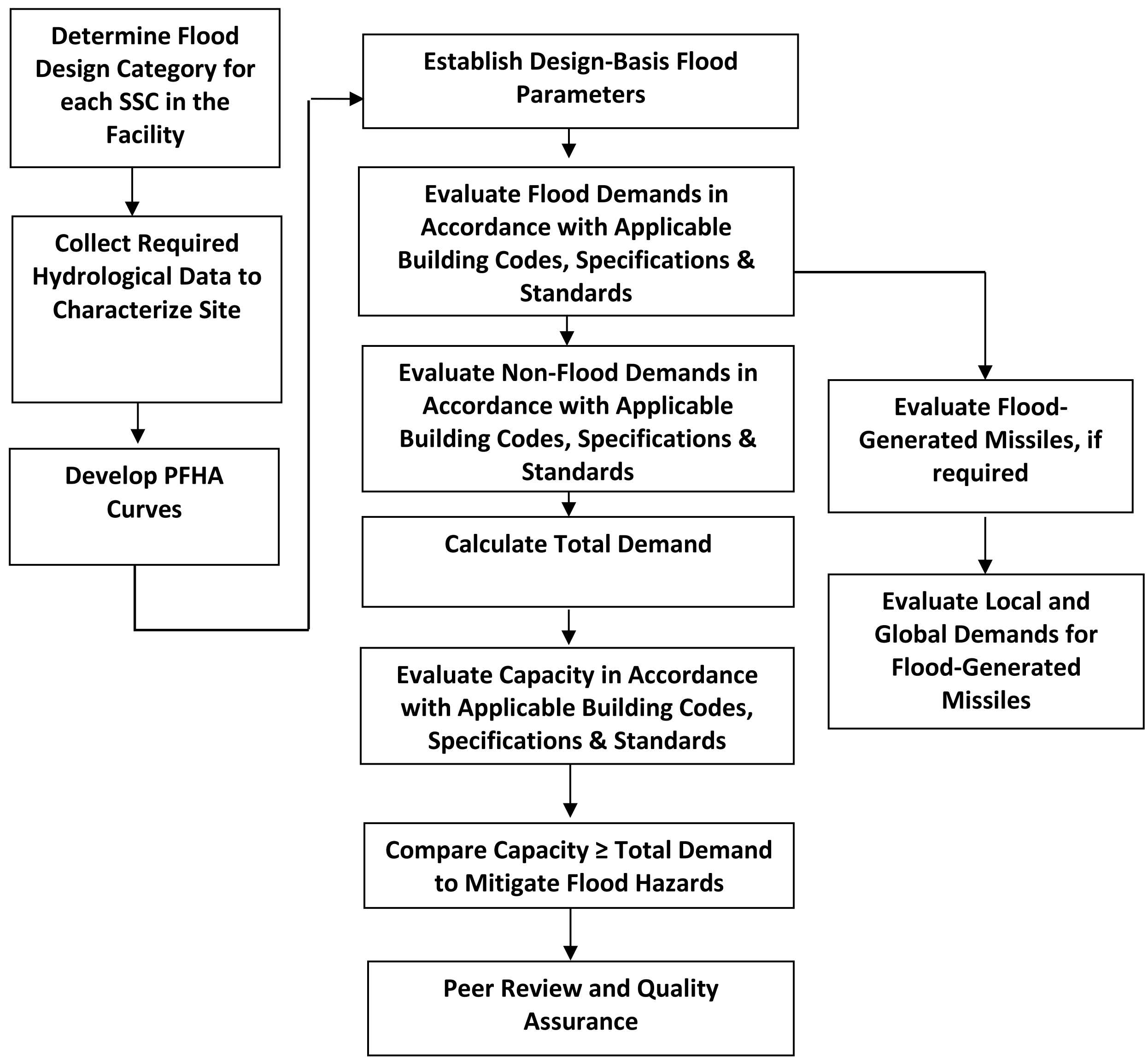

Figure 5-1. Flood Design and Evaluation Procedure

Flood design categorization is based on the severity of unmitigated consequences of SSC failure resulting from all applicable flood-related hazards listed in Table 5-1 of the Standard. Flood event combinations in Table 5-4 of the Standard should also be considered. Failure consequences are determined in terms of calculated radiation and chemical doses to co-located workers and the public from unmitigated releases. Table 2-1 of the Standard provides dose criteria for NPH-related SSC failures in nuclear facilities. When applying flood design methodologies, the following considerations should be kept in mind:

1. **SSCs Vulnerable to Submersion:** Certain safety-related electrical and mechanical SSCs may fail to perform their safety functions when partially or wholly submerged, splashed or sprayed during and after flooding events even though they may be capable of withstanding hydrodynamic and hydrostatic pressures or other mechanical loads resulting from the same events. SSCs with this failure mode should be identified and protected by locating these above the DBFL or by providing engineered controls. The DBFL for such SSCs is determined from DBRPs specified in Table 5-2 of the

Standard and the event combinations in Table 5-4 of the Standard. For other SSCs that are not vulnerable to submersion, the DBRPs are conservative.

2. **Common Cause Failure:** Like seismic events, flooding events often result in common cause failures as described in ANSI/ANS-2.26-2004 (R2010). In determining the FDC of an SSC, the potential common cause failure effects of multiple SSC failures from a single flood event should be considered. This recommendation applies to similar SSCs as well as to an entire facility. For example, assume that several SSCs are submerged in a flooding scenario, causing all of them to fail and the failure of only one SSC would result in a radioactive or toxic chemical dose above the dose threshold of FDC-2, but below the threshold of FDC-3. This makes the SSC a candidate for FDC-2 designation. However, if the same flooding scenario causes several of these SSCs to also fail, and the combined radioactive or toxic chemical dose resulting from the failure of these SSCs is above the dose threshold of FDC-3 but below the threshold of FDC-4, then these SSCs would be designated as FDC-3 and not FDC-2.

3. **Flood Protection Systems:** Flood protection systems should be designed to withstand the hydrostatic and hydrodynamic forces associated with the DBFL for the FDC of that SSC. Such a barrier, enclosure, or a dike should be high enough to provide standard-required freeboard above the DBFL to protect the SSC. These protection systems should be designed based on a DBFL determined using the appropriate return periods specified in Table 5-2 of the Standard.

4. **Mitigating Effects of Barriers:** In determining the FDC of a given SSC, the mitigating effects of a flood barrier cannot be used, even though credit for the barrier can be taken in determining the hydrostatic and hydrodynamic loads on the SSC.

5. **Multiple DBFLs for the Same Site:** As stated in Section 5.5.2 of the Standard, the DBFL for a facility or a site is the highest projected flood level, considering all the credible flooding sources for the site, and corresponding to the Design Basis Return Period (DBRP) for the SSC's flood design categorization. However, the intent of this provision is not to design all SSCs, irrespective of their FDCs, to the DBFL corresponding to the highest FDC of the SSCs. Instead, SSCs may be designed using DBFLs appropriate for the FDCs. For example, if a facility consists of two groups of SSCs, one in FDC-2 and the other in FDC-3, these two groups of SSCs will be designed to withstand two different DBFLs, one corresponding to a FDC-2 DBRP and the other corresponding to a FDC-3 DBRP.

6. **Large Site with Varying Topography:** The design basis flood may vary from facility to facility. For example, if a stream runs through a site, an FDC -3 facility located upstream at a higher elevation might have a different design basis flood than an FDC-3 facility located further downstream.

7. **Building FDC versus FDCs of SSCs Inside:** The building FDC can be the same or different from FDCs of SSCs inside the building. In determining the FDCs of the building and the SSCs inside, the adverse interaction effects of failure among these, and the effect of common cause failures resulting from the design basis flooding events, should be considered. Typically, the FDC of the building is the same as the SSC with the highest FDC, because it is very difficult to demonstrate that the building failure would not adversely affect an SSC inside. However, the DBFL for the building and that for SSCs inside the building may not be the same because the building's failure mode may be structural while the SSCs may fail from water intrusion.

5.2 Site Characterization for Flood-Related Design

When performing site characterization for flood design, the following additional considerations are important:

- **PFHA Hydrologic Data Needs:** A technically sound site-specific PFHA rests upon collection of regional-scale and local-scale hydrology data to support evaluation of each source of flooding and characterization of the site. The data collection effort should include a comprehensive literature search. Technical experts from any of the various federal organizations responsible for evaluating flood hazards should be consulted. These organizations include the National Oceanic and Atmospheric Administration (NOAA), Federal Emergency Management Agency (FEMA), Bureau of Reclamation (USBR), U.S. Army Corps of Engineers (USACE), U.S. Geological Survey (USGS), U.S. Department of Agriculture, NWS, and Tennessee Valley Authority. Flood hazard publications by the NRC, such as NUREG/CR-7046, *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America*, may also be consulted.

- **Additional PFHA Data Sources:** For applicable hydrologic hazards, certain additional data should be used in the performance of a PFHA:

  - Walkdown of site and vicinity;
  - Site-specific and regional topographic maps;
  - Aerial photographs of the site and vicinity;
  - Hydrologic data (e.g., stream gauge data);
  - Historical flood event reports including paleoflood information;
  - FEMA flood insurance studies;
  - Elevation component of the Federal Flood Risk Management Standard;
  - Dam-break studies; and
  - Local and regional flood history, to include potential causes of flooding under extreme conditions and date, level, peak discharge, and other relevant information for every recorded flood event.

- **PFHA Literature Search**: Depending on the complexity of the flood hazards to be evaluated, the literature search may involve gathering of information on alternative hydrological or meteorological modeling approaches and data interpretations. This information can be important input when estimating uncertainties.

- **Flood Combinations:** Table 5-1 of the Standard lists the flood sources to be considered for site characterization. The highest water level that can result from each of these sources, and from a combination of some of these sources, is then determined. Table 5-4 of the Standard lists the event combinations to be considered.

- **Topographic Effects**: For FDC-1 and FDC-2 SSCs, when flooding in the vicinity of a specific facility is subject to small-scale variations in local topography (such as channeling of flood waters into a narrow valley), it may be prudent to revise the results of a regional flood hazard estimate to incorporate site-specific topographic effects. Alternatively, a site-specific PFHA that includes more-detailed hydraulic and hydrologic analyses should be performed. These analyses should account for the flow disturbance caused by local topography and other site-specific conditions.

- **DBFL and Multiple Flood Hazards**: For sites located on rivers or streams, the meteorological and hydrologic events that produce intense local precipitation are often distinct from those which produce high river flows. In this instance, various aspects of the design for an SSC should be determined by different flood hazards. As a result, the term DBFL is used in a general sense that applies to the multiple flood hazards that may be included in the design basis.

Appendix B of this handbook offers information on the probabilistic and phenomenological characteristics of flooding hazards from river flooding, dam failure, storm surge, seiche, and tsunami.

5.4 Probabilistic Flood Hazard Assessment and Determination of Flood Design Parameters

In the Standard, return periods in Table 5-2 are higher than those in Table 5-3 because they apply to SSCs assumed to fail unconditionally due to submergence or water intrusion. For these SSCs, it is not possible to provide margin in the flood design of an SSC for the flood levels from stream and river flooding—the SSCs are either above the flood level or they are not. For these SSCs, the annual probability of functional failure is the same as the annual probability of hazard occurrence. However, properly designed structural components do not fail unconditionally due to submergence or water intrusion because of built-in structural design conservatism. For this reason, the return periods in Table 5-2 should be used to determine DBFLs for establishing building elevations, heights of protective barriers, and heights of waterproofing, while the shorter return periods in Table 5-3 should be used to determine design basis flood loads on building structural walls and protective barriers.

There are two main objectives in preparing a PFHA for any FDC:

- Evaluation: The consideration of the complete set of data, models, and methods proposed by the larger technical community that are relevant to the hazard analysis; and

- Integration: The representation of the center, body, and range of technically-defensible interpretations in light of the evaluation process, informed by the assessments of existing data, models, and methods.

These objectives apply at all levels of analysis. However, not all facilities or sites require the same level of probabilistic analysis. This grading concept is similar to that used in PSHAs, in which four levels of analysis are defined.

The following factors should be considered when determining the level of analysis to be used:

1. The highest FDC of SSCs in the subject facility,

2. The level of technical complexity and/or uncertainty associated with one or more elements of the PFHA (e.g., meteorological, hydrological, hydraulic, local conditions), and

3. For sites with controversial technical issues, the degree of defensibility of the PFHA required to support the analytical results.

Table 5-1 below shows the recommended levels of analysis as a function of these factors, while Table 5-2 provides a summary of each level of analysis. Table 5-2 has been developed from a similar table in ANSI/ANS-2.29-2008 (R2016). As part of the process for developing and implementing a PFHA, the analyst should discuss with the SME the failure modes and vulnerability of SSCs to inundation, spray or wave effects, hydrostatic and hydrodynamic loads, and debris impact. The PFHA results are used for the determination of the DBFL and its characterization for use in the design criteria. Appendix B of this handbook provides a discussion of the background and elements for conducting a site-specific PFHA.

Table 5-1. Guidance for Selection of PFHA Level

Highest FDC of SSC*	Flood Hazard Complexity or Level of Controversy	Recommended Minimum PFHA Level	Recommended Minimum PFHA Level for Existing Facilities**
FDC-5	High	3 or 4	3
	Low	3 or 4	2
FDC-4	High	3 or 4	3
	Low	3 or 4	2
FDC-3	High	3	2
	Low	2	1

Notes:

***** PFHA is not mandatory for sites and facilities with no SSCs higher than FDC-2.
****** For existing facilities where an earlier PFHA conforming to new facility PFHA level requirements is available.

Table 5-2. General Process Description of Various PFHA Levels

PFHA level	Approach	General Process Description
1	TI (Technical Integrator—Use available Information)	Information available in the technical literature and data supporting development of national flood hazard maps are used. Other resources such as site-specific flood source characterizations, flood screening analyses, and PFHAs performed as part of a previous flood hazard study, can be used. The responsibility for the use of existing resources lies with the PFHA analyst, who should judge their adequacy with respect to DOE-STD-1020-2016 requirements. As required by DOE-STD-1020-2016, or as necessary to update databases, updates or modifications of an existing flood study are performed in a manner consistent with DOE-STD-1020-2016. Internal peer review of the process and the technical evaluation is required.
2	TI (Technical Integrator—Interaction with experts)	Same process as PFHA Level 1. In addition, the analyst interacts with SMEs and proponents of alternative scientific interpretations. Participatory Peer Review Panel (PPRP) is required.
3	TI (Technical integrator—Experts are brought together to debate and interact on technical issues)	Same process as PFHA Level 2. In addition, resource experts and proponents participate in workshops with the project team to discuss the available data sets, scientific and modeling issues, and sources of aleatory and epistemic uncertainty. These workshops are designed to support the technical integrator's evaluation of uncertainties and modeling of flood hazards and the final integration into a composite distribution. PPRP is required.
4	TFI (Technical Facilitator Integrator)	Same process as PFHA Level 3. In addition, involves formal elicitation of input from teams of evaluation experts. The teams participate in workshops coordinated by the TFI. PPRP is required.

5.5 SSC Design and Evaluation to Mitigate Flood-Related Hazards

When applying the Standard's flood-related requirements and guidelines, the following considerations should be kept in mind:

- **Options for Flood Hazard Mitigation:** Mitigation of flood hazards may be achieved using a variety of alternative design options. For instance, an attractive design alternative for new facilities is to choose a site at an elevation above the DBFL or outside of the flood hazard zone. Separate protective structures such as levees, floodwalls, drainage systems, and diversions can be designed and constructed to protect an entire site or individual facilities. However, in some circumstances these options may be unavailable. For these situations, SSCs should have adequate strength to withstand hydrostatic and hydrodynamic forces resulting from floods and be protected from debilitating water intrusion by, waterproofing or other means.

- **Barriers for Flood Protection:** A common mechanism to protect sites and/or an individual facility or SSCs from the effects of flooding is to provide flood protection systems such as floodwalls, levees, dams, detention basins, and temporary barriers such as inflatable dams, stop-logs, and sandbags. Often, these engineered flood mitigation controls are coupled with administrative controls, which might be emergency procedures to monitor developing events or defined set points when temporary barriers need to be installed. When flood protection systems are used to mitigate the effects of floods, their design should provide sufficient margin, taking into account inherent uncertainties in the performance of such systems.

- **Design Basis Flood Levels**: The DBFL for an SSC is based on its FDC. DBFL is expressed in terms of elevation from mean sea level. Hence, two SSCs having the same FDCs and exposed to the same sources of flooding, but located at different elevations, can have different DBFLs.

- **Locating SSCs Above DBFL:** Though it is not mandatory to locate an entire facility above the DBFL, all SSCs in a facility that may lose their safety function when submerged should be positioned above the DBFL if effective mitigation of the effects on the SSCs cannot be achieved. Alternatively, these SSCs can be protected such that their safety function can be maintained during and after submergence. For a site with many SSCs in FDC-3, FDC-4 and FDC-5 categories, it is highly desirable that the site be located above the DBFL.

- **Determining Flood Forces:** The flood forces on all SSCs should be determined using Chapter 5 of ASCE/SEI 7-10.

- **Combining Hydrostatic and Hydrodynamic Pressures with Flood-Borne Missile Impact Loads:** Floods pick up and transport objects that have high surface to weight ratios. Hydrostatic and hydrodynamic floodwater pressures and missile impact loads can occur simultaneously, and should be combined while applying the load combination in Section 5.5.1.2 and 5.5.1.3 of the Standard. The load combinations for flood loads in Section 5.5.1.2 of the Standard are based on the load combinations in ACI 349-06 and ANSI/AISC N690-2012 for extreme environmental loads.

- **Flow Complexities Associated with Local Terrain and Topography:** The site-specific PFHA for any of the applicable 13 types of potential flooding sources listed in Table 5-1 of the Standard needs to consider the water body flow complexities associated with the effects of the site-specific local terrain and topography. Techniques used to address complex site-specific effects in PFHAs should include all types of complex flow mechanisms.

- **Precipitation-Based Flood Level:** Section 7 of the Standard addresses SSC design to mitigate flooding caused by extreme precipitation. A comparison of the flood level resulting from a Precipitation-Based Flood Level should be conducted, and the higher of the two levels used for design purposes.

- **Use of SMEs:** The appropriate methodology for a flood hazard caused by a combination of NPHs should be selected by an SME.

6.0 CRITERIA AND GUIDELINES FOR LIGHTNING DESIGN

For lightning hazards, SSC categorization is not necessary. Instead, the preferred approach is to identify the SSCs whose safety function may be adversely affected by lightning hazards. This approach yields two categories of SSCs, one for which lightning protection is required, and the other for which lightning protection is optional. NFPA 780-2017, *Standard for the Installation of Lightning Protection Systems*, can be used in identifying the SSCs that would need lightning protection.

Section 3.2.1 of Chapter X in DOE STD-1212-2012*, Explosives Safety*, contains a recommendation that visual inspections of explosives facilities should account for seasonal variations in weather. At some sites, snow, wind, and ice during the winter can damage lightning protection systems. At these sites, inspections should be performed in early spring, prior to the start of the peak electrical storm season. Some sites in California should conduct visual inspections prior to the autumn-winter storm season. Lightning protection systems should always be inspected if a witness has seen a lightning strike to a facility with a safety-related SSC, or there is observed damage to any lightning protection component, especially surge protectors.

Consideration should be given to providing lightning protection for low-hazard conventional facilities such as laboratories and office buildings. Special attention should be paid to bus shelters, guard stations, or other structures where personnel may seek shelter from severe weather conditions.

<u>Additional Resources</u>

- "Mitigating Lightning Hazards," Lawrence Livermore National Laboratory, *Science and Technology Review*, May 1996 (https://www.llnl.gov/str/pdfs/05_96.pdf).

- The DOE Explosives Safety Committee. (See Appendix B of DOE-STD-1212-2012.)

7.0 CRITERIA AND GUIDELINES FOR PRECIPITATION DESIGN

7.1 Precipitation Design Categorization

Section 7 of the Standard provides criteria and guidelines for designing SSCs subject to extreme precipitation. The five major steps in the design and evaluation process of SSCs subject to precipitation hazards are described and illustrated in Figure 7-1 below.

7.1.1 General Approach

Step 1. Determine Precipitation Design Categories

As shown in Figure 7-1, the first step in precipitation design and evaluation is the determination of PDC by applying the provisions given in Sections 2.3 and 7.1 of the Standard. Accordingly, SSCs are placed in PDC-1 through PDC-5 categories based on the unmitigated consequences of SSC failure resulting from flood. For DOE nuclear facilities, the dose criteria in Table 2-1 of the Standard are to be applied.

Step 2. Perform Precipitation Parameter Characterization

Section 7.2 of the Standard provides requirements and guidance for performing site precipitation characterization while Section 7.3 provides criteria for determining precipitation design parameters for precipitation hazards. The primary purpose of site characterization for precipitation hazards is to determine the elevation of the highest water level that can result from precipitation. Localized flooding from extreme precipitation should be integrated with the regional flood hazard studies. The size of the region to be investigated and type of data pertinent to the investigations are determined by the nature of the region. In essence, an overall determination of the watershed area and coverage characteristics draining to the site along with the identification of any major rivers, streams, tributaries will help in calculating peak runoff flows and velocities to the site. Concentrated flows and calculated times of concentration should be determined for the proper sizing of engineered stormwater hydraulic conveyance structures and components.

Step 3. Perform Probabilistic Precipitation Hazard Assessment (PPHA)

In accordance with Section 7.4 of the Standard, a PPHA is required. PPHA results are presented in the form of a hazard curve correlating precipitation levels and return periods from which a design basis precipitation level (DBPL) is selected for each PDC. For sites which only have PDC- 1 and PDC-2 SSCs, sufficient precipitation hazard data may exist in the published documents, as discussed in Section 7.2.2 of the Standard, to use for the site PPHA.

Step 4. Design SSCs to Mitigate Precipitation-Related Hazards

Typical adverse effects of precipitation are: (1) SSC failure from water intrusion or submergence, and (2) structural failure from hydrostatic and hydrodynamic loads resulting from localized flooding, including ponding on the roof. Hence, the elevation of the water level that can result from a localized flood is an important precipitation design parameter. The highest water level that can result at the site from precipitation, determined using criteria given in Section 7 of the Standard, is compared with the DBFL for the SSC determined using criteria given in Section 5 of the Standard, and the higher elevation is used as the design basis. Section 7.5 of the Standard provides design and evaluation criteria for SSCs to mitigate precipitation hazards.

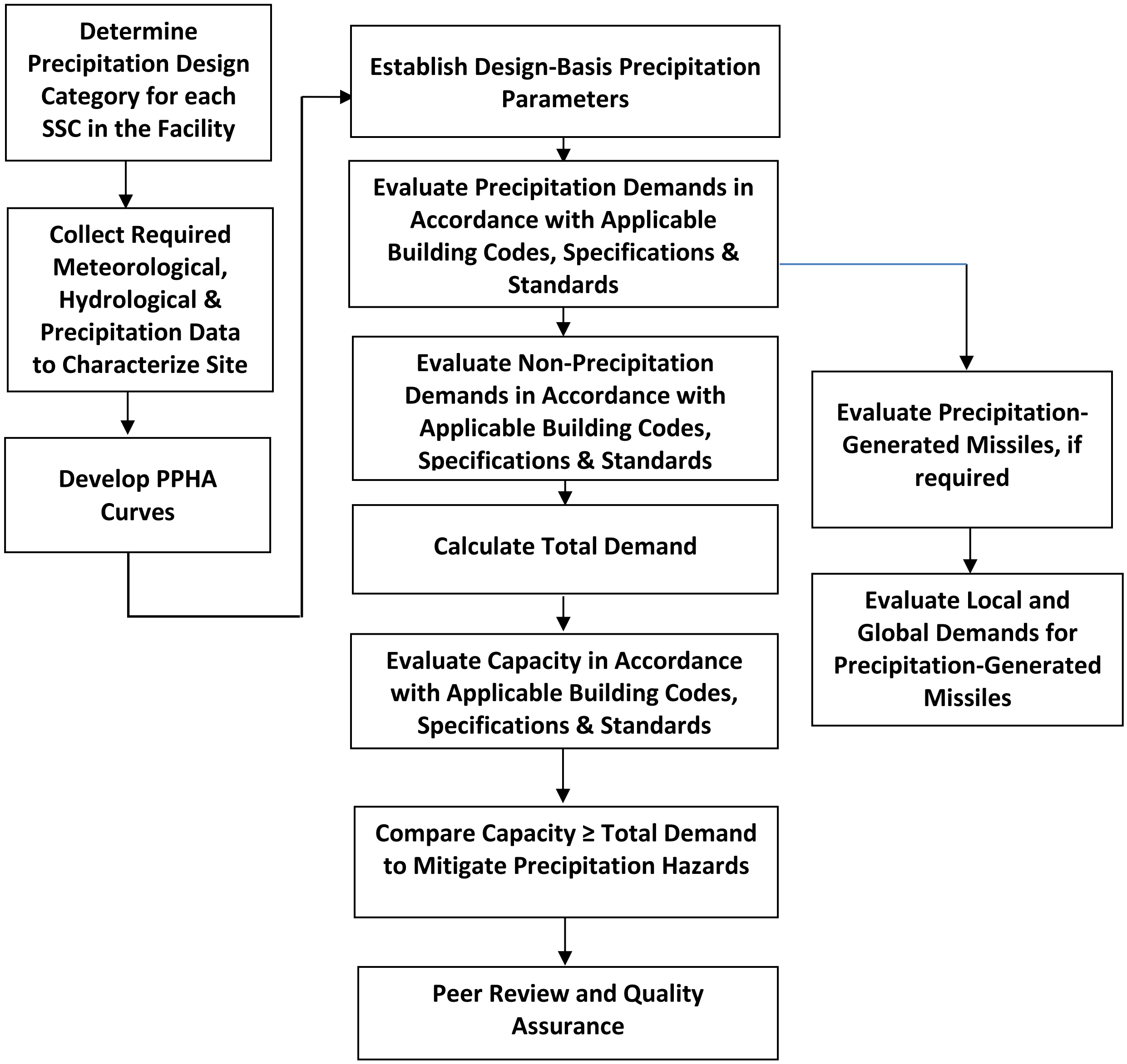

Figure 7-1. Precipitation Design and Evaluation Procedure

Clarification and guidance for implementing the categorization requirements of Section 7.1 of the Standard are provided in the paragraphs below:

1. **Follow Flood Guidelines.** The categorization of SSCs for extreme, local, onsite precipitation hazards should follow the same guidelines identified for the flood hazard categorization (i.e., the FDCs) in Section 5.1 of the Standard.

2. **PDC = FDC.** In some cases, the PDC and FDC for the SSCs may be the same. For example, if the flood hazard at a site is influenced by precipitation in the watersheds where the site is located, the FDC and PDC of the SSC categorization process are likely to produce similar results.

3. **PDC > FDC.** Some SSCs at a site could be located such that their FDCs do not affect their design, but the onsite precipitation hazard, by itself, could have an effect. In these cases, the PDC could be higher than the FDC.

4. **Regional/Local Effects.** Because regional and site-specific considerations can affect the outcome of SSC categorization for precipitation design, the analyst should consider both regional and local effects of precipitation.

7.2 Site Characterization for Precipitation-Related Design

Clarification and guidance for implementing the site characterization requirements of Section 7.2 of the Standard are provided in the paragraphs below:

1. **Site Characterization.** To start the site characterization process, land area and site boundaries need to be defined. A detailed description of the site topography and the surrounding area should be prepared to accurately define the watersheds that contribute to the local site flooding due to extreme precipitation. The location of the site with respect to nearby streams, lakes, or other significant water bodies needs to be considered to determine the relationship between flooding caused by regional precipitation and flooding caused by localized, onsite precipitation. The site grade and other pertinent elevations of facility structures and openings in the structures also need to be defined to assess effects of flooding caused by runoff from nearby watersheds.

2. **Storm Drainage Sewer System.** Underground storm sewers for DOE sites should be included in the site characterization.

3. **Changes to the Site Footprint.** Civil engineering changes to site grading generally result when a facility footprint is modified or a new facility is constructed nearby. These changes are influenced by permits issued for the National Pollutant Discharge Elimination System. These permits contain requirements for prevention of pollution from storm water runoff. Such requirements affect the grading around the new facility and the erosion and sedimentation controls (such as rock check dams and detention ponds) installed to prevent undesirable effects of storm water runoff. In these situations, communication with the site’s environmental safety and health organization would be helpful. Anticipated civil engineering changes to the site grade characteristics over the design life of the facilities should be considered to the extent possible. Ultimate buildout of a site should be considered, taking into account the maximum impervious area that could adversely affect site drainage characteristics and site flooding conditions.

4. **Topography.** The topographical characteristics of the site and the physical design of the facilities should be described in sufficient detail to allow a rational determination of final, post-construction site grade elevations.

7.3 Determination of Precipitation Design Parameters for Precipitation-Related Hazards

Section 7.3 of the Standard provides requirements and guidance for determining the design parameters for facility SSCs to mitigate precipitation hazards. Two types of hazards may result from onsite precipitation: site flooding and building roof ponding.

When the precipitation exceeds site drainage capacity or infiltration rate, the water level at the site rises. This submerges SSCs and causes buoyancy and pressure loads on submerged, buried, and partially buried structures. The section also identifies the data collection requirements for the characterization of site flooding-related hazards. Ten types of data will assist in the characterization process:

a. Site precipitation data; rainfall intensities
b. Soil infiltration capacity and indices including Manning's coefficients;
c. SCS soil types, runoff coefficients;
d. Site drainage capacity;
e. Local topographical characteristics of drainage areas;
f. Local topographic characteristics including peaks, valleys, depressions, slope;
g. Site storm water drainage data;
h. Data on site-located dams, levees, dikes, and bodies of water;
i. Geotechnical investigations; data on ground water table, regional and local aquifers and their sources; and
j. Local well log records.

7.4 Probabilistic Precipitation Hazard Assessment and Determination of Precipitation Design Parameters

- **PDC-1 and PDC-2 SSCs.** The DBPL for facilities with only PDC-1 and PDC-2 SSCs can be established based on Table 7-1 and Table 7-2 of the Standard.
- **PDC-3 through PDC-5 SSCs.** For determining the DBPL for facilities with PDC-3 through PDC-5 SSCs, it is necessary to perform a site-specific PPHA. The results of a site-specific PPHA are presented as a plot of return periods of precipitation events versus the precipitation levels of the corresponding events.
- **Return Periods, Table 7-1.** The return periods in Table 7-1 of the Standard are used to define the design basis precipitation flooding levels caused by precipitation runoff. The only exception is that for PDC-4, FDC-4, and WDC-4 SSCs, the DBRPs were selected such that these SSCs achieve the same design PG as the seismic PG for SDC-4 SSCs.
- **Return Periods, Table 7-2.** The return periods in Table 7-2 of the Standard are also used to define the design basis precipitation structural loads resulting from precipitation. Structures affected by such loads include roofs, dikes, protective structures, exterior walls, and doors. These return periods are smaller than the return periods in Table 7-1 of the Standard because since the design process for extreme precipitation loads intentionally provides structural design margin.

7.5 SSC Design and Evaluation to Mitigate Precipitation-Related Hazards

Roof ponding hazards may arise (a) when a roof is designed with a parapet and (b) when the roof drainage capacity is below the precipitation rate. In performing the analysis, one should assume a precipitation duration and the failure of primary roof drains. The roof may fail as a result of excessive loading due to water ponding or, in cold climates, by the accumulation of ice or snow. The following seven types of data will assist in the characterization process: (a) roof area, (b) roof slope, (c) parapet height, (d) roof drain sizes, (e) roof dead load, (f) roof live load, and (g) roof structural data. There also may be unique scenarios to be considered where off-normal conditions inhibit roof drainage for extended periods of time (e.g., during volcanic ash fall event, or ice storm).

Varying degrees of conservatism are introduced in the design process by means of load combinations. The load combinations to be used for PDC-3, PDC-4 or PDC-5 SSCs are defined in Section 7.5.4.3 of the Standard, and are based on strength design, ANSI/AISC N690-2012 and ACI 349-13. Load combinations defined in IBC-2015 for Risk Category II and IV should be used for PDC-1 and PDC-2 SSCs.

Section 7.5.4.3 of the Standard discusses the load combinations for DBPL extreme precipitation loads on building structure roofs, where R and S are defined as the extreme rainfall and snow loads. Section 9.2.7 of ACI 349-13 contains load provisions for extreme flood that may be used to represent extreme rainfall and snow loads. Section NB2-5d(3) of ANSI/AISC N690-2012 treats a fluid load as a dead load which, combined with an extreme environmental load combination NB2-7 (Wt=0) of ANSI/AISC N690-2012 may be used to represent extreme rainfall and snow loads. Note that if live load opposes the effects of rain or snow loading then a case without live load should also be considered. Roofs with parapets should have scuppers to prevent significant water accumulation if the primary drainage becomes blocked. Note that a range of rainfall durations should be considered and that a high intensity, short duration rainfall event may govern roof scupper design.

Underground storm sewers for DOE sites should be designed using a mean precipitation return period, considering the relative proportions of runoff to be carried by the sewers and the surface runoff. Proposed ultimate buildout of the site should be considered factoring in existing and proposed impervious and pervious areas. The DBPL should be determined accordingly, accounting for the capacity of the sewers. The analyst may choose to evaluate the site storm water management system for the highest category DBPL as a limiting case. If the results of this analysis demonstrate that flooding does not compromise the site SSCs, then it may be concluded that the site storm water management system is adequate. Local flooding in streets and parking lots may occur due to DBPL precipitation. This is acceptable if the effect of local flooding does not exceed the design requirements. However, if flooding does have an unacceptable impact, increased drainage capacity and/or flood protection should be considered.

8.0 CRITERIA AND GUIDELINES FOR VOLCANIC ERUPTION DESIGN

Section 8 of the Standard discusses characterization of volcanic hazards that might impact DOE facilities, as well as design considerations to mitigate certain hazards. The local hazards commonly associated with volcanoes, such as lava flows, mudflows, pyroclastic flows, ballistic projections, and asphyxiating gases, are nearly impossible to fortify against. Such hazards can only be avoided by siting facilities at a suitable distance from potentially active volcanoes.

Volcanic ashfall and accompanying lightning, as well as volcanic gases, can affect facilities hundreds of kilometers downwind of a volcano. Several DOE facilities are within this range of potentially active volcanoes and thus require some degree of volcanic hazard analysis (VHA). Ground motion associated with volcanic eruptions need not be considered as part of a VHA, as the hazard from such ground motion should be considered as part of the seismic hazard characterization.

8.1 Applicable Sites

Volcanoes pose a hazard to DOE facilities only in the Western U.S. (see Section 8.1 of the Standard). The Standard states that volcanic hazards are to be assessed at DOE sites and facilities lying within 400 kilometers (km) - approximately 250 miles—of a volcanic center that erupted within the Quaternary Period, defined as the last 2.6 million years. All DOE facilities west of the 105° meridian, a common demarcation of the eastern boundary of the Western U.S., lie within 400 km of a Quaternary volcano. The Pantex Plant in northern Texas also lies within 400 km of several volcanic centers in New Mexico. Facilities more than 400 km from a Quaternary volcano need not characterize the volcanic hazard.

The criterion of 400 km from a volcano is established based on the distance of significant ashfall from a large eruption. Beyond 400 km, the air concentration of volcanic ash from a large eruption is likely to be bounded by that of a major dust storm. Moreover, the ash accumulation at such a distance will be low enough that any structural loads imposed on facility roofs will be bounded by snow loads or other transient roof loads. The 400 km cut-off is based on worldwide eruption data compiled by Newhall and Hoblitt. This work derives probabilistic ashfall thicknesses for volcanoes of varying volcanic explosivity index (VEI). The VEI was first defined by Newhall and Self, *The Volcanic Explosivity Index (VEI): An Estimate of Explosive Magnitude for Historical Volcanism* (Newhall and Self, 1982) and it ranges from 0-8. High VEIs are more explosive, voluminous eruptions, and they are less common than low VEI eruptions.

The more recent work *Constructing Event Trees for Volcanic Crises* (Newhall and Hoblitt, 2002) provides several tables of expected ashfall thicknesses for eruptions of various VEI. For the most explosive eruptions, those ranging from VEI 4-8, the median ashfall thickness 200 km from the vent is 6.8 cm. The expected thickness would be approximately half this value at 400 km. Most eruptions in the Western U.S. are VEI 4 or lower, so these thicknesses are conservative estimates. Therefore, roof loads from ash accumulation 400 km from a VEI 4 eruption would be bounded by other precipitation loads. In addition, airborne ash 400 km from a VEI 4 eruption would likely have effects no worse than those of a dust storm. The analysis in the Newhall and Hoblitt paper is the basis for excluding volcanoes more than 400 km from a facility from a VHA.

The Quaternary Period is a convenient boundary on the geologic time scale at which to limit consideration of volcanoes. The 2012 version of the geologic timescale published by the Geological Society of America (Geological Society of America, 2012) places the beginning of the Quaternary Period at 2.6 million years before present. A volcano that has shown no activity during the Quaternary Period is very unlikely to pose a hazard to any facility over the time period of concern for DOE facilities, which is assumed to be no more than 100 years. A time window of 2.6 million years also ensures consideration of events with an annual recurrence frequency of 1E-6, the lower frequency bound of extremely unlikely events that should be considered in the accident analyses. Given that the period of concern is approximately 100 years, the Standard's volcanic hazard characterization criteria cannot be used to evaluate longer-term facilities such as a geologic repository.

8.2 Volcanic Hazard Assessment

Volcanic hazards should be assessed using a graded approach. The level of effort required for a VHA should vary depending on the proximity of volcanoes to a site, the age and frequency of eruptions from the volcanoes, the vulnerability of facilities to the effects of volcanic eruptions, and the hazards posed to the public if the facilities are affected by an eruption [i.e., Volcanic Design Category (VDC)].

Section 8.2 of the Standard lists the volcanic hazards to be considered in a VHA. Most sites are far enough from Quaternary volcanoes (more than 100 km/62.1 miles) that only hazards from ashfall, lightning, and gases need to be considered. Local volcanic hazards, such as lava flows, pyroclastic flows, mud flows (lahars), ballistic projections, and asphyxiating gases, are unlikely to spread beyond 100 km (62.1 miles) unless an eruption has magnitude above VEI 6. Such catastrophic eruptions are not only rare, they occur only after years or decades of geologic indicators leading up to an eruption. Therefore, eruptions above VEI 6 need not be considered in a VHA for facilities in the Western U.S.

Facilities generally cannot be fortified against the more local volcanic hazards, but if the hazards exist, they should be assessed and discussed in a VHA. A facility potentially subject to the proximal hazards would be expected to have a facility shutdown and evacuation plan to address the eruption hazards for which mitigation is not feasible, such as large lava flows, lahars, pyroclastic flows, and ballistic projections.

The Quaternary Period is divided into the Holocene Epoch, the time between present and approximately 10,000 years ago, and the Pleistocene Epoch, from 10,000 years until 2.6 million years ago. Volcanoes that were active only in the Pleistocene Epoch may be less likely to erupt over the coming decades, thus posing a lower hazard and requiring less investigation. Volcanoes that have been active during the Holocene Epoch require more detailed characterization.

Sites or facilities with SSCs categorized as VDC-1 or VDC-2 only do not require an extensive VHA. Such an assessment may be limited to a tabulation of Quaternary volcanoes within 400 km, their distances from the site, the best available knowledge of their eruption histories from the published literature, and a discussion of prevailing winds and probabilities of ashfall at the site.

The completed VHA should serve as input to facility accident analyses and SSC design. It should contain enough detail for safety analysts to understand the probability of volcanic eruptions occurring and the consequences of an eruption at facilities. The VHA will be used during a facility accident analysis to establish design criteria for affected SSCs. The VHA will need to be referenced by the DSA for affected facilities.

8.3 Site Characterization of Volcanic Hazards

The first step of volcanic hazard characterization should be a compilation of volcanic vents with Quaternary activity that lie within 400 km (250 miles) of the facility or site of interest. Geologic maps produced by the USGS or university researchers are excellent sources by which to identify volcanoes. However, such sources may provide little detail on the eruption ages or the explosivity of the eruptions. A comprehensive listing of eruptions in the Pleistocene Epoch is contained in *Volcanoes of the World* (Third Edition, Seibert, et al., 2011). This listing includes only eruptions with VEI 4 and higher. A compilation of volcanoes active within the Holocene Epoch is provided by the Smithsonian Institution's Global Volcanism Program (Smithsonian Institution, 2013). This compilation is searchable online at http://www.volcano.si.edu/search_volcano.cfm.

Ashfall Hazard

The primary volcanic hazard concern to DOE facilities is ashfall, also known as tephra. As indicated by the ashfall thickness probability tables provided in Newhall and Hoblitt, eruptions with VEI 3 and lower present very little ashfall hazard to any facility more than 100 km (62.1 miles) away. Therefore, any volcano not included in Seibert, et al., and more than 100 km from the site of interest, requires no detailed characterization. If within 400 km (250 miles) of a site, it should be included in the compilation for completeness.

The primary goal of an ashfall VHA is to develop an annual probability of exceedance (APE) that certain ashfall thicknesses can be expected at a location. This requires estimates of each volcano's eruption frequency, range of eruption volumes, and likely deposition thicknesses based on regional wind patterns. A secondary goal may be to determine airborne ash concentrations at a facility during an ashfall event.

The Seibert, et al., compilation provides further references for all cataloged eruptions. For any Quaternary eruptions that merit consideration in a VHA, references should be consulted to discover the details of a given volcano's eruption timing and characteristics. The USGS has specific volcano observatories for Cascade, California, and Yellowstone volcanoes. Researchers affiliated with these observatories, or local universities, are excellent contacts for providing references on volcanoes of interest to a site. The USGS Volcano Hazards Program website (http://volcanoes.usgs.gov/) provides additional information on these observatories.

Extensive original research on volcanoes potentially affecting a site should not be necessary. From the available information, estimates or ranges of eruption parameters (frequency and magnitude, or eruption volume) should be compiled for constructing an ashfall hazard analysis. The best example of an ashfall hazard analysis for a DOE facility is USGS Open-File Report 2011-1064, *Estimate of Tephra Accumulation Probabilities for the U.S. Department of Energy's Hanford Site, Washington* (Hoblitt and Scott, 2011). This report estimates the ashfall thicknesses at the Hanford Site. Other sites are unlikely to have available eruption data comparable to the Hanford Site, as the Cascade volcanoes are the most active in the continental U.S. and have been extensively studied. With sparse data, a Monte Carlo simulation to sample the ranges of eruption frequency, volume, and wind direction can be used. The Hoblitt and Scott report excludes from its calculation all Cascade volcanoes other than Mount St. Helens, as the hazard from Mount St. Helens clearly dominates any contribution from the others. It is appropriate to exclude from the ashfall hazard calculation any volcanoes that pose a hazard an order of magnitude or more below the dominant hazard volcano.

The Hoblitt and Scott report focuses on the likely ashfall accumulation with an APE of 1E-4. This is the appropriate APE for VDC-3 facilities, and the Hanford Site facilities contain no SSCs categorized higher than VDC-3. If a DOE site contains facilities categorized higher than VDC-3, then ashfall accumulations with lower APEs (4E-5 for VDC-4, 1E-5 for VDC-5) should also be calculated. Tables 8-1 and 8-2 of the Standard provide return periods (the inverse of APEs) for ashfall hazards for VDCs 1 through 5.

Airborne concentration of ash during an ashfall event may be a concern for some facilities, especially those with ventilation systems drawing outside air. Accurately estimating airborne concentrations during ashfall is difficult, and such estimates will be imprecise. An estimate requires the analyst to assume an eruption duration and time over which the expected ashfall accumulates. This can yield a sedimentation rate that can be used to calculate an airborne concentration. Some assumptions about ash particle size, which impacts settling velocity and airborne density, may also need to be made. The Hanford Site analysis of ashfall loads in *Volcano Ashfall Loads for the Hanford Site* (Snow and Nelson, 2012) discusses airborne concentration calculations and their limitations in some detail. It also considers airborne concentrations from re-suspension of ash deposits after deposition. However, subsequent analyses sponsored by the DOE Office of River Protection (ORP) revealed that these calculations of airborne ash concentration during an initial ashfall event and subsequent re-suspension do not have a sound technical basis. The techniques employed in Snow and Nelson (2012) should not be used to determine airborne ash concentrations. ORP has commissioned work to develop more defensible techniques for estimating airborne concentration during initial ashfall and re-suspension events, and these analyses are ongoing.

Many DOE sites within 400 km (250 miles) of a Quaternary volcano may have a very small ashfall hazard when calculated using the probabilistic technique demonstrated by Hoblitt and Scott. To provide hazard perspective, a deterministic analysis of the ashfall hazard from the highest hazard volcano would be helpful. For example, if a volcano located 150 km (93 miles) from the site has erupted only once during the Quaternary, with an eruption volume of 2 km^3 (VEI 5), and it does not lie upwind in the predominant regional wind direction, the probabilistic analysis will likely show a very small ashfall hazard to the site. However, a deterministic analysis of the ashfall at the site, from a VEI 5 eruption that blows directly toward the site, would illustrate the worst-case scenario. Such a deterministic result need not be the design basis ashfall event for the site; the design basis event should be based on the probabilistic analysis. Nonetheless, the deterministic event is informative for considering beyond-design-basis events.

Volcanic ash clouds often generate extensive lightning. Lightning accompanying an ashfall event does not require any additional design consideration beyond lightning from thunderstorms. If a site is subject to ashfall, then lightning should also be considered a credible hazard and the design considerations in Section 6 of the Standard apply.

Volcanic eruptions produce gases that can affect human health and have a deleterious effect on equipment. The primary volcanic gaseous emissions, aside from water vapor, are carbon dioxide and sulfur dioxide. Volcanic gases are unlikely to pose a health hazard more than 100 km (62.1 miles) from a volcano due to atmospheric mixing and dilution. However, the sulfur dioxide can cause acid rain at greater distances downwind, so this should be considered along with ashfall in a VHA.

Local Volcanic Hazards

Local volcanic hazards include lava flows, pyroclastic flows, mud flows (lahars), ballistic projections, and asphyxiating gases. If a facility is located within 100 km of a volcano, then these local hazards should be characterized in a VHA. Topography between a volcano and a facility might preclude lava flows and lahars from reaching the facility. If so, such topographic barriers should be discussed and these hazards would require no further characterization. If no topographic barriers exist, the hazards from lava flows and lahars should be evaluated along with those from ballistic projections, pyroclastic flows and gases.

As with ashfall hazards, this portion of the VHA should use volcano eruption history (estimated frequency and magnitudes) to derive probabilities of the various local hazards affecting the site. Much information can be obtained from past research performed by the USGS or university researchers, or from other technical reports. Past studies of nearby volcanoes may be adequate to characterize volcano behavior. However, easily obtainable data that would enhance understanding of eruptive history of volcanoes within 100 km of a site should also be collected.

If a local volcanic hazard has an annual probability of occurrence near 1E-6 or higher, it should be considered in a facility accident analysis.

8.4 Design Considerations for Volcanic Hazards

Ashfall is the primary volcanic hazard considered in design. Section 8.4 of the Standard discusses design considerations for mitigating ashfall effects. The TPGs for the five VDCs are similar to those for seismic hazards given in Table 1-3 of ASCE/SEI 43-05. Because two different failure mechanisms should be considered in design for ashfall hazards, two tables for ashfall hazard return periods (inverse of APEs) are provided in the Standard. The two failure mechanisms are structural failure from ashfall loading, and functional failure of mechanical systems from ash clogging, or electrical malfunctioning. Structural designs for ashfall loads are performed by static, or equivalent static, methods more akin to the methods for structural loading from flood waters than from seismic motions. Therefore, the inherent conservatism in design for ashfall loads should be approximately the same as that for flood loads, and the hazard return periods and risk reduction factor (RRF) values for SSCs should be the same. The hazard return periods for ashfall structural loads in Standard Table 8-1 mirror the flood hazard return periods in Standard Table 5-3.

SSCs subject to functional failure under a design basis hazard have no inherent design conservatism, so they are assumed to fail unconditionally. For example, certain equipment may fail immediately if inundated: if the design basis flood level is exceeded, the SSC fails. Some SSCs may be subject to functional failure due to ashfall, such as filter clogging or other mechanical or electrical malfunction. Such SSCs should be designed to withstand a hazard with an APE equal to the TPG, i.e., an RRF equal to 1. For this reason, hazard return periods for ashfall functional failures, provided in Standard Table 8-2, match those in Standard Table 5-2.

The ashfall hazard analysis should provide estimated thicknesses with return periods corresponding to the VDC level(s) for the facilities of interest. The density of ash deposits is a key parameter for load calculations. Hoblitt and Scott suggest a dry ash density of 1 – 1.25 g/cm^3 (62.4 – 74.0 lbs/ft^3) should be used with their accumulation estimates. The density value may vary based on volcano type and distance from a volcano, with the value decreasing with distance from the source. The USGS document *Volcanic Ash-Effects to Buildings and Mitigation Strategies* states that dry, uncompacted ash densities can range from 0.5 – 1.3 g/cm^3 (32.1 – 81.2 lbs./ft^3) (see http://volcanoes.usgs.gov/ash/build/). Wet, compacted ash densities can range from 1 – 2 g/cm^3 (62.4 –125 lbs/ft^3).

The possibility of rain or snow adding to ash density before it is removed from structures should be considered. Snow and Nelson provide an example of treating moisture addition from rainfall in a probabilistic manner. Once total ashfall loads are derived including the effect of added moisture, the loads should be considered in combination with other loads, as described in Section 8.4.4 of the Standard.

Section 8.4.6 of the Standard lists some considerations in ventilation design; paramount is filter loading. Designs to accommodate ashfall should account for the airborne concentration, small particle size, and duration of an ashfall event. As noted above in Section 8.3.1, airborne concentration estimates are likely to be imprecise, and a range of values may need to be considered. The small size and abrasiveness of volcanic ash particles may pose unique hazards that should be evaluated in ventilation system design. Ash deposits can be easily re-suspended, so the effects of an ashfall event are likely to linger in a region for days or weeks. In accordance with Section 8.4.6 of the Standard, any effects of ashfall and volcanic gases on other mechanical and electrical systems will also be evaluated.

In general, SSCs cannot be designed or fortified to protect against the local volcanic hazards of lava flows, lahars, pyroclastic flows, ballistic projections, and asphyxiating gases. However, for low volume, low velocity lava flows, robust physical barriers may provide protection, similar to flood protection. If local volcanic hazards do pose credible accident scenarios that cannot be mitigated by design features, then emergency planning for facility shutdown and evacuation may be the only option to protect facility workers.

9.0 MAJOR MODIFICATION AND PERIODIC EVALUATION OF EXISTING NUCLEAR FACILITIES

9.1 Major Modifications of Existing Hazard Category 1, 2 and 3 Nuclear Facilities

A major modification is defined by 10 CFR Part 830.3, *Definitions*, as a modification "that substantially changes the existing safety basis for the facility." Section 5 of DOE-STD-1189-2016 provides criteria to determine when a facility modification meets this definition.

A major modification may include adding new SSCs, changing existing SSCs, or both. For these modifications, requirements given in the Standard would apply to (a) all newly-constructed SSCs and (b) existing SSCs requiring modification or whose failure may adversely affect the safety function of new SSCs.

9.2 Periodic Review and Update of NPH Assessments

The purpose of the ten-year NPH assessment review is to ensure the NPH assessment maintains a viable technical basis and to screen information that could significantly change the results of existing assessments. This information could take the form of new data sets, new modeling techniques, or new assessment methods. Most often, the ten-year assessment review will involve additional data, or new interpretations of data, that were not available during the previous assessment. Examples affecting PSHAs could be an expanded earthquake catalog, discovery of a new fault, or re-interpretations of existing data that modify the number and/or magnitude of past earthquakes. New seismic hazard modeling techniques, such as new ground motion attenuation models applicable to the region of interest, could also impact hazard results. A change in assessment methods could also affect the outcome of an assessment. For example, the results of PSHAs completed prior to development of the Senior Seismic Hazard Analysis Committee (SSHAC) process may be significantly different than assessments completed with a SSHAC pedigree.

Estimates of hazard changes will tend to be imprecise. An expected increase in hazard results should lead the reviewer toward recommending a new assessment. If hazard results appear to suggest a likely decrease from the earlier assessment, indicating the current assessment is conservative, this could support a recommendation against spending the resources on a new assessment. Regardless of predicted changes to hazard results, large changes to major hazard inputs alone could provide the justification for a new assessment to ensure the NPH assessment maintains a viable technical basis.

Section 6.2 of NUREG-2117, *Practical Implementation Guidelines for SSHAC Level 3 and 4 Hazard Studies*, contains a helpful discussion of how an existing hazard assessment should be evaluated for continued use. The discussion elaborates on several of the points listed above and reiterates that the reviewer should focus on changes to hazard inputs and the effects on hazard results.

The review of existing NPH assessments should be performed by a knowledgeable individual within the contractor organization, or a subcontractor, who is familiar with the existing assessments, the current NPH research near the site, and the latest hazard modeling techniques.

In 2015, DOE's Office of Nuclear Safety (AU-30) published a review on the implementation of periodic NPH assessment reviews at DOE Sites. This review resulted in the following recommendations to DOE site and facility managers to enhance the effectiveness and efficiency of NPH assessment reviews:

- Develop written procedures to guide the conduct of NPH assessment reviews in a consistent, efficient, and effective manner.

- Maintain a single document containing summaries of all NPH analyses and a log of scheduled periodic review dates. This document will be particularly valuable for sites with multiple nuclear facilities, as it can be incorporated by reference in different facility specific DSAs, simplifying the DSA maintenance at large sites.

- Consider undertaking early peer reviews and discussions with the technical experts, DOE management, and other stakeholders on the respective site's evaluation and recommendations regarding existing NPH analyses before embarking on new ones. This can be done as part of the periodic assessment review process, and ideally lead to an upfront consensus and avoid future rework.

- Establish continuous NPH data collection programs (e.g. subsurface, regional flooding, meteorological, seismic monitoring data) as part of an over-arching site-wide NPH program plan to ensure that up-to-date data will be available when performing a periodic review of the NPH assessment or initiating a new hazard analysis. Program Offices and Sites Offices should coordinate access to expertise across DOE on NPH related matters to overcome a shortage of such expertise.

- For sites with facilities under the control of multiple Program Offices, the Program/Site Offices should collaborate on their NPH review assessment effort.

The full AU-30 report is available on the DOE NPH website: http://energy.gov/ehss/natural-phenomena-hazards-program.

The requirement imposed by DOE Order 420.1C and DOE-STD-1020-2016 to perform reviews of site and facility NPH assessments every ten years and whenever significant changes are identified is applicable to DOE nuclear facilities with safety SSCs classified as NDC-3 or higher. This requirement is equally applicable, however, to comparable older DOE nuclear facilities that have not adopted the NDC classification. For these older DOE nuclear facilities, if they have SSCs classified as performance category (PC)-3 or higher, the periodic NPH assessment requirement applies.

DOE nuclear facilities not having safety SSCs classified as NDC-3 or higher located on sites that *do* contain such facilities may need to perform updates to NPH assessments because of potential interactions.

DOE sites having no nuclear facilities with safety SSCs classified as NDC-3 or higher (or PC-3 or higher) should review NPH maps from model building codes or national consensus standards every ten years, or whenever significant changes are identified, and take further action based on the significance of the new information.

9.3 Facility Condition Assessments

Section 9.3 of the Standard provides direction on evaluating the performance of existing SSCs against a new hazard level and defines a process for determining whether any SSCs should be upgraded to withstand a higher hazard level.

9.3.1

The assessment of facility condition is conducted in four steps:

Step 1: Compare the new and old hazard levels. [§9.3.2(a)]

The applicable hazard value, or hazard curve, depends on the NDC level of an SSC. Non-seismic NPH assessments derive a particular mean value (e.g., wind speed, precipitation amount, flood elevation, snow/ash load) corresponding to each NDC level. Comparing non-seismic hazard levels from a new assessment to the existing design values is a straightforward process. If the new hazard does not exceed the existing design basis hazard, no further evaluation with regard to that hazard is required. If the hazard has increased, proceed to Step 2 of this process. Note that any increase to a hazard level for a nuclear facility should be examined through the facility's Unreviewed Safety Question process.

In the case of seismic hazards, comparing new and old facility hazard levels is more complicated because (1) the hazard is represented by a spectrum of acceleration values between roughly 0.1 and 100 Hz; (2) the seismic hazard is specified in ASCE/SEI 43-05 at different return periods for different SDC; and (3) the seismic hazard may be specified at the rock outcrop, free-field surface or as a Foundation Input Response Spectrum (FIRS). Additionally, a PSHA often specifies the hazard as a Uniform Hazard Response Spectrum (UHRS) while a DRS is used in building evaluation. Care should be taken to ensure that current and previous spectra are being compared on a consistent basis. ASCE/SEI 43-05 has procedures to convert rock outcrop spectra to free-field and in-layer motions and to develop DRS from UHRS.

If the new spectral values are less than or equal to the corresponding values at each frequency on the old spectrum, then no further evaluation with regard to seismic hazard is required. If the new values are greater than the old values at each frequency, then proceed to Step 2 of this process below. For the intermediate cases where the new values are greater than the old spectral values only at selected frequencies, a contractor may elect to simply proceed to Step 2. Alternatively, additional evaluation may be avoided if the fundamental response frequencies are known for all SSCs of interest. For each SSC subject to evaluation, compare the new and old hazard values at the SSC's dominant frequencies. If the old value exceeds the new value, no further evaluation of that SSC is required. However, if the new value is higher, evaluate that SSC in accordance with Step 2.

Step 2: Compare as-built capacity to current load demand. [§9.3.2(b), (c)]

If an applicable hazard level has increased, the as-built load capacity of all facility SSCs of NDC-3 or higher should be compared to the revised load demand for that SSC. The as-built capacity (C) may be available from existing facility design documents. If a valid as-built capacity for an SSC is not available from design documents, an engineering analysis may be used to determine as-built capacity. Section 9.3.7 of the Standard lists documents that may be available to support as-built capacity calculations. These documents include the as-built drawings and specifications, facility modification records, results of SSC walk-downs, and ductile design details. If an SSC shows signs of significant deterioration, the degraded capacity should be estimated and the evaluation performed against this value instead.

The demand on the SSC is the load (forces, moments, stresses, displacements) that would result from the new hazard level through application of the design standards identified in the Standard. The demand should consider load combinations—NPH load plus non-NPH loads—in accordance with DOE-STD-1020-2016. Therefore, the demand is the minimum load to which the SSC is required to be designed if it were being designed today as part of a new facility. If the original design of an SSC is conservative, the as-built capacity may exceed the demand calculated for the increased hazard. If capacity exceeds demand for a given SSC, then no further analysis is necessary for this SSC.

The capacity and demand calculations should be documented for all facility SSCs of NDC-3 or higher. These comparisons should be performed by engineers competent to design the SSCs evaluated, and the results should be peer reviewed by other competent engineers. For SSCs that have demand exceeding the as-built capacity, proceed to Step 3.

The ductile detailing requirements of ASCE 43-05 apply for an existing SSC to have an ASCE 43-05 Inelastic Energy Absorption Factor greater than unity. For existing SSCs with non-conforming ductile detailing, a project-specific Inelastic Energy Absorption Factor, greater than unity, may be developed if fully justified and peer reviewed. ASCE/SEI 41-13 contains a rich set of deformation limits for non-conforming seismic detailing that may be used to develop ASCE 43-compatible Inelastic Energy Absorbing Factors.

Performance using nonlinear methods or project-specific deformation limits should be compared to the SSC's Target Performance Goal. If used to justify design basis earthquake performance, a technical basis for seismic margin similar to ASCE 43-05 designed structures should be demonstrated. For example, a building structure with nonductile elements that have less than a 1% probability of unacceptable performance at the DBE and less than a 10% probability of unacceptable performance at 1.5 times the DBE meets the ASCE 43-05 Section 1.3 Alternate Criteria and is acceptable. Margin estimates such as fragility analysis can also be used to justify nonductile building performance. Nonlinear methods should not be used in conjunction with the provisions in 9.3.3 of the standard without technically justified estimates of seismic margin for the structure being evaluated.

Step 3: Analyze the gap between existing capacity and current design standards. (§**9.3.3, 4, 5)**

For SSCs evaluated in Step 2 that have demand exceeding capacity, further evaluation is necessary. If an SSC's demand exceeds its as-built capacity by less than 10 percent, that SSC can be considered acceptable. As discussed in Section 9.3.3 of the Standard, the risk of SSC failure to serve its safety function is likely to be small when capacity is within 10 percent of demand, and strengthening the SSC to gain a small reduction in risk may not be cost-effective. The following example illustrates this comparison for seismic capacity and demand.

> **Example #1:** Assume the response of a SSC is dominated by horizontal seismic loading and the SSC was originally designed to withstand a horizontal seismic acceleration of 0.3g at its dominant frequency. Its original design basis considered the combination of seismic demand (D_s) and non-seismic demand (D_{ns}), and the seismic demand was 30 percent of the non-seismic demand. The original total demand was thus $1.3D_{ns}$, and the original design capacity was matched to this value, so $C = 1.3D_{ns}$. A new hazard assessment yields a horizontal seismic acceleration of 0.4g at the dominant frequency of the SSC. The seismic demand is now 40 percent of the non-seismic demand, so the total demand is now $1.4D_{ns}$. Calculating the increase in demand (i.e., $(1.4D_{ns}-1.3D_{ns})/1.3D_{ns} = 0.077$) shows that the new demand is only 7.7 percent higher than the as-built capacity. Since this increase in demand is less than 10 percent, this SSC is deemed acceptable and no further evaluation is necessary.

If an SSC's demand exceeds its as-built capacity by more than 10 percent, an additional evaluation may still find the SSC acceptable. Section 9.3.4 of the Standard describes an allowance for evaluating an existing SSC against the lower hazard posed by an NPH APE of twice the value required for a new design. However, the reduction in hazard level is capped at 20 percent. If the as-built capacity exceeds the demand posed by the lower hazard value, the SSC is acceptable. The following example illustrates such a comparison, again for an increase in seismic hazard.

> **Example #2:** An SSC was originally designed to withstand a horizontal seismic acceleration of 0.3g at its dominant frequency. Its original design basis considered the combination of seismic demand (D_s) and non-seismic demand (D_{ns}), and the seismic demand was 30 percent of the non-seismic demand. The original total demand was thus 1.3 D_{ns}, and the original design capacity was matched to this value, so $C = 1.3D_{ns}$. A new PSHA yields a horizontal seismic acceleration of 0.5g at the dominant frequency of the SSC. The SSC is categorized as SDC-3, and thus the 0.5g value is derived from a UHRS with a mean APE of 4E-4 (i.e., ground motion with a 2,500-year return period). The seismic demand is now 50% of the non-seismic demand, so the total demand is now $1.5D_{ns}$. Calculating the increase in demand (i.e., $(1.5D_{ns}-1.3D_{ns})/1.3D_{ns} = 0.154$), the new demand exceeds the as-built capacity by more than 10 percent. At the dominant frequency of interest, the new seismic hazard assessment yields a horizontal seismic acceleration of 0.42g with a mean APE of 8E-4 (ground motion with a 1,250-year return period). This represents a hazard reduction of 16 percent (0.50g-0.42g)/0.50g = 0.16, so it is an allowable reduction in hazard. The seismic demand is $0.42D_{ns}$, so the total demand is $1.42D_{ns}$. This still exceeds the as-built capacity of $1.3D_{ns}$, so this SSC will need to be included in a plan for facility upgrades.

In this example, seismic demand is a relatively small fraction of total demand, so reducing seismic hazard by 16 percent does not have a significant impact on total demand. The following example illustrates Ds as a greater fraction of total demand, as well as addressing a situation where hazard decreases more than 20 percent when mean APE is doubled.

> **Example #3:** An SSC was originally designed to withstand a horizontal seismic acceleration of 0.8g at its dominant frequency. Seismic demand was twice non-seismic demand, so $Ds = 2D_{ns}$ and total demand was thus $3D_{ns}$. The original capacity was matched to this value, so $C = 3D_{ns}$. A new seismic hazard assessment yields a horizontal seismic acceleration of 0.95g at the dominant frequency of the SSC. The SSC is categorized as SDC-3, and thus the 0.95g value is derived from a UHRS with a mean APE of 4E-4. D_s is now $2x(0.95g/0.8g)\ D_{ns} = 2.375D_{ns}$, so total demand is $3.375D_{ns}$. The new demand $(3.375\ D_{ns} - 3\ D_{ns})/3\ D_{ns} = 0.125$ exceeds as-built capacity by more than 10 percent. At the dominant frequency of interest, the new seismic hazard assessment yields a horizontal acceleration of 0.75g with a mean APE of 8E-4. This represents a hazard reduction of (0.95g-0.75g)/0.95g = 0.21, which exceeds the maximum allowable hazard reduction of 20 percent. A maximum 20-percent reduction is allowable, which corresponds to a hazard of 0.95g*0.8 = 0.76g. This 0.76g hazard is 95 percent of the original 0.8g seismic demand value. As a result, with a seismic demand of 0.76g, D_s is now $2x(0.76g/0.8g)\ D_{ns} = 1.9D_{ns}$, and total seismic demand at this hazard level is $D_{ns} + 1.9D_{ns} = 2.9D_{ns}$. This demand is lower than the as-built capacity of $3D_{ns}$, so the SSC can be considered acceptable without further evaluation or upgrades.

In the examples above, for the ease of illustration, the comparison between existing seismic demand and new seismic demand was performed considering only the dominant dynamic mode of the SSC assuming that the consideration of other modes would not change the conclusion. A complete analysis would consider the combined seismic demand from all three components of ground motion, computed in accordance with ASCE 4, and nonseismic demand, using the load combination in ASCE 43.

If an SSC has capacity less than demand and cannot meet either of the two above criteria for allowable relief, then the SSC is deemed deficient. In this case, the SSC may be unable to perform its safety function during a DBE, so the contractor should review the condition using its potentially inadequate safety analysis (PISA) process. This, and the Unreviewed Safety Question Determination (USQD) that would follow a PISA declaration, are discussed in Section 2.4 of DOE G 424.1-1B, *Implementation Guide for Use in Addressing Unreviewed Safety Question Requirements*. The PISA and USQD may indicate that, based on potential failure consequences of an SSC, near-term actions to mitigate accident consequences should be taken. If so, the contractor should take easily executable actions without delay. Likewise, if any deficient SSCs can be upgraded quickly and inexpensively, the contractor should perform such upgrades, consistent with contract requirements.

Step 4: Evaluate deficient SSCs (§9.3.7)

For SSCs that are found deficient, a fragility analysis or seismic margin study may be performed to assist in the PISA and USQD, and to justify continued operation of the facility. Guidance for performing such fragility analysis can be obtained, as appropriate for the type of facility, from *Methodology for Developing Seismic Fragilities,* EPRI Report TR-103959, June 1994; *Seismic Fragility Application Guide,* EPRI Report 1002988, December 2002; *Seismic Performance Assessment of Buildings,* FEMA Report P-58, September 2012; *Quantification of Building Seismic Performance Factors,* FEMA Report P-695, June 2009; *A Methodology for Assessment of Nuclear Power Plant Seismic Margin* (Revision 1)*,* EPRI Report NP-6041-M, August 1991; and *Seismic Analysis of Safety-Related Nuclear Structures and Commentary,* ASCE/SEI 4-98.

Section 9.3.7 of the Standard mentions median material properties. Standard 1020-2016 invokes IBC-2015 and ASCE 43-05 which invoke various material codes (ACI, AISC, etc.) to calculate SSC capacities. The capacity calculations should be consistent with the requirements of the specific material codes. In-situ material properties, including the effects of aging, may be considered provided the variability of material properties is considered in a manner consistent with the material code. Median material properties and variability are used in median-centered fragility analyses.

9.4 NPH Evaluation of SSCs in Existing Hazard Category 1, 2 and 3 Nuclear Facilities

Section 9.4 of the Standard provides guidelines for evaluating existing facilities with respect to seismic, wind, flood, and precipitation hazards. Section 9.4.1 states: “If the existing facility can be shown to meet the design and evaluation criteria presented in Section 3 of this Standard and good seismic design practice had been employed, the facility would be judged to be adequate for potential seismic hazards to which it might be subjected.”The phrase *good seismic design practice* indicates that the facility meets the ductile detailing requirements of Section 3, which are defined in either IBC-2015 or ASCE/SEI 43-05.

10.0 QUALITY ASSURANCE, USE OF EXPERTS, AND PEER REVIEW

a. For design activities in nuclear facilities, performed under a QA program based on the ASME NQA-1 standard, attention should be paid to applicable NQA-1's provisions on organization, training and qualifications, work processes, design control, audits, and design documents such as specifications, drawings, procedures and instructions.

b. Design control, which ensures that the design will perform its intended function, includes (1) defining and documenting the design organization, (2) creating criteria documents, (3) identifying applicable industry codes and standards, (4) choosing assumptions and methodologies, (5) defining output documents, (6) verifying and controlling analytical software, (7) establishing change control, (8) identifying design approvals and reports, (9) planning for independent assessments, (10) verifying the design, and (11) arranging for peer review.

c. Design verification, an integral part of design control, covers verifying and checking the adequacy of the analysis and design by any one or a combination of the following methods: (1) design reviews verifying input, output, material specifications, methodology, and assumptions, (2) use of alternate calculation methods, and/or (3) a suitable testing program. Design verification is performed by individuals or groups other than those who prepared the design.

d. Quality assurance is important in performing site investigations for characterizing NPHs to ensure that the data and the methods used to collect data are reliable. To ensure this, NPH site investigation should be performed under a QA program meeting the general quality assurance requirements of Section 10 of the Standard.

11.0 REFERENCES

Executive Orders

EO 11988 (amended by EO 13690), Floodplain Management, 1977.

EO 13690, Establishing a Federal Flood Risk Management Standard and a Process for Further Soliciting and Considering Stakeholder Input, 2015.

EO 13717, Establishing a Federal Earthquake Risk Management Standard, 2016.

Code of Federal Regulations

10 CFR Part 830, Nuclear Safety Management.

10 CFR Part 851, Worker Safety and Health Program.

DOE Directives and Standards

DOE O 420.1C, Chg. 1, *Facility Safety* (2015).

DOE O 414.1D, *Quality Assurance*, Admin Chg. 1, 2011.

DOE G 414.1-4A, *Safety Software Guide* (when issued).

DOE G 420.1-2, *Guide for Mitigation of Natural Phenomena Hazards for DOE Nuclear and Non-Nuclear Facilities*, 2012.

DOE G 424.1-1B, *Implementation Guide for Use in Addressing Unreviewed Safety Question Requirements*.

DOE-STD-1189-2016, *Integration of Safety into the Design Process.*

DOE-STD-1020-2002, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities.*

DOE-STD-1020-2012, *Natural Phenomena Hazards Analysis and Design Criteria for Department of Energy Facilities.*

DOE-STD-1020-2016, *Natural Phenomena Hazards Analysis and Design Criteria for DOE Facilities.*

DOE-STD-1021-93, Natural Phenomena Hazards Performance Categorization Guidelines for Structures, Systems, and Components.

DOE-STD-1212-2012, *Explosives Safety*.

Other DOE Documents

DOE/EH-0545, *Seismic Evaluation Procedure for Equipment in U.S. Department of Energy Facilities,* 1997.

Heller A., et al., *Mitigating Lightning Hazards, Lawrence Livermore Laboratory*, UCRL-52000, 1996, available at https://str.llnl.gov/str/pdfs/05_96.1.pdf.

Kennedy, R., et al., *Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards*, UCRL015910, 1989, available at https://www.nrc.gov/docs/ML0322/ML032230449.pdf.

Nelson, T.A., Hossain, Q.A., & Murray, R.C., *Guidelines for the Development of Natural Phenomena Hazards Design Criteria for Surface Facilities*, Lawrence Livermore National Lab, 1992, available at http://cedb.asce.org/CEDBsearch/record.jsp?dockey=0080382.

Snow, R.L., Nelson, E.A., *Volcano Ashfall Loads for the Hanford Site*, Washington River Protection Solutions,WHC-SD-GN-ER-30038 Rev 2,, 2012

SSRAP Cable Tray Report, *Review Procedure to Assess Seismic Ruggedness of Cantilever Bracket Cable Tray Supports*, (SAND92-0140, Rev. 3, Part II, UC-523), Sandia National Laboratories, 1991.

SSRAP Report, *Use of Seismic Experience Data to Show Ruggedness of Equipment in Nuclear Power Plants*, SAND92-0140, Rev. 4, Part I, UC-523, Sandia National Laboratories, 1991.

U.S. Nuclear Regulatory Commission Documents

JLD- ISG-13-01, Interim Staff Guidance, “Guidance for Assessment of Flooding Hazards Due to Dam Failure Dam,” U.S. Nuclear Regulatory Commission, 2013, available at http://pbadupws.nrc.gov/docs/ML1315/ML13151A153.pdf.

JLD-ISG-2012-06, Interim Staff Guidance, “Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment," U.S. Nuclear Regulatory Commission, January 2013, available at http://pbadupws.nrc.gov/docs/ML1231/ML12314A412.pdf.

NUREG/CP-0302, Proceedings of the Workshop on Probabilistic Flood Hazard Assessment January 29–31, 2013, available at: http://www.nrc.gov/reading-rm/doc-collections/nuregs/conference/cp0302/.

NUREG/CR-6372, Senior Seismic Hazard Advisory Committee (SSHAC), “Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use of Experts”, 1997, available at http://www.nrc.gov/reading-rm/doc-collections/nuregs/contract/cr6372/.

NUREG-1563, “Branch Technical Position on the Use of Expert Elicitation in the High-Level Radioactive Waste Program.” U. S. Nuclear Regulatory Commission, 1996, available athttp://pbadupws.nrc.gov/docs/ML0335/ML033500190.pdf.

NUREG-2117, “Practical Implementation Guidelines for SSHAC Level 3 and 4 Hazard Studies”, Rev 1, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, 2011, available at http://www.nrc.gov/reading-rm/doc-collections/nuregs/staff/sr2117/.

NUREG/CR-4461, Rev. 2, *Tornado Climatology of the Contiguous United States*, U.S. Regulatory Commission, 2007, available at https://www.nrc.gov/docs/ML0708/ML070810400.pdf.

NUREG/CR-6372, Senior Seismic Hazard Advisory Committee (SSHAC), *Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use of Experts*, prepared for the U.S. Nuclear Regulatory Commission, Vol. 1, 1997, available at https://www.nrc.gov/reading-rm/doc-collections/nuregs/contract/cr6372/.

NUREG/CR-6728, *Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-Consistent Ground Motion Spectra Guidelines*, 2001, available at https://www.nrc.gov/reading-rm/doc-collections/nuregs/contract/cr6728/.

NUREG/CR-7005, *Technical Basis for Regulatory Guidance on Design-Basis Hurricane Wind Speeds for Nuclear Power Plants*, U.S. Nuclear Regulatory Commission, 2009, available at https://www.nrc.gov/docs/ML0935/ML093521428.pdf.

NUREG-7134, "The Estimation of Very Low Probability Hurricane Storm Surges for Design and Licensing of Nuclear Power Plants in Coastal Areas," U.S. Nuclear Regulatory Commission,2012, available at: http://pbadupws.nrc.gov/docs/ML1231/ML12310A025.pdf.

Regulatory Guide 1.76, Rev. 1, *Design Basis Tornado and Tornado Missiles for Nuclear Power* Plants, U.S. Nuclear Regulatory Commission, 2007, available at https://www.nrc.gov/docs/ML0703/ML070360253.pdf.

Regulatory Guide 1.221, Design Basis Hurricane and Hurricane Missiles for Nuclear Power Plants, U.S. Nuclear Regulatory Commission, 2011, available at https://www.nrc.gov/docs/ML1109/ML110940300.pdf.

National Institute of Standards and Technology

NIST GCR 11-917-12, "ICSSC RP-8, Standards of Seismic Safety for Existing Federally Owned and Leased Buildings", 2011.

NISTIR 5734, "ICSSC RP-5, ICSSC Guidance on Implementing Executive Order 12941 on Seismic Safety of Existing Federally Owned or Leased Buildings", 1995.

National Fire Protection Association

NFPA 780-2017, *Standard for the Installation of Lightning Protection Systems*, 2017.

U.S. Army Corps of Engineers

EM 1110-2-1619, "Risk-Based Analysis for Flood Damage Reduction Studies," U.S. Army Corps of Engineers, Washington, DC, 1996, available at: http://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1619.pdf.

MGS Engineering Consultants, Inc., “Stochastic Modeling of Extreme Floods on the American River at Folsom Dam, Appendix B - Precipitation Magnitude-Frequency Characteristics for the American River Watershed,” U.S. Army Corps of Engineers, 2005, available at: http://www.hec.usace.army.mil/publications/ResearchDocuments/RD-48B.pdf.

MGS Engineering Consultants, Inc., “Stochastic Modeling of Extreme Floods on the American River at Folsom Dam, Appendix M - Sensitivity Analysis for the Stochastic Model of Extreme Floods for the American River at Folsom Dam,” U.S. Army Corps of Engineers, 2005, available at: http://www.hec.usace.army.mil/publications/ResearchDocuments/RD-48M.pdf.

Regan, P., “An Examination of Dam Failures vs. Age of Dams,” U.S. Society on Dams 29th Annual Conference Proceedings, in U.S. Army Corps of Engineers, *Managing Our Water Retention Systems*,2009, available at:
Army Corps of Engineers, Managing Our Water Retention Systems. Nashville, Tennessee, 2009.

Scharffenberg, W.A., and Fleming, M.J., “Hydrologic Modeling System: HEC-HMS,” CPD-74A, User’s Manual, Version 4.2, U.S. Army Corps of Engineers, 2010, available at: http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Users_Manual_4.2.pdf.

USACE, “HEC-RAS: River Analysis System – Applications Guide,” CPD-70, Version 5.0.3, Hydrologic Engineering Center, U.S. Army Corps of Engineers, 2010, available at: http://www.hec.usace.army.mil/software/hec-ras/documentation/HEC-AS_5.0.3_ Applications_Guide.pdf.

USACE, “Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System.” Final Report of the Interagency Performance Evaluation Task Force, Volume VIII - Engineering and Operational Risk and Reliability Analysis, U.S. Army Corps of Engineers, 2007, available at:
http://biotech.law.lsu.edu/katrina/ipet/Volume%20I%20FINAL%2023Jun09%20mh.pdf.

Other Government Documents

FEMA-433, “Using HAZUS-MH for Risk Assessment,” Federal Emergency Management Agency, 2004, available at:
http://www.fema.gov/media-library-data/20130726-1530-20490-0925/fema433_cvr_intro.pdf.

FEMA-2009, “Quantification of Building Seismic Performance Factors”, Report P-695, 2009, available at: https://www.fema.gov/media-library-data/20130726-1716-25045-9655/fema_p695.pdf.

Godesky, M., “Presentation to the National Research Council Committee on Flood Maps,” Federal Emergency Management Agency, Washington D.C., 2007.

Hoblitt, R.P., and Scott, W.E., *Estimate of Tephra Accumulation Probabilities for the U.S. Department of Energy’s Hanford Site*, Washington, U.S. Geological Survey Open-File Report 2011-1064, 2011.

Jelesnianski, C.P., J. Chen and W.A. Shaffer, “SLOSH: Sea, Lake and Overland Surges from Hurricanes,” U.S. Department of Commerce, and National Weather Service, National Oceanic and Atmospheric Administration, NOAA Technical Report NWS 48, 1992, available at: http://slosh.nws.noaa.gov/sloshPub/pubs/SLOSH_TR48.pdf.

Office of Water Data Coordination, “Guidelines for Determining Flood Flow Frequency,” Interagency Advisory Committee on Water Data of the U.S. Department of Interior, Geological Survey, Office of Water Data Coordination, Bulletin #17 B of the Hydrology Subcommittee, 1982, available at: http://water.usgs.gov/osw/bulletin17b/dl_flow.pdf.

U.S. Bureau of Reclamation, “Hydrologic Hazard Analysis,” Best Practices Chapters, U.S. Department of the Interior, 2012, available at: http://www.usbr.gov/ssle/damsafety/Risk/BestPractices/07-HydrologicHazard20121126.pdf.

U.S. Bureau of Reclamation, “Best Practices in Dam and Levee Safety Risk Analysis.” U.S. Department of the Interior, 2012, available at: http://www.usbr.gov/ssle/damsafety/Risk/BestPractices/00-Cover20121203.pdf.

U.S. Bureau of Reclamation, “Hydrologic Hazard Curve Estimating Procedures,” Research Report DSO-04-08, 2004, available at: http://www.usbr.gov/ssle/damsafety/TechDev/DSOTechDev/DSO-04-08.pdf.

U.S. Geological Survey, Volcanic *Ash: Effects & Mitigation Strategies*, available at http://volcanoes.usgs.gov/ash/build/index.html.

Industry Consensus Standards

ACI 349-13, *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary*, 2014.
ANSI/AISC N690-12, *Specification for Safety-Related Steel Structures for Nuclear Facilities*, 2012.

ANSI/ANS-2.3-2011 (R2016), *Estimating Tornado, Hurricane, and Extreme Straight-Line Wind Characteristics at Nuclear Facility Sites*, 2011.

ANSI/ANS-2.26-2004 (R2010), *Categorization of Nuclear Facility Structures, Systems, and Components for Seismic Design,* 2004.

ANSI/ANS-2.27-2008 (R2016), *Criteria for Investigation of Nuclear Facility Sites for Seismic Hazard Assessment*, 2008.

ANSI/ANS-2.29-2008, *Probabilistic Seismic Hazards Analysis*, 2008.

ANSI/ANS-3.11-2015, *Determining Meteorological* Information *at Nuclear Facility Sites,* 2015.

ASME NQA-1-2008 with NQA-1a-2009 addenda, *Quality Assurance Requirements for Nuclear Facility Applications*, 2009.

ASCE 4-98, *Seismic Analysis of Safety-Related Nuclear Structures*, 2000.

ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures*, 2013.

ASCE/SEI 41-13, *Seismic Evaluation and Retrofit of Existing Buildings*, 2013.

ASCE/SEI 43-05, *Seismic Design Criteria for Structures, Systems and Components in Nuclear Facilities,* 2005.

EPRI*,* NP-5228, *Generic Seismic Ruggedness of Power Plant Equipment*, 1991, available at https://www.epri.com/#/pages/product/NP-5223-SLR1/.

EPRI, Technical Report 103959, *Methodology for Developing Seismic Fragilities*, 1994, available at https://www.epri.com/#/pages/product/TR-103959/.

EPRI, 1002988, *Seismic Fragility Application Guide*, December 2002, available at https://www.epri.com/#/pages/product/000000000001002988/.

EPRI, 1019309, *Seismic Qualification Reporting and Testing Standardization (SQURTS) Test Report Database*, 2009, https://www.epri.com/#/pages/product/1019309/.

EPRI*,* 1024889, *Seismic Instrumentation at Nuclear Power Plants,* January 2012, available at https://www.epri.com/#/pages/product/000000000001024889/.

IBC-2015, *International Building Code®*, International Code Council, 2015.

Other References

Annaka, T., K. Satake, T. Sakakiyama, K. Yanagisawa, and N. Shuto, "Logic-tree Approach for Probabilistic Tsunami Hazard Analysis and its Applications to the Japanese Coasts," Pure and Applied Geophysics, 164 (577–592), 2007.

Applied Research Associates, Inc., "HAZUS-MH Advanced Severe Storm Coastal Risk Assessment Methodology," prepared for the National Institute of Building Sciences, 2006.

Booij, N., et al., "SWAN Cycle III version 40.51: User Manual," Delft University of Technology, The Netherlands, 2006 available at: http://falk.ucsd.edu/modeling/swanuse.pdf.

Bowles, D., "Risk Analysis as Applied to Dam Safety Fundamentals: Risk Modeling Power Existing Dam," IDSRM, Utah State University, Presented at Polytechnic University of Valencia, Valencia, Spain, 2003.

Bunya, S., J.et al., "A High Resolution Coupled Riverine Flow, Tide, Wind, Wind Wave and Storm Surge Model for Southern Louisiana and Mississippi: Part I–Model Development and Validation." Monthly Weather Review, American Meteorological Society, 138(2), 345-377, 2010, available at: http://journals.ametsoc.org/doi/pdf/10.1175/2009MWR2907.1.

Downes, G.L. and M.W. Stirling, "Groundwork for Development of a Probabilistic Tsunami Hazard Model for New Zealand" International Tsunami Symposium Proceedings, session 1 (293-301), 2001, available at: http://nthmp-history.pmel.noaa.gov/its2001/Separate_Papers/1-06_Downes.pdf.

Fontaine, T.A. and K.W. Potter, “Estimating Probabilities of Extreme Rainfalls,” American Society of Civil Engineers, Journal of Hydraulic Engineering, 115(11), 1562-1575, 1989.

Freeman, G.E., R.R. Copeland, M.A. Cowan, “Uncertainty in Stage-Discharge Relationships,” Stochastic Hydraulics ’96, Proceedings of the Seventh IAHR International Symposium on Stochastic Hydraulics, Mackay, Queensland, Australia, , 1996, A.A. Balkema, Rotterdam, Netherlands.

Hydronia, “RiverFlo-2D: Two Dimensional Finite Element River Dynamics Model Argus One Edition: User’s Guide, Release v3.0,” Hydronia, LLC, Pembroke Pines, FL, March 2012, available at: http://www.hydronia.net/downloads/documentation/.

McCann, Jr., M.W., et al.,“Preliminary Safety Evaluation of Existing Dams,” Volume II User’s Manual, The John A. Blume Earthquake Engineering Center, Department of Civil and Environmental Engineering, Stanford University, Report 70, November 1985, available at: https://stacks.stanford.edu/file/druid:mp335bn5254/TR70_Mccann.pdf.

McCann, Jr., M.W., “Seismic Risk of a Co-Located Portfolio of Dams – Effects of Correlation and Uncertainty,” 3rd International Week on Risk Analysis, Dam Safety, Dam Security, and Critical Infrastructure Management, Polytechnic University of Valencia, Valencia, Spain, 2011.

National Research Council “Estimating Probabilities of Extreme Floods—Methods and Recommended Research,” National Academy Press, Washington, D.C., 1988.

National Research Council of the National Academies, “Mapping the Zone, Improving Flood Map Accuracy,” National Academy Press, Washington, D.C., 2009.

Newhall, C.G., and Hoblitt, R.P., Constructing Event Trees for Volcanic Crises, Bulletin of Volcanology, v. 64, pp. 3-20, 2002.

Newhall, C.G., and Self, S., The Volcanic Explosivity (VEI): An Estimate of Explosive Magnitude for Historical Volcanism, Journal of Geophysical Research, v.87, pp. 1231-1238, 1982.

Pacific Gas & Electric Company, “Methodology for Probabilistic Tsunami Hazard Analysis: Trial Application for the Diablo Canyon Power Plant Site,” PEER Workshop on Tsunami Hazard Analyses for Engineering Design Parameters, Berkeley CA, 2010.

Park, J., Bazzurro, J.P., & Baker, J.W., “Modeling Spatial Correlation of Ground Motion Intensity Measures for Regional Seismic Hazard and Portfolio Loss Estimation,” 10th International Conference on Application of Statistics and Probability in Civil Engineering (ICASP10), Tokyo, Japan, 2007.

Platzman, G.W. “The Dynamical Prediction of Wind Tides on Lake Erie,” Tech Report #7, Dept. of Geophysical Science, University of Chicago, 1963.

Reed, J.W., et al., “A methodology for Assessment of Nuclear Power Plant Seismic Margin” (EPRI-NP--6041-M-Rev1). United States, 1991

Schaefer, M.G., “Regional Analyses of Precipitation Annual Maxima in Washington State,” Water Resources Research, 26(1), 119-131, 1990.

Seed, H.B., K.L. Lee, I.M. Idriss, “Analysis of the Sheffield Dam Failure,” Journal of Soil Mechanics and Foundations Division, ASCE 95(6), 1453–1490, 1969, available from the National Information Service for Earthquake Engineering (NISEE).

Seismic Qualification Utility Group, *Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment*, Rev. 2, 1992, NRC Accession # ML040560263.

Shuto, N. “Numerical Simulation of Tsunamis,” in *Tsunami Hazard: A Practical Guide for Tsunami Hazard Reduction*, E. N. Bernard (ed.), Springer Reprint, 2012.

Smith J. M., and A.R. Sherlock, “Full-plane STWAVE with Bottom Friction: II. Model Overview,” U.S. Army Engineer Research and Development Center, Vicksburg, MS, 2007, available at: http://www.aquaveo.com/software/sms-stwave.

Smith, J.M., Sherlock, A.R., and Resio, D.T., “STWAVE: Steady-state Spectral Wave Model User’s Manual for STWAVE,” Version 3.0, ERDC/CHL SR-01-1, U.S. Army Engineer Research and Development Center, Vicksburg, MS, 2001, available at http://chl.erdc.usace.army.mil/Media/2/4/4/erdc-chl-sr-01-11.pdf.

Stanford University “National Performance of Dams Program,” at http://npdp.stanford.edu.

Thio, H.K., P. Somerville and G. Ichinose, “Probabilistic Analysis of Strong Ground Motion and Tsunami Hazards in Southeast Asia,” Journal of Earthquake and Tsunami, 01 (119), 2007.

Thio, H.K., P. Somerville, and J. Polet, “Probabilistic Tsunami Hazard in California,” PEER Report 2010/108 (331), University of California atBerkeley, 2010, available at: http://peer.berkeley.edu/publications/peer_reports/reports_2010/web_PEER2010_108_THIOetal.pdf.

Titov V.V. and C.E. Synolakis, “Extreme Inundation Flows During the Hokkaido-Nansei-Oki Tsunami,” Geophysical Research Letters, 24(11):1315-1318, 1997, available at: http://onlinelibrary.wiley.com/doi/10.1029/97GL01128/pdf.

Titov V.V. and F.I. González, “Implementation and Testing of the Method of Splitting Tsunami (MOST) Model,” NOAA Technical Memorandum, ERL PMEL-112, NOAA/Pacific Marine Environmental Laboratory, Seattle, Washington, 1997, available at: http://www.pmel.noaa.gov/pubs/PDF/tito1927/tito1927.pdf.

Titov V.V., “Numerical Modeling of Long Wave Runup,” Ph.D. Thesis, University of Southern California, Los Angeles, California, 1997, available at: http://search.proquest.com/science/docview/304370736.

Toro, G.R., et al., “Efficient Joint Probability Methods for Hurricane Surge Frequency Analysis,” Ocean Engineering, Vol. 37(1), 125-134, 2010.

URS Corporation and Jack R. Benjamin & Associates, Inc., "Delta Risk Management Strategy, Phase 1: Risk Analysis Report," prepared for the California Department of Water Resources, 2009, available at: http://www.water.ca.gov/floodsafe/fessro/levees/drms/phase1_information.cfm#.

Vickery, P., and Wadhera, D., "Statistical Models of Holland Pressure Profile Parameter and Radius to Maximum Winds of Hurricanes from Flight Level Pressure and H*Wind Data," Journal of Applied Meteorology and Climatology, Vol. 47, p. 2497, ff., October 2008.

Westerink, J.J., et al., "A Basin to Channel Scale Unstructured Grid Hurricane Storm Surge Model Applied to Southern Louisiana," Monthly Weather Review, American Meteorological Society, 136(3):833–864, 2008.

Yankee Atomic Electric Company, "Probability of Extreme Rainfalls and the Effects on the Harriman Dam," Report YAEC-1405, Framingham, MA, 1984.

Appendix A: NPH Design and Evaluation Methodology Using Target Performance Goals (TPG)

Since the early 1990s, the methodology of the Department of Energy's (DOE's) Natural Phenomena Hazard (NPH) design of SSCs has been based on achieving TPGs that are selected using a graded approach in relation to the safety significance. The NPH design and evaluation criteria presented in the Standard also uses this method. The criteria used are intended to ensure that structures, systems, and components (SSCs) in DOE facilities can perform their intended safety functions in the event of design basis NPH events. Such events include earthquakes, extreme winds, flood, precipitation, lightning, and volcanic eruption hazards.

For NPH design purposes, SSCs are sorted into five different design categories depending on the severity of failure consequence. Each SSC in a facility is assigned an NPH Design Category (NDC) using the categorization criteria and procedure described in ANSI/ANS-2.26-2004, *Categorization of Nuclear Facility Structures, Systems, and Components for Seismic Design*, with certain modification made in the 2012 and 2016 versions of DOE-STD-1020. In DOE-STD-1020-2016 ("the Standard"), the NDC of an SSC determines the rigor of the analytical method to determine the NPH demand and stringency of the design acceptance criteria. Thus, for example, NDC-1 and NDC-2 SSCs may be analyzed and designed using IBC-2015, subject to the limitations given in the Standard. However, in accordance with the Standard, NDC-3 or higher SSCs are analyzed using the more rigorous analytical methods and design acceptance criteria specified in the Standard and references therein.

In a performance goal (PG)-based method of NPH design, performance is measured by TPGs expressed as an annual probability of an SSC's failure to perform its intended safety function in the event of a NPH event. When such a PG-based method of design is used, the annual probability of an SSC's failure resulting from a given type of NPH event is likely not to exceed the preselected target PG. DOE Order 420.1C, Chg. 1, *Facility Safety*, mandates use of the Standard's design provisions, and the Standard in turn adopts with certain modifications, ANSI/ANS-2.26, ASCE/SEI 43-05, and IBC-2015. The Standard's provisions establish a graded approach for NPH requirements by defining five NPH NDCs for SSCs, with each NDC aimed at a numerical TPG.

In the 2002 version of the Standard, numerical TPGs were selected based on an SSC PC that is not identical to NDC of the SSC. The term "Performance Category" of an SSC is defined as its annual failure rate (i.e., its numerical target PG), the required analysis and design requirements and its permissible deformation level (i.e., Limit State). See Table A-1 below.

Table A-1. Numerical Target Performance Goals and Permissible Deformation Levels for Various Performance Categories in DOE-STD-1020-2002

SSC Performance Category (PC)	Numerical Target Performance Goal	Limit State or Permissible Deformation Level
PC-0	No requirement specified	No requirement specified
PC-1	Not explicitly specified, but is the same as that achieved for facilities designed by IBC 2000 Category II	Not explicitly specified, but is the same as that achieved for facilities designed by IBC 2000 Category II
PC-2	Not explicitly specified, but is the same as that achieved for Important facilities designed by IBC 2000 Category IV	Not explicitly specified, but is the same as that achieved for Important facilities designed by IBC 2000 Category IV
PC-3	1×10^{-4} per year	Limited permanent deformation
PC-4	1×10^{-5} per year	Limited permanent deformation

In the Standard, the five PCs above have been replaced by five NPH Design categories (NDCs), NDC-1 through NDC-5. However, the term "NDC" of an SSC defines its annual failure rate (that is, its numerical target PG). Limit State or SSC failure definition is uncoupled from NDC and is defined separately based on safety functions of the SSC. Selection of Limit States (for seismic hazards) and SSC failure definitions (for non-seismic hazards) are discussed in Section 5 and Appendix B of American National Standards (ANSI)/American Nuclear Society (ANS) 2.26; numerical target performance goals (TPGs) for various NDCs are listed in Table A-2 below:

Table A-2. Numerical Target Performance Goals NPH Design Categories in DOE-STD-1020-2016

NPH Design Category (NDC)	Numerical Target Performance Goal
NDC -1	Not explicitly specified, but is the same as that achieved for IBC-2015 Risk Category II Facilities
NDC -2	Not explicitly specified, but is the same as that achieved for IBC-2015 Risk Category IV Facilities
NDC -3	Same as PC-3, i.e., 1×10^{-4} per year *
NDC -4	4×10^{-5} per year *
NDC -5	Same as PC-4, i.e., 1×10^{-5} per year *

* Explicitly targeted only for seismic design (see ANSI/ANS-2.26-2004 and ASCE/SEI 43-05); for non-seismic NPH design, target PG values have not been quantified.

Table A-2 is created to show equivalency between PC vs NDC were necessary to make DOE-STD-1020-2016 consistent with ANSI/ANS-2.26 and ASCE/SEI 43-05. SSCs are designed using NPH criteria appropriate for achieving the target TPGs for NDCs that are selected primarily based on the safety consequences of SSC failure.

The TPG in Table A-2 provides an initial mapping between the SSC Performance Categories (PC) used in DOE-STD-1020-2002 and the current NDC, i.e. PC-1→NDC-1, PC-2→NDC-2, PC-3→NDC-3 and PC-4→NDC-5. Comparing the PC limit state description in Table A-1 to the NDC limit state description in ANSI/ANS-2.26 and ASCE 43-05 refines the initial mapping to PC-3→(NDC-3, LS-C) and PC-4→(NDC-5, LS-C). An important caveat to this initial mapping is the treatment of common-cause failure and system interaction (commonly referred to as “two-over-one” phenomena). The PC source SSC, defined by DOE-STD-1021-93, *Natural Phenomena Hazards Performance Categorization Guidelines for Structures, Systems, and Components*, may differ significantly with the NDC source SSC, defined by ANSI/ANS-2.26-2004 (R2010). Specifically, one method used by ANSI/ANS-2.26 to prevent system interaction puts the source SSC in the same NPH design category as the target structure, system and component (SSC), and then allows the source SSC to have a larger limit state. This approach removes the qualitative assessment of the potential target failure inherent in DOE-STD-1021-93. A second important caveat is that the initial mapping also assumes that the NPH hazard used for the PC and NDC target performance goals in Table A-2 are consistent.

The TPGs correspond to annual probabilities of SSC damage due to NPHs and do not extend to consequences beyond SSC damage. The annual probability of exceedance (APE) of SSC damage as a result of NPHs is a combined function of the APE of the event, factors of safety introduced by the design/evaluation procedures, and other sources of conservatism. DOE-STD-1020-2016 criteria specify hazard APE, response evaluation methods, and permissible behavior criteria for each NPH and each NDC category such that desired TPGs are achieved for either design or evaluation. The ratio of the seismic hazard APE and the seismic PG APE is designated as R_p in ASCE/SEI 43-05 (called the risk reduction ratio, R_R in DOE-STD-1020-2002). This ratio establishes the level of conservatism to be employed in the seismic design or evaluation process. For example, if the PG and hazard annual probabilities are the same (i.e., $R_p = 1$), the design or evaluation approach would not introduce conservatism. However, if conservative design or evaluation approaches are desired, the hazard APE can be larger than the PG annual probability, $R_p > 1$. In the criteria presented in the Standard, the hazard probability and the conservatism in the design/evaluation method are not the same for all NPHs, but these were intended to be the same as those in DOE-STD-1020-2002. However, in both versions of the Standard, the accumulated effect of each step in the design/evaluation process is aimed at the PG probability values, which are unique to each NPH.

Appendix B: Probabilistic Flood Hazard Analysis

B.1 Flood Hazards Overview

In many ways, flood hazards differ significantly from all other natural phenomena hazards (NPHs). As an example, it is often relatively easy to eliminate potential flood hazards listed in DOE-STD-1020-2016 Table 5-1 as a potential contributor to damage at a site through application of strict siting requirements or location of important structures, systems, and components (SSCs) at levels above plant grade. Similarly, the opportunity to effectively utilize warning systems and emergency procedures to limit damage and personnel injury is significantly greater in the case of flooding than it is for seismic or extreme winds, since the onset and magnitude of flood conditions is more predictable, enabling better emergency preparedness and response mechanisms.

This appendix describes a general framework for conducting a probabilistic flood hazard analysis (PFHA). The framework is applicable to those external flood sources that have not been screened out by physical or other arguments (e.g., probabilistic bounding assessments) as described in Section 5.4 of DOE-STD-1020-2016 ("the Standard" hereafter). The purpose of a PFHA for a site or facility is to evaluate the effects on the facility caused by floods. The evaluation is used to determine the design basis flood level (DBFL,), which for SSCs is risk-informed and therefore defined probabilistically.

The exposure of the Department of Energy (DOE) facilities to flood hazards can vary significantly from site-to-site and from facility-to-facility within a site. A site or facility may be exposed to the effects of multiple sources of flooding and associated hazards such as inundation and hydrodynamic and hydrostatic impacts. For instance, a facility located near a river may be exposed to the effects of local precipitation, riverine flooding, controlled releases from an upstream dam, or the flooding that results from a dam failure. These particular sources of flooding may be independent of one another, or in some cases, correlated. Controlled releases from an upstream dam may be caused by extreme precipitation that also overwhelms the capacity of onsite drainage. Table B-1 lists various causes of flooding and the flood hazards that may be experienced. (Refer to Section 5.3 and Tables 5-1 and 5-4 of the Standard.)

Table B-1. Flood Sources, Causes, Contributing Factors, and Potential Flood Hazards

Flood Source	Causes and Contributing Factors	Potential Flood Hazards
Riverine Flooding	Precipitation and/or snow melt Debris jams Ice jams Controlled releases from upstream dams	Inundation Hydrodynamic forces Wave action Sedimentation Ice loads
Dam Failure – Uncontrolled Releases	Dam failure initiated by the following earthquake hazards: ground shaking, fault displacement, landslides, seiche, etc. Dam failure initiated by intrinsic (non-NPH related) factors or events: embankment instability, seepage and piping, foundation instability Failure of an upstream dam Releases from an upstream dam Uncontrolled releases may also occur as a result of the failure of gates or outlet works	Inundation Erosion Hydrodynamic loads Sedimentation
Levee, Dike or Floodwall Failure	Levee, dike, floodwall failure initiated by earthquakes and their effects Intrinsic factors or non-NPH related events Upstream dam failure Upstream dam releases Uncontrolled releases may also occur as a result of the failure of gates or outlet systems	Inundation Erosion Hydrodynamic loads Sedimentation
Precipitation/ storm runoff	Precipitation Snowmelt	Inundation Hydrodynamic loads
Tsunami	Earthquakes (even events where ground motion is not locally felt)	Inundation Hydrodynamic loads
Seiche	Earthquake High Winds	Inundation Hydrodynamic loads
Storm surge	Hurricanes, tropical storms and tropical depressions Squall lines	Inundation Hydrodynamic loads
Wave action	Winds, often accompanying storm surge Hurricanes, tropical storms and tropical depressions Squall lines	Inundation Hydrodynamic loads
Debris	River flooding—can contribute to dam overtopping, compromise dam spillway capacity Local precipitation and clogging of drainage systems	Hydrodynamic loads Impact loads

B.2 Probabilistic Framework

B.2.1 Overview

Floods are temporally- and spatially-variant stochastic events whose occurrence and magnitude (amount of precipitation or peak discharge) are defined in terms of their frequency, return period, or probability of occurrence. As with other NPH phenomena, the ability to accurately estimate the frequency or probability of future occurrences is subject to uncertainties in the historic record, data limitations, an incomplete understanding of the physical phenomena, and modeling uncertainties (For further information, see NUREG/CP-0302, 72017, *Proceedings of the Workshop on Probabilistic Flood Hazard Assessment* (PFHA). In spite of these challenges, technical developments in the past 20-30 years have established a reasonable framework for performing probabilistic assessments of extreme natural phenomena events. Such assessments can be used to support regulatory decision-making and to determine design basis loads (For further information, NUREG-2117, 2011, *Practical Implementation Guidelines for SSHAC Level 3 and 4 Hazard Studies*). Once design basis loads are established, engineered controls can be effectively designed and constructed, and administrative controls can be implemented to provide additional safeguards.

The probabilistic framework for developing PFHAs built on the following components:

- A probabilistic uncertainty model that describes the stochastic nature of random flood events and hazards, termed an "aleatory uncertainty model";
- A probabilistic uncertainty model that accounts for parametric knowledge-based uncertainties on the frequency and magnitude of flood events and hazards, termed "epistemic uncertainty model";
- Utilization of all current, relevant and applicable information on both recent floods and floods evidenced in the geologic record (e.g., paleofloods);
- Use of computational modeling capabilities; and
- Explicit modeling of epistemic uncertainties.

The last of the listed factors includes the formal, structured elicitation of expert interpretations, evaluation and integration of sources of uncertainty, and participatory peer review that validates the evaluation process and provides technical review and oversight (SSHAC process – see below for details).

Probabilistic evaluation of NPHs can be a particularly complex undertaking. In general, available data may be inadequate to fully define the parameters in an aleatory model, especially when estimating frequencies of occurrence of 10^{-3} per year or less. In addition, there may be different models and/or alternative interpretations that are scientifically viable and consistent with the present state-of-knowledge. In these cases, it is often necessary to elicit interpretations from experts on available and emerging evidence, establish parameter estimates, and evaluate and weigh the applicability of different models and alternative interpretations. The use of experts in structured, formal processes, such as the drafting of a PSHA, has been recognized as an acceptable approach in regulatory environments to evaluate complex technical problems, to develop more comprehensive model inputs, and to evaluate epistemic uncertainties. (For further information, see NUREG-2117, Revision 1, 2011, and NUREG-1563, *Branch Technical Position on the Use of Expert Elicitation in the High-Level Radioactive Waste Program*, 1996.)

B.2.2 Taxonomy of Uncertainties

Conducting a PFHA presupposes a conceptual framework for evaluating and modeling sources of uncertainty. This subsection describes such a framework. Estimating the future frequency of occurrence of flood hazards can be viewed in the context of applicable engineering models (which may be analytical, empirical, or statistical) and estimates of model parameters. With respect to evaluating and modeling uncertainty, there are two types of uncertainties that contribute to the estimate of the frequency of flooding: aleatory and epistemic.

Aleatory uncertainty refers to the inherent randomness of events or properties and the stochastic nature of meteorological and hydrological phenomena causing floods. The effects of these contributing events are predicted in terms of their frequency of occurrence, or the fraction of the time an event of a given type may occur. An example of a source of aleatory variability is the frequency of occurrence per year of significant rainfall events in at a specific location.

Epistemic, or knowledge-based uncertainty, has two components. It refers to a lack of knowledge about meteorological and hydrological phenomena, or the physical processes that drive such events which limits the analyst's ability to accurately model events of interest, and is termed "model epistemic uncertainty." Contributing to this lack of knowledge are limitations in available data that affect the assessment of model parameters. These limitations are termed "parametric epistemic uncertainty." When data are limited, parameter estimates used to analytically represent the data base may result in large uncertainties, traditionally viewed in terms of large statistical confidence intervals on parameter estimates.

A hazards model generally includes deterministic or statistical models of the physical phenomena and probabilistic/statistical models of data or processes used to estimate the occurrence of events of interest (rainfall, watershed response and flooding). By their very nature, models are, at best, a limited representation of reality, subject to several parameter uncertainties.

To systematically identify and assess the source of uncertainties, a taxonomy to partition the types of uncertainty in terms of their effect on models and estimates of model parameters needs to be constructed. Table B-2 shows a taxonomy for classifying four types of uncertainties: (1) model aleatory, (2) model epistemic, (3) parameter aleatory, and (4) parameter epistemic. This taxonomy offers a number of benefits in developing and quantifying a probabilistic hazards model. It supports the identification of all sources of uncertainty and the characterization of these uncertainties as aleatory or epistemic and their evaluation. Because the classifying of different sources of uncertainty can be difficult to make, the table below may be helpful in avoiding double-counting or omissions.

Table B-2. Taxonomy of Uncertainties

	Epistemic Uncertainty	Aleatory Uncertainty
Modeling	Uncertainty about a model and the degree to which it can predict events.	Variability not explained by a model. This may be variability that is attributed to elements of the physical process that are not explicitly modeled and therefore represents variability (i.e., random differences) between model predictions and observations.
Parametric	Uncertainty associated with estimating model parameters from available data, indirect measurements or other evidence.	Similar to aleatory modeling uncertainty due to systematic, but random variations associated with parameters of a model. An example would be storm-to-storm variation in hurricanes with the same high-level parameters, but which differ due to smaller-scale elements of the storms that are not modeled but still have a systematic effect. This represents an aleatory inter-event variability that may be considered independent from event to event.

B.2.3 PFHA Results

It is useful to identify and visualize the type of results that may be generated in a PFHA when both modeling and parametric aleatory and epistemic uncertainties are explicitly modeled. Figures B-1 and B-2 illustrate examples of a flood-frequency result for a riverine flooding evaluation that may be the result of extreme precipitation events.

Figure B-1 shows the final aggregate estimate of the flood hazard. In this figure, the hazard is characterized as the peak flood elevation. Also shown is the epistemic uncertainty in peak flood elevation levels at a given frequency of exceedance per year, which is a function, in part, of the model and parameter uncertainties in estimating flow discharges and stages. Figure B-1 also shows, for a given peak flood elevation level, the uncertainty in the estimated frequency of exceedance per year. This uncertainty is defined by a probability distribution that shows families of curves, which quantify all the sources of epistemic uncertainty in the hazard analysis. The uncertainty is represented on the plot by selected fractiles (0.05, 0.15, 0.5, 0.85 and 0.95, and the mean). The range of results reflects the model and parameter uncertainties in modeling hydrologic processes for large and extreme floods. Some floods shown in the model results are likely larger than events that have been observed in the historic record.

Studies of extreme flooding events lead to the following observations:

- Generally, not a single event but rather a combination of different events or other factors produces a particular outcome such as a flood exceeding a specific flood elevation. In the case of riverine flooding, high flows may be the result of extreme precipitation resulting from extra-tropical storms, seasonal rainfall events such as monsoons, and early warm spring temperatures causing snowmelt.

High flows may also be caused by precipitation from a storm that has stalled for an extended period of time over a basin, due to an omega block in the atmosphere.

- There are aleatory and epistemic uncertainties in hydraulic evaluations and thus in resultant flood levels.

- The evaluation of epistemic uncertainty in the PFHA is a difficult and complex process. Accordingly, it is recommended that participatory peer review of the evaluation process be carried out (see NUREG-2117, Revision 1, 2011).

These observations are illustrated in Figure B-2, Parts (a), (b), and (c). The complexity of sources of epistemic uncertainty illustrated in Figure B-2 has implications for either comprehensive PFHAs and for simplified evaluations that may be conducted as part of screening evaluations. In a screening evaluation, the objective typically is to show that either: (a) flood levels cannot physically reach elevations where SSCs are located, or (b) the frequency of occurrence of floods that can reach SSCs is sufficiently low (less than 1E-6 per year) that the risk to the facility is negligible and therefore acceptable.

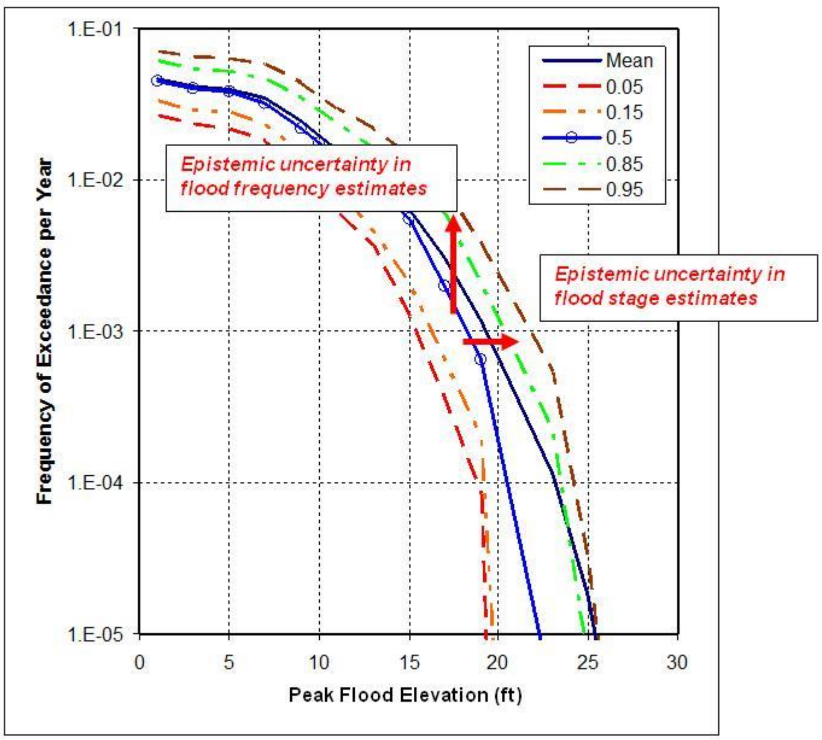

Figure B-1. Flood Hazard Results for a Riverine Flood Site

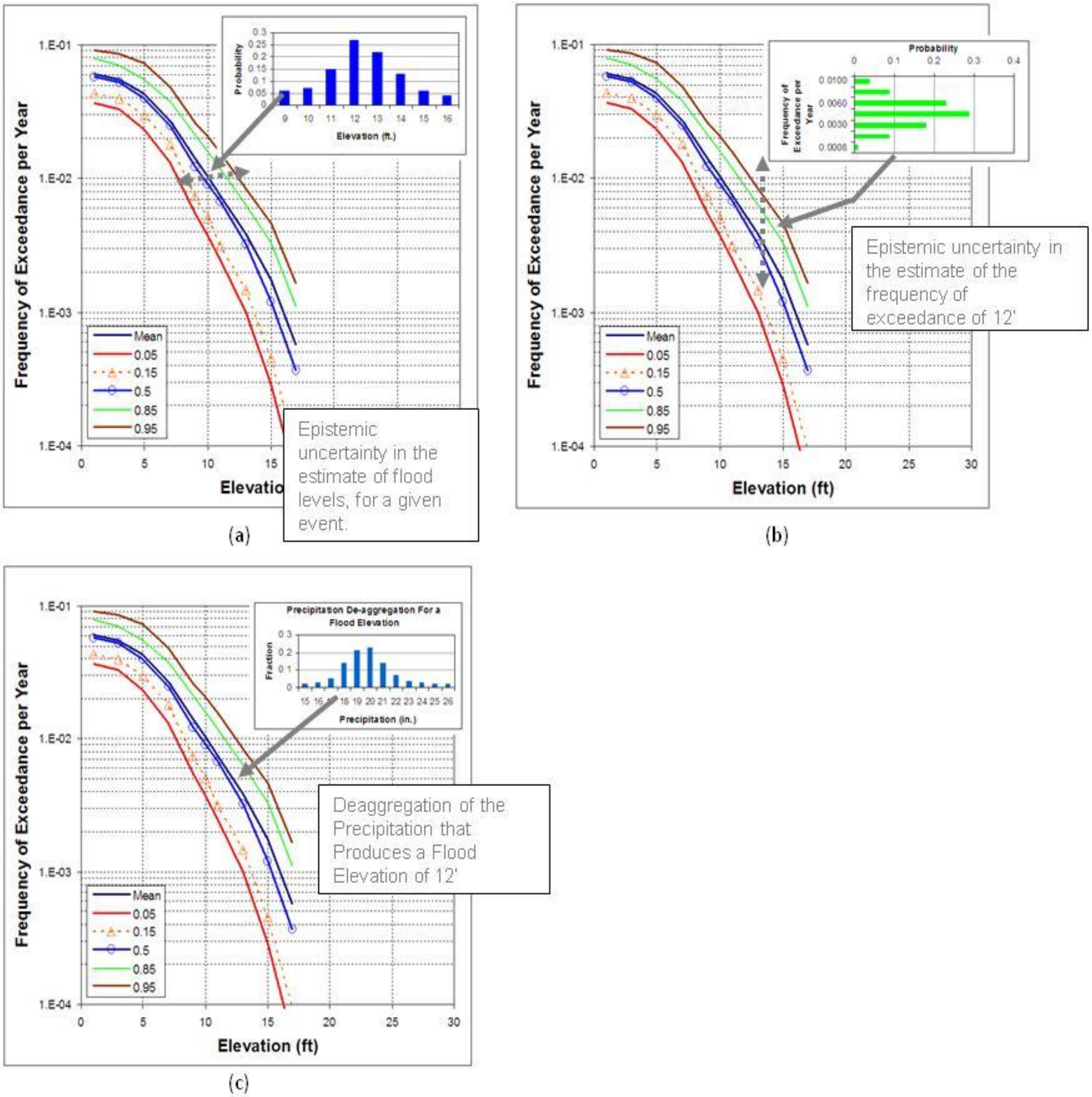

Figure B-2. Factors that Contribute to an Estimate of the Extreme Flood Elevations

(a) Epistemic uncertainty in the estimate of flooding levels that may occur for a given flood event;
(b) Epistemic uncertainty in the frequency of exceedance of flood elevations; and
(c) De-aggregation of the precipitation events that, in combination with other factors such as antecedent conditions, can produce a flood elevation of 12 feet.

Demonstrating that these objectives are met requires consideration of the sources of epistemic uncertainty. A successful demonstration should show with reasonable confidence that a particular flood hazard does not pose an unacceptable risk to a facility in comparison to the applicable flood design Performance Goal (PG).

B.2.4 Flood Hazard Characterization

Prior to conducting the PFHA, the analyst should discuss with design and engineering organizations what parameters they may require to characterize the flood hazard at each facility to determine appropriate controls. While peak flood elevation is a necessary and common characterization of the severity of flooding, this parameter alone may not fully characterize the extent of the hazard to important SSCs. Accordingly, the analyst and the designers should jointly address the following considerations:

- Warning time needed to take mitigation and protective actions for various flooding scenarios;
- Potential for debris, sediment, or other waterborne objects to be transported to the SSCs;
- Duration of flooding above critical SSC elevations;
- Potential for waves or spray action that could compromise electrical or electronic SSCs;
- Flow velocities and the potential for erosion or other site damage; and
- Hydrodynamic, hydrostatic, or impact loading considerations.

With respect to emergency preparedness and response considerations, the analyst should work with the operations and emergency management organizations to determine how to best characterize the flood hazard with respect to the planning stages of the PFHA.

B.3 Probable Maximum Methods

For many years, probable maximum methods have been used to estimate precipitation, riverine floods, storm surges, etc., to define design basis flood events. The intent of these methods was to produce conservative bounding flood scenarios used to design flood protection features with sufficient safety margins. These earlier methods were deterministic, and as such, the frequency of occurrence of flooding was not estimated and the various uncertainties considered in the conservative estimate of flood levels were not considered. Because TPGs are now set using probabilistic standards, probable maximum methods can no longer be used.

B.4 PFHA Goals, Elements and Models

B.4.1 Overview

Prior to addressing PFHA techniques associated with specific flood types, general aspects of conducting PFHAs are discussed with various site organizations. These aspects include the overall goal of the analysis, the organizational structure of the PFHA team, and the responsibilities of the team participants.

The Senior Seismic Hazard Advisory Committee (SSHAC) process (see NUREG-2117, Rev. 1, and NUREG/CR-6372, 1997, *Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard and Risk Consistent Ground Motion Spectra Guidelines*), which was originally developed for PSHAs, is uniquely suited to conducting probabilistic analysis of other types of NPHs. A key feature of the SSHAC process is the organizational structure it employs, which identifies the roles and responsibilities of members of the PFHA team and of different types of Subject Matter Experts (SMEs). SSHAC guidelines also provide an outline for conducting a SSHAC Level 3 analysis and project plan.

B.4.2 Overall Goal of a PFHA

A cornerstone of the SSHAC process is the establishment of a clear specific goal for the process. This goal will guide SMEs in the evaluation and integration of sources of aleatory and epistemic uncertainty into a final estimate of the hazard. As stated by the SSHAC guidance:

> The fundamental goal of a SSHAC process is to conduct and fully document the activities of evaluation and integration, defined as:
>
> Evaluation: The consideration of the complete set of data, models, and methods proposed by the larger technical community that are relevant to the hazard analysis.
>
> Integration: Representing the center, body, and range of technically defensible interpretations in light of the evaluation process (i.e., informed by the assessments of existing data, models, and methods).

Achieving this goal requires that: (a) evaluations be conducted such that a complete understanding of the present state-of-knowledge of the technical community is reasonably achieved; and, (b) all parametric and modeling sources of uncertainty are identified and modeled in a technically sound, complete and transparent fashion.

B.4.3 Elements of a PFHA

Development of a PFHA will generally involve these steps:

1. Develop a project plan for the PFHA that is consistent with the level of analysis, which identifies the roles and responsibilities of the project staff, which defines the scope of the evaluation to be carried out, which identifies the PFHA products that should be generated, and which defines documentation and recordkeeping requirements.

2. Gather data to support the PFHA. Data to be collected includes:

    - Past regional and local precipitation and flood history, including paleofloods;

    - Site-specific data on topography, local sub-surface geology, local river configurations, natural and anthropogenic site drainage characteristics;

    - Facility-specific topographic data and locations of all SSCs; and

    - Literature search for prior analyses of extreme flood events, interpretations of meteorological conditions during these events, and studies of relevant environmental factors for each flood initiator.

3. Conduct an evaluation of the events and conditions that may lead to flooding site SSCs.

4. Identify existing models or develop other engineering models (hydrologic or hydraulic) applicable to the assessment of flood hazards of concern to the site.

5. Conduct Software Quality Assurance (SQA) of any models to be applied, using the criteria in DOE Order (O) 414.1D, *Quality Assurance*.

6. Identify sources of aleatory uncertainty and, using the above methods and models, develop a site-specific and phenomena-specific probabilistic aleatory model.

7. Identify and evaluate sources of epistemic uncertainty in the aleatory models and establish approaches to quantify those uncertainties.

8. Using the above models, quantify the frequency of occurrence and epistemic uncertainty in the flood hazards.

9. As needed, conduct feedback evaluations to assess the integrity of the epistemic uncertainty modeling.

Workshops and meetings are a useful tool, and a required element of a SSHAC Level 3 or 4 analysis, in developing a site-specific PFHA. They should be employed to assist in data gathering, the evaluation and modeling of sources of uncertainty, and the integration of model and parameter estimates into uncertainty distributions.

While the process for conducting hydrologic and hydraulic assessments is well-defined, estimating the frequency of events that may affect facilities is subject to considerable epistemic uncertainty that should be explicitly evaluated and included in the PFHA. As a consequence, a number of the steps in the PFHA deal with identification of sources of uncertainty and modeling epistemic uncertainties.

B.4.4 PFHA Modeling for Various Flood Initiators

The remaining sections of this appendix provide tools to select the appropriate PFHA model and to perform PFHAs. Approaches to modeling flood hazards are outlined for these five most common flood hazard initiators in the following subsections:

- Section B.5: Riverine floods, including effects of snowmelt and excessive precipitation;
- Section B.6: Upstream dam failure, including controlled releases during flood events;
- Section B.7: Storm surge due to hurricanes, tropical storms and tropical depressions;
- Section B.8: Flooding induced by seiches; and
- Section B.9: Flooding induced by tsunamis.

B.5 PFHA for Riverine Flooding

B.5.1 Overview

Riverine flooding may occur as a result of precipitation run-off, a rainstorm on terrain covered by an antecedent snowpack, or a rapidly melting snowpack due to unusually high temperatures in late-winter or early-spring. Precipitation run-off effects can be magnified if the ground is already saturated from previous rainstorms due to limited percolation.

On many river systems, dams have a significant and generally beneficial impact on downstream discharge during periods of normal flow, as well as during extreme precipitation and extreme wind meteorological events. For example, dams limited flooding damage during the extreme precipitation-induced Midwest floods of 1993 and 2011. However, operational errors in the management of a dam system, or breaching of upstream dams, can result in significant downstream flows and concomitant flood elevations. The contribution of uncontrolled and controlled releases from dams to riverine flooding, and riverine floods in the absence of dams, and the performance of levee systems are separately addressed in Section B.6.

A summary of frequency estimates for riverine-based hydrologic hazards is presented in U.S. Bureau of Reclamation (USBR) and other Federal guidelines;[7] see also *A High Resolution Coupled Riverine Flow, Tide, Wind, Wind Wave and Storm Surge Model for Southern Louisiana and Mississippi: Part I–Model Development and Validation*. (Bunya, et al., 2010) These references discuss several methodologies that extend hydrological methods to address low frequency flood hazard conditions. However, none of these methodologies addresses the issue of evaluating sources of modeling and parametric epistemic uncertainty.

B.5.2 PFHA for Riverine Flooding

Riverine flooding PFHAs are routinely performed as part of flood insurance studies, in which nearly a million river miles have been analyzed.[8] These river flood studies are conducted for the purpose of estimating the flood elevations that have a 0.01 and 0.002 annual frequency of exceedance (100-year and 500-year return period). However, none of these studies formally evaluate sources of epistemic modeling and parametric uncertainty.

As already stated, riverine flooding can be caused by a combination of precipitation, snowmelt, and unusually high spring temperature periods after a snowy winter. Moreover, flood levels can be increased by the effects of ice jams, debris, wind waves, and levee system failures.

A probabilistic approach for evaluating riverine flooding has the following components:

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[7] Swain, R.E., England, Jr, J.F., Bullard, K.L., Ruff, D.A., *Hydrologic Hazard Curve Estimating Procedures*, U.S. Department of the Interior Bureau of Reclamation, Research Report DSO-04-08, June 2004.

Office of Water Data Coordination, *Guidelines for Determining Flood Flow Frequency*, Interagency Advisory Committee on Water Data of the U.S. Department of Interior, Geological Survey, Office of Water Data Coordination, Bulletin #17 B of the Hydrology Subcommittee, March 1982.

Bureau of Reclamation, *Hydrologic Hazard Analysis*, Best Practices Chapters, U.S. Department of the Interior Bureau of Reclamation, April 2010.

[8] Godesky, M., "Presentation to the National Research Council Committee on Flood Maps," Federal Emergency Management Agency, Washington D.C., November 8, 2007.

1. Regional and site-specific data regarding river stream flows and precipitation.

2. Physically-based hydrologic models to evaluate river flows for a wide range of precipitation events and concurrent conditions (i.e., ice jamming, levee failures);

3. Physically-based hydraulic models to estimate flow depths and velocities for river flows of varying volumes, depths, and duration; taking into account the performance limitations of levee systems;

4. Probabilistic aleatory uncertainty model to estimate the frequency of occurrence of flood levels; and,

5. Probabilistic model of the sources of epistemic uncertainty in the physically-based models and the uncertainty in implementing the aleatory uncertainty model.

In regard to the first item above, limited site data may be extended by assessing available paleoflood information. Collection and treatment of paleoflood data are found in the USBR's *Hydrologic Hazard Analysis*, found at: https://www.usbr.gov/ssle/damsafety/risk/BestPractices/Chapters/II-2-20150612.pdf . Figure B-3 provides an overall schematic of the approach for physically-based riverine flood modeling.[9]

B.5.3 Riverine Flood Modeling

Modeling riverine flooding can be divided into three steps: (1) determination of precipitation, snowmelt, and temperature variation events, (2) hydrologic modeling of the watershed; and (3) hydraulic modeling to estimate flood surface elevations. A number of software tools are available to conduct the hydrologic and hydraulic modeling aspects of the analysis. Of these, the most extensively used is the Hydrologic Engineering Center (HEC) toolset developed by the U.S. Army Corp of Engineers (USACE). The following three models are part of the HEC toolset:

1. HEC-Hydrologic Monitoring System (HMS) is designed to simulate the precipitation-runoff processes of watershed systems.[10] This tool covers a wide range of applications from large watersheds to local urban basins.

2. HEC-River Analysis System (RAS) is a one-dimensional modeling tool used to estimate water-surface elevations.[11] The results of HEC-HMS may be used as input to an HEC-RAS calculation.

[9] Schaefer, M.G. (MGS Engineering Consultants, Inc.), "Stochastic Modeling of Extreme Floods on the American River at Folsom Dam (Appendix M: Sensitivity Analysis for the Stochastic Model of Extreme Floods for the American River at Folsom Dam), U.S. Army Corps of Engineers, Hydrologic Engineering Center, California, 2005.

[10] Scharffenberg, W.A., and Fleming, M.J., "Hydrologic Modeling System: HEC-HMS," CPD-74A, User's Manual, version 3.5, U.S. Army Corps of Engineers, Washington D.C., August 2010

[11] USACE, "HEC-RAS: River Analysis System," CPD-70, Hydrologic Engineering Center, U.S. Army Corps of Engineers, Washington D.C., January 2010.

3. FLOW-2D is a software tool that performs two-dimensional hydraulic modeling.[12] The use of two-dimensional modeling is often required for the solution of problems involving overland flow, or where more accurate estimates of flow velocities and direction are required.

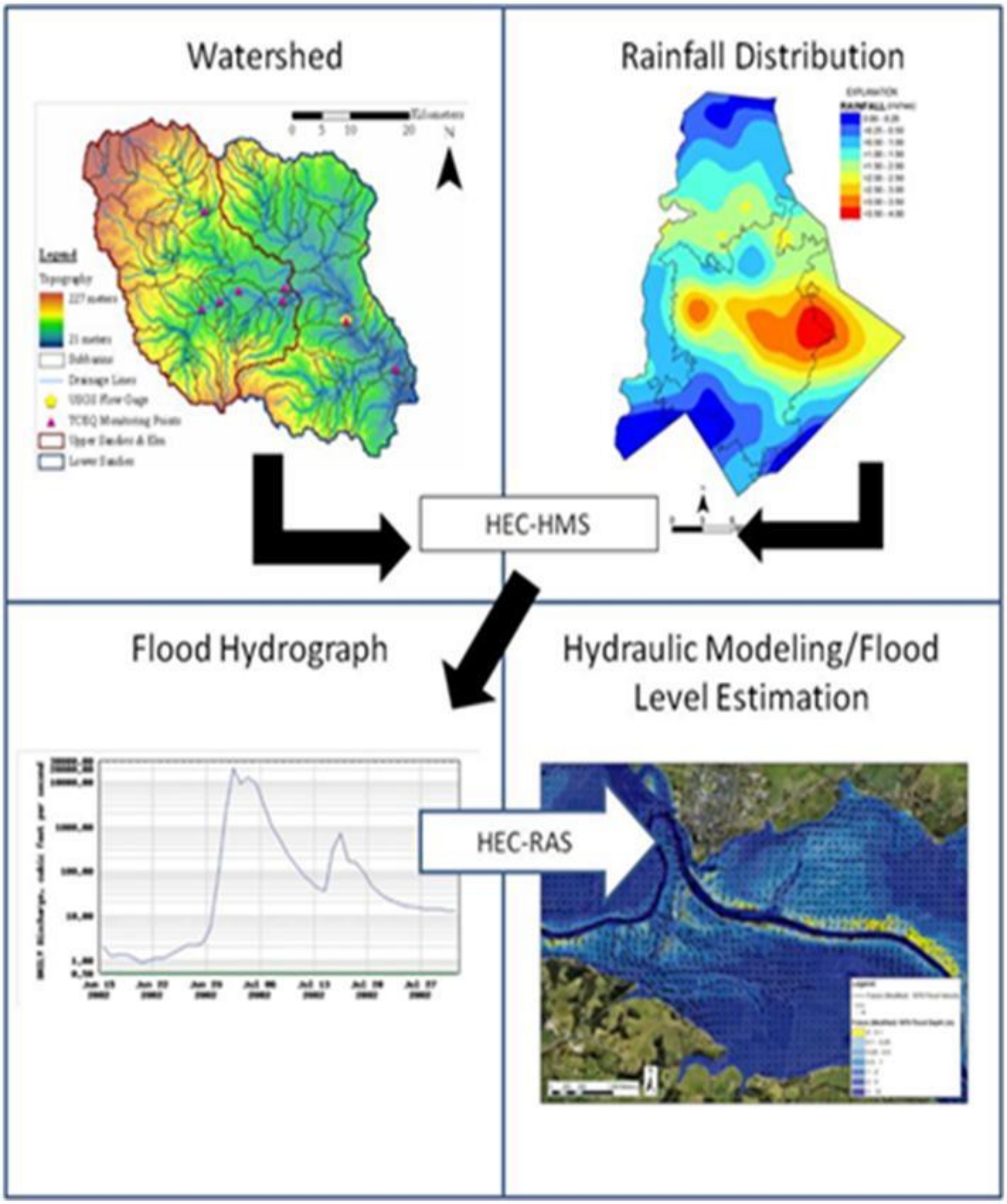

Figure B-3. Physically-Based Riverine Flood Modeling

[12] Hydronia "RiverFlo-2D: Two Dimensional Finite Element River Dynamics Model Argus One Edition: User's Guide, Release v3.1," Hydronia, LLC. Pembroke Pines, FL, March 2012.

B.5.4 Probabilistic Aleatory Uncertainty Model for Riverine Flooding

The analysis of flood frequencies requires two steps. The first step estimates the frequency of exceedance of river flows, represented by $\nu(q)$, and the second step estimates the flood elevations.

Step 1: The frequency of exceedance distribution can be expressed as:

$$\nu(q) = \sum \int \nu(p)\, f(s(t,x)\, f(ac) dp$$

(Equation B-1)

Where,

$\nu(p) =$ frequency of occurrence of total precipitation

$f(s(t,x)) =$ probability mass function on the temporal and spatial pattern of precipitation events

$f(ac) =$ probability mass function on antecedent conditions

The expression in Equation B-1 is a generalization of the stochastic flood model.[13] The expression includes all important factors influencing river flows. Factors not accounted for include snowmelt and temperature variations.

Step 2: The assessment of the frequency of exceedance of river flood elevations, $\lambda(E>e)$, can be expressed as:

$$\lambda(E > e) = \int \nu(q) P(E > e \mid q) dq$$

(Equation B-2)

Where,

$\nu(q) =$ frequency of occurrence of river flows, q

$P(E > e \mid q) =$ probability distribution on the river flood elevation, e, being exceeded, given a river flow, q.

[13] Schaefer, M.G., “Regional Analyses of Precipitation Annual Maxima in Washington State,” Water Resources Research, 26(1), 119-131, 1990.

Fontaine, T.A. and K.W. Potter, “Estimating Probabilities of Extreme Rainfalls,” American Society of Civil Engineers, Journal of Hydraulic Engineering, 115(11), 1562-1575, 1989.

MGS Engineering Consultants, Inc., “Stochastic Modeling of Extreme Floods on the American River at Folsom Dam, Appendix B-Precipitation Magnitude-Frequency Characteristics for the American River Watershed,” prepared for the U.S. Army Corps of Engineers, Institute for Water Resources, Hydrologic Engineering Center, California, 2005.

MGS Engineering Consultants, Inc., “Stochastic Event Flood Model Improvements and Extreme Storm Analyses for A.R. Bowman Watershed,” prepared for U.S. Department of the Interior, Bureau of Reclamation, Dam Safety Office, USBR Report No. DSO-03-02, 2003.

The solutions to Equations B-1 and B-2 can be determined in a number of ways. For instance, the "Stochastic Event Flood Model Improvements and Extreme Storm Analyses for A.R. Bowman Watershed" uses Monte Carlo and other simulation methods to estimate the storm hydrographs and river inflows (MGS Engineering Consultants, Inc., 2003). Similarly, Equation B-2 can be evaluated using simulation or by direct integration.

Although the aleatory variability in estimated flood elevations (i.e., P (E>e|q), is often not considered, studies indicate there is considerable variability in flood elevations that have an annual frequency of exceedance of 0.01; this variability can be as much as 2.5 feet, but averages about 1.0 foot.[14] An illustration of this variability is shown in Figure B-4.

Paleoflood data is extracted from past significant floods by studying the geomorphology, sedimentology and geobotany of the affected area. Techniques using paleostage indicators and paleodischarge data may also be used to inform observed data and increase the ability to extrapolate to desired low annual recurrence intervals.

By using a stochastic sampling process to generate random, or synthetic, storms, watershed features and combined meteorological events, more realistic estimates of flood frequency can be established. In this approach, the rainfall duration/rate and watershed parameters, which are included in HEC-RAS output, are modeled as stochastic variables. To perform a probabilistic analysis, a large number of synthetic storms are created on the watershed and site and combined with a range of probabilistically-distributed antecedent and watershed conditions as shown in Figure B-5. A flood frequency curve is then established by statistical sampling of a large number of scenarios and ranking resulting flood level results, as shown in Figure B-6. An example of a peak discharge exceedance curve is also illustrated in Figure B-6.

[14] National Research Council of the National Academies, "Mapping the Zone, Improving Flood Map Accuracy," National Academy Press, Washington, D.C., 2009; Freeman, G.E., R.R. Copeland, M.A. Cowan, "Uncertainty in Stage-Discharge Relationships," Stochastic Hydraulics 1996 Proceedings of the Seventh IAHR International Symposium on Stochastic Hydraulics, Mackay, Queensland, Australia, July 29-31, 1996, A.A. Balkema, Rotterdam, Netherlands.

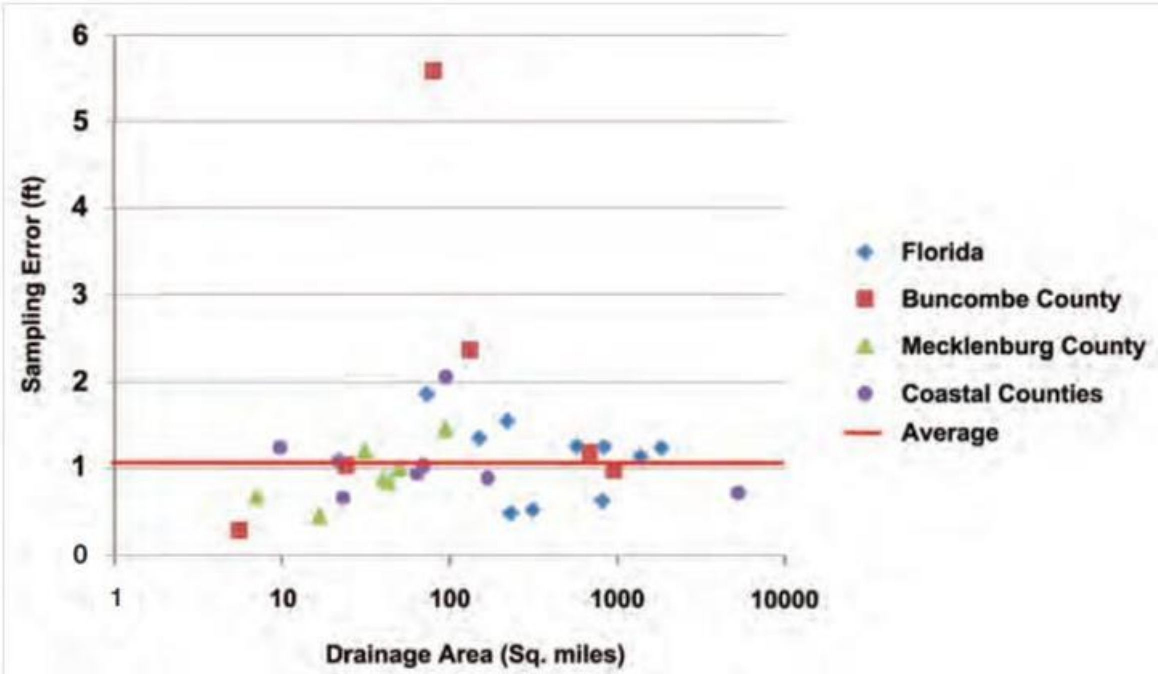

Figure B-4. Variability of Flood Stages with an Annual Frequency of Exceedance of 0.0115

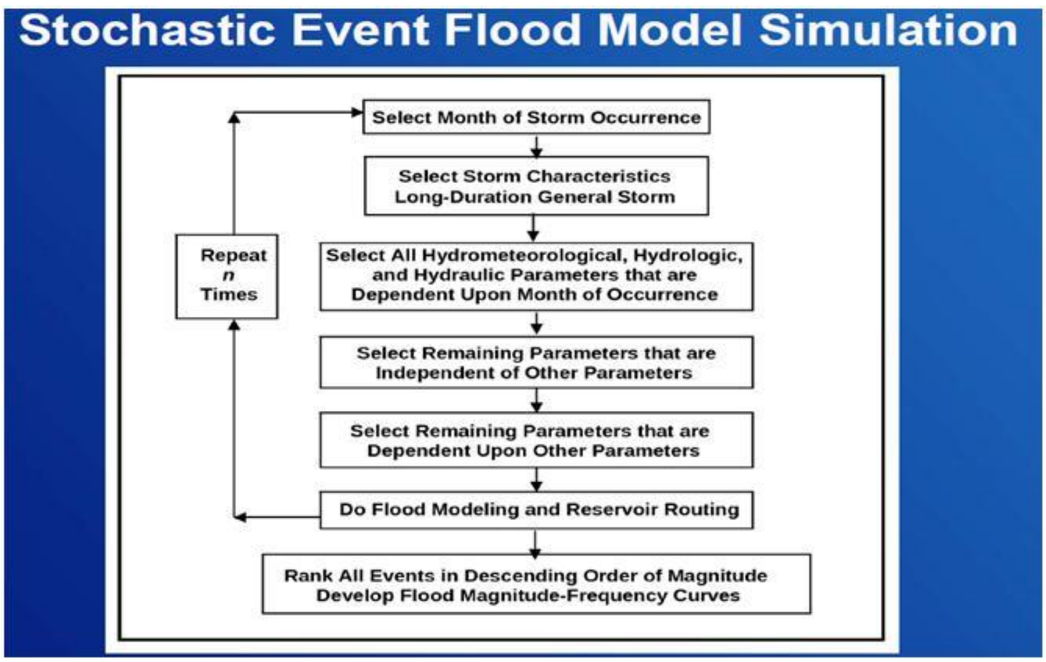

Figure B-5. Stochastic Process for a River Flood Model Simulation[16]

[15] Id. note 11.

[16] Schaefer, M.G. (MGS Engineering Consultants, Inc.), "Stochastic Modeling of Extreme Floods on the American River at Folsom Dam (Appendix M: Sensitivity Analysis for the Stochastic Model of Extreme Floods for the American River at Folsom Dam), U.S. Army Corps of Engineers, Hydrologic Engineering Center, California, 2005.

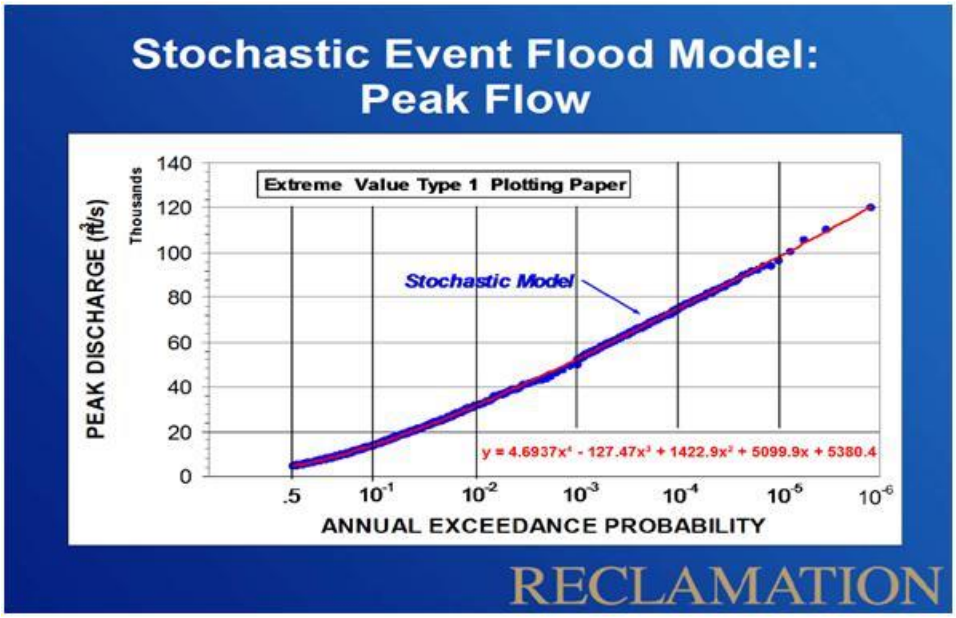

Figure B-6. Example of Flood Frequency and Peak Discharge Exceedance Probability[17]

B.5.5 Probabilistic Epistemic Uncertainty Model for Riverine Modeling

Epistemic uncertainty in estimates of riverine flooding arises from a number of sources.[18] These sources include:

- Model uncertainty in the hydrologic and hydraulic modeling of storm events and river hydraulics;
- Parametric uncertainty in the estimate of the parameters of each element of the aleatory model (see Equations B-1 and B-2);
- Model uncertainty in frequency distribution of extreme precipitation; and
- Alternative interpretation of meteorological regions and storms for resampling.

Uncertainty in estimating the 100-year flood zone was estimated in "Mapping the Zone, Improving Flood Map Accuracy"[19] and the Freeman, et al. proceedings. The USACE has also recognized the uncertainty in flood stage estimates.

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[17] Ibid.

[18] Freeman, G.E., R.R. Copeland, M.A. Cowan, "Uncertainty in Stage-Discharge Relationships," Stochastic Hydraulics 1996, Proceedings of the Seventh IAHR International Symposium on Stochastic Hydraulics, Mackay, Queensland, Australia, July 29-31, 1996, A.A. Balkema, Rotterdam, Netherlands.

[19] Id. note 11.

B.6 PFHA for Flooding Associated with Controlled and Uncontrolled Releases from Dams

B.6.1 Overview

The potential failure of an upstream dam, due to operator error or structural damage, can significantly affect the flooding on a river. In certain circumstances, events at downstream dams can have an upstream impact, such as loss of a reservoir that provides a source of cooling water. Typically, three categories of dam failure are considered in dam risk studies: (1) hydrologic-induced; (2) seismically-induced; and (3) so-called "sunny day" failures that occur due to intrinsic factors and forces. Other events (i.e., high winds, ice loading) have been caused incidents at dams, but these are beyond the scope of this appendix.

During major hydrologic-induced events, releases from upstream dams can be a significant factor with respect to flooding. Downstream effects of dam operations depend on a number of factors, including: (1) operational characteristics of upstream dams; (2) river management for large systems that have multiple dams; (3) antecedent state of reservoirs and the watershed prior to the event that initiated high inflows; (4) potential for uncontrolled releases that may occur as a result of operational errors; (5) failure of reservoir control systems such as spillway gates; and (6) dam breaches due to floods, seismic events, upstream dam failures, and intrinsic events.

B.6.2 Evaluation of Events Due to Controlled Releases of Dam Operations and Events Due to Uncontrolled Releases

Flooding events at the Fort Calhoun and Cooper Nuclear Plants in 2011 resulted from the combined effect of: (a) extreme spring precipitation, (b) heavy antecedent snow cover, and (c) early and rapid snowmelt due to milder than normal spring temperatures. This major hydrological event occurred in the Missouri River watershed and in the Midwest in general. Although a flood ensued, the flows on the Missouri River were planned and controlled releases from upstream dams on the river and its tributaries. Thus, even at "planned" release levels, these discharges have the potential to create significant flood challenges at a site. The evaluation of controlled releases is a part of the hydrologic analysis performed for the river system, as previously described in Section B.5. However, in addition to controlled release evaluations, the reliability of the dam system should be evaluated as part of the hydrologic modeling to assess the probability of uncontrolled releases.

Dam failures, resulting from natural event insults or from operational errors, or from failure of systems (e.g., gates) that are intended to control the reservoir can result in uncontrolled releases and downstream flooding. These events can be more severe than controlled releases since they may:

- Result in larger pathways through which flows are released, as in the case of a dam breach;
- May occur without warning, as in the case of a dam breach caused by an earthquake; and
- May drain the entire reservoir.

Uncontrolled releases due to operator error and/or major structural failures may occur during normal, non-flood conditions or during major hydrologic events. To evaluate the likelihood and severity of flooding that could occur, a probabilistic risk analysis for the dam system is required[20] (See Bowles, 2003).

B.6.3 Events and Effects of Dam Failures on Downstream Releases

B.6.3.1 Overview

Dams are constructed for a number of different purposes: (1) hydropower generation, (2) water supply, (3) flood control, (4) recreation, (5) navigation, and (6) irrigation. Many dams serve more than a single purpose. The potential downstream effect of dam operations depends on the role the dam is assigned in river level management. For instance, hydropower generation dams and flood control dams are operated differently under normal conditions. Reservoir levels at hydropower generation facilities are typically kept at high levels year-round to ensure consistent power generation. Gate systems at these dams are designed and operated to control flows throughout the year and to maintain reservoir levels within a narrow depth range. Conversely, flood control dams are operated in response to seasonal precipitation patterns. For example, prior to the flood season, reservoir levels are lowered to provide room for storage in anticipation of spring precipitation and snowmelt.

Several mechanisms can contribute to dam failure, as shown in Figure B-7, and the manner in which the dam failure occurs can influence the magnitude of the event. Figure B-7 also shows the causes of dam failures during the period of 1975-2001. Flood/overtopping is by far the most likely failure cause.

A number of events may contribute to controlled or uncontrolled releases from upstream dams, which are summarized in Table B-3 and discussed further in this subsection.

Table B-3. Events Resulting in Controlled or Uncontrolled Releases from Dams

Initiating Event	Controlled/Uncontrolled Releases	Description
Normal Flow (Non-flood) Conditions	Controlled/Uncontrolled	During normal flow conditions, uncontrolled releases may occur as a result of operator error or the failure/operational error of gate level control systems. These releases are controlled in the sense that they are defined by the systems where the releases are made (e.g., over the spillway). However, they are uncontrolled in the sense that the releases were not intentional.

[20] A probabilistic analysis would be required only if it has been determined that uncontrolled releases associated with operational errors or structural failure of gates could result in flows high enough to cause flooding at the plant.

Initiating Event	Controlled/Uncontrolled Releases	Description
Hydrological Events - Extreme Precipitation, Snowmelt and Runoff	Controlled	During extreme precipitation events, exacerbated by snowmelt and runoff, releases that occur downstream will vary depending on the type of spillway system, river management standards, etc. For un-gated spillways, flows are defined by the spillway size and the size of the flood event, the latter measured by flow depth over the spillway. For gated spillways, releases are established according to the project rule curve and operating procedures. These controlled releases are made to appropriately manage reservoir levels as well as the flow rates downstream.
	Uncontrolled	Uncontrolled releases may occur as a result of dam breaching due to overtopping and erosion, instability, internal erosion and piping of embankments, or failure of gate systems.
Seismic Events - Earthquakes	Controlled	As a result of a seismic event, one or more SSCs of a dam system may be damaged or fail; but the dam itself has not failed. In this case, there may be a number of different scenarios, some resulting in controlled and/or uncontrolled releases. Controlled releases may occur if there is a need to immediately lower the reservoir sufficiently enough to adequately reduce the hydrostatic load on the dam and other reservoir retaining structures. These releases may be immediate and necessary to prevent breaching of the dam and an uncontrolled release of the reservoir. The size of these releases will depend on the reservoir level at the time and the discharge capacity of gates, and the availability and outflow capacity of low level outlets at the dam.
	Uncontrolled	Uncontrolled releases may occur as a result of a breach of the dam, or failure of gate systems or outlets. The breach of a dam may be immediate, as a result of seismically-initiated damage, or it may develop later.

Initiating Event	Controlled/Uncontrolled Releases	Description
Intrinsic Events/Factors not related to NPHs	Controlled	In the event a dam is experiencing stability or other issues such as seepage, internal erosion of piping, controlled releases may be made in order to reduce the hydrostatic loading on the dam. Potentially, these releases could be significant and lead to downstream flooding.
	Uncontrolled	Dam failures and uncontrolled releases can occur during normal, "sunny day" operations. The failure modes differ, depending on the type of dam. The warning time for these failures will also vary depending on the nature of the failure mode and the degree of monitoring and inspection that is done at the dam.

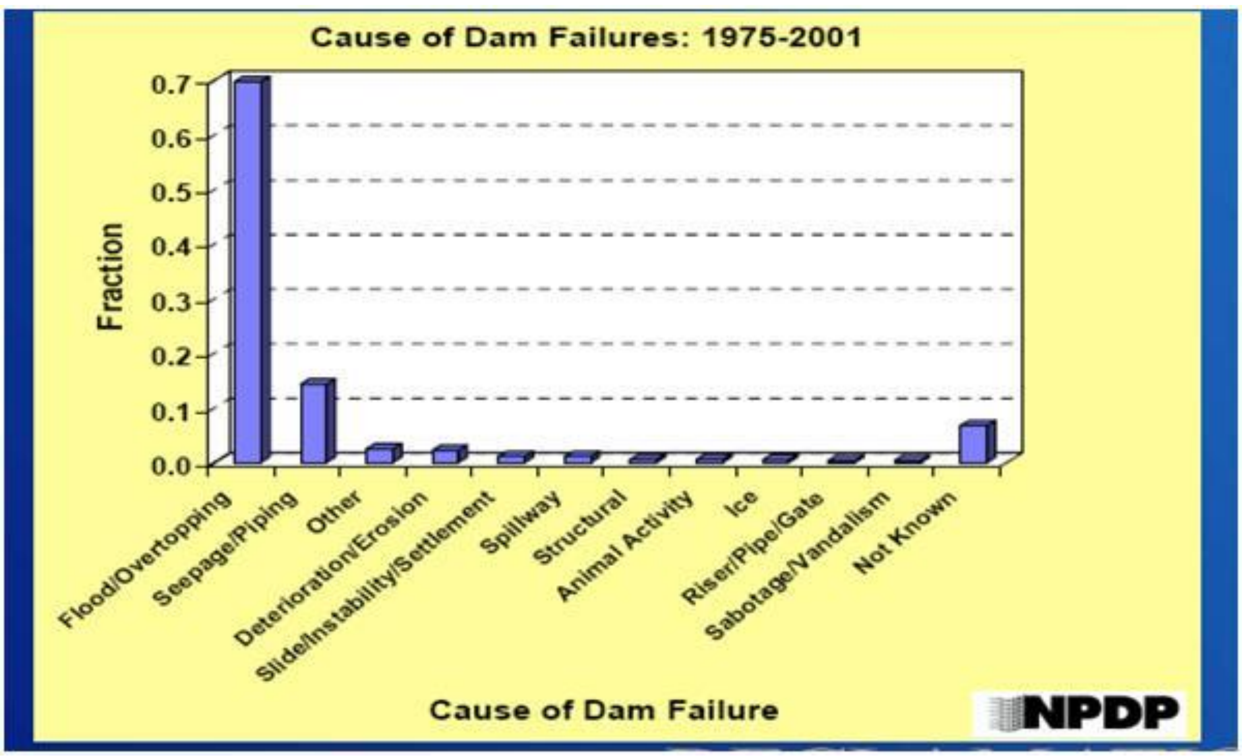

Figure B-7. Distribution of the Causes of Dam Failures in the U.S. (1975-2001)

B.6.3.2 Normal Flow (Non-Flood) Conditions and Dam Operations

Operational events that lead to downstream releases at dams are controlled by the project rule-curve, release schedules, and environmental commitments to local regulators. These releases are typically within riverbanks and are defined by operating procedures. Under certain circumstances, when large releases are necessary, dam owners may have release limits that legally require them to notify downstream stakeholders. In this circumstance, the magnitude of these releases is known prior to making the decision to discharge, and the travel time of flows and water levels is also well understood. Releases in this category are unlikely to pose a hazard to DOE nuclear facilities and do not have to be considered in the PFHA.

During normal flow conditions, operational accidents can cause uncontrolled releases from a dam. These releases may occur as a result of operator error, the failure of structures (e.g., spillway gate), or the failure of control systems. To assess the probability of occurrence of these events, a probabilistic risk analysis would be required. However, before such a risk analysis is undertaken, a deterministic assessment should be performed to establish the maximum size of releases that could potentially occur and whether such releases would lead to unacceptable flood levels. Limits on the size and rate of such releases will depend on the size of spillway gates and the control logic of their operation.

B.6.3.3 Hydrologic Events – Extreme Precipitation and Dam Failure

Given the occurrence of a major hydrologic event, a number of potential dam failure modes may cause an uncontrolled release of water from a reservoir. Significant controlled releases from an upstream dam may add to the effects of an extreme precipitation event. Controlled and uncontrolled releases may occur as described below.

Controlled Releases during a major hydrologic event may be caused by one or more of the following events:

- Spill over non-gated spillways;
- Spill over gated spillways;
- Release through low-level outlets;
- Release through the powerhouse; and
- Spill over the dam crest due to overtopping.

Depending on the type of dam and the condition of the downstream channel and foundation, a dam may be able to withstand considerable overtopping without resulting in dam failure (Gibson Dam failure, 1964).

Uncontrolled Releases during a major hydrologic event may be caused by:

- Structural failure of dam gates due to the hydrostatic loading and debris;
- Operational errors leading to gate failure;
- Structural failure of the dam prior to overtopping (e.g., seepage and piping of an embankment; instability of the embankment or concrete dam section); and
- Dam overtopping resulting in erosion of the dam crest and eventual breaching of the embankment.

Extreme hydrologic events may be caused by simultaneous extreme precipitation, snowmelt, and runoff. When the combined water volume of the extreme precipitation, snowmelt, and runoff exceeds reservoir storage and the outflow capacity of the emergency spillway and other available outlets, the structural and operational integrity of the dam system may be compromised. From a hydrological perspective, dams can fail due to overtopping of unprotected portions of the dam, leading to an erosion of the dam face, or piping erosion failure resulting in deterioration of the embankment. Overtopping of the dam may also result in erosion of the dam's foundation.

The susceptibility of dams to fail due to hydrological forces is based on the type of dam, the reservoir water level with respect to its design, condition of the dam, and operational strategies. As discussed earlier, operational strategies can create intentional downstream flooding in a controlled manner. Hydrological failures are typically characterized by some warning, as flood events typically develop over a time-scale that is well-understood by dam owners and operators. As such, dam operators should take time to inform downstream nuclear sites: (a) when reservoirs reach critical levels, (b) when operational actions to significantly open flood gates are planned, or (c) when signs of distress are apparent. The time that may be available is dependent on the watershed and the distance to downstream DOE facilities. This time may be used by the downstream plants to plan for an event by installing temporary protective barriers, performing drills or taking other emergency preparedness actions based on the perceived threat.

B.6.3.4 Dam Failures Caused by Seismic Events

Historically, dams have performed well during seismic events. Table B-7 does not show any dam failures due to seismic events for the 1975-2001period. In a limited number of instances, earthquake-induced ground-shaking events have led to dam failure and uncontrolled release of the reservoir. One such case is the failure of Sheffield Dam in southern California during the 1925 Santa Barbara, California, earthquake (Seed, et al., 1969). The dam failed due to liquefaction in the foundation. Ground shaking during the 1971 San Fernando, California, earthquake resulted in a failure of the upstream slope of the Lower Van Norman Dam embankment. However, due to the reservoir restrictions that were in place at the time and the ability to lower the reservoir immediately after the earthquake through a controlled release, the failed embankment did not overtop or become unstable. In a number of earthquakes worldwide, dams have performed well despite strong ground shaking. For instance, dams have performed well during recent major earthquakes in China, Japan, Indonesia, and South America. Despite the generally satisfactory performance of dams during seismic events, seismic risk studies of dams indicate that the risk of failure and uncontrolled release of the reservoir can be high.

The potential seismic modes of dam failure vary depending on the type of dam, its foundation, and operational characteristics. A seismic event can lead to the immediate or near-immediate failure of a dam near the epicenter. Even when immediate failure does not occur, a dam may be damaged by ground-shaking, with failure developing some time later. In the immediate failure case, prior warning at a plant downstream may be limited to the travel time of the flood wave. In the delayed failure case, additional warning time may be available.

For cases in which upstream dams and facilities are located in the same vicinity, the same seismic event may adversely affect the performance of both and inflict damage on infrastructure such as roads, bridges, and power plants. To assess the seismic risk for these cases, special consideration should be given to earthquake and ground motion correlations.[21] In these circumstances, a seismic event could lead to a simultaneous loss of offsite power, damage to the plant, and an upstream dam failure.

While the frequency of seismically-induced dam failures should be extremely low based on worldwide experience, consequences may be serious. Because of these potential consequences, the PFHA for a DOE facility located near a dam should evaluate flooding caused by a seismically-induced dam failure.

B.6.3.5 Sunny Day Failures

Experience from statistical evaluations of "sunny day" dam failures, as well as the results of probabilistic risk studies for dams, indicates that these events may be significant contributors to the likelihood of an uncontrolled release. Compared to failures resulting from severe weather events, "sunny day" failures usually provide less warning time to initiate mitigating actions, since precursor events are not apparent. "Sunny day" failures may be caused by: (1) design flaws, (2) ineffective defensive measures in the dam design, (3) faulty construction, (4) aging, and (5) poor preventive maintenance. There is some evidence to suggest that past experience may not be indicative of future performance.

B.6.4 Evaluation of Dam Break Flooding

Selection of an appropriate methodology to estimate peak flood discharge due to a dam break is needed for a realistic assessment of downstream flood risk. Conservative deterministic calculations may assume instantaneous dam collapse due to a catastrophic structural failure. However, these results may significantly overstate potential downstream flooding. In performing realistic risk-informed performance-based assessments, the analyst should consider the dam failure mechanism, the type of dam, and realistic representations of dam rupture dynamics and flows using fault tree analysis and Failure Mode and Effects Analysis (FMEA)**.** In 2013, the Nuclear Regulatory Commission (NRC) issued Interim Staff Guidance (ISG) on dam failures as a part of its response to the Fukushima accident in Japan (See NRC, 2013). In treating uncertainty, it may be helpful to invoke the SSHAC process: (a) investigate the impact of alternative dam failure discharge models, (b) solicit peer-review, and (c) employ the expert elicitation process to determine the most representative analysis.

The economic losses and human tragedies associated with dam failures have spurred development of risk-informed techniques for identifying dam failure modes and estimating the consequences of dam failure. These techniques are similar to event tree modeling methods. Dam breach evaluations often assume that dam failures are complete, which is a very conservative assumption. However, evidence for partial failures in the dam data bases is somewhat limited. For this reason, downstream flood impacts should be estimated based on a range of dam failure modes.

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[21] Park, J., Bazzurro, J.P., & Baker, J.W., "Modeling Spatial Correlation of Ground Motion Intensity Measures for Regional Seismic Hazard and Portfolio Loss Estimation," 10th International Conference on Application of Statistic and Probability in Civil Engineering (ICASP10), Tokyo, Japan, 2007.

URS Corporation and Jack R. Benjamin & Associates, Inc., "Delta Risk Management Strategy, Phase 1: Risk Analysis Report," prepared for the California Department of Water Resources, 2009.

McCann, Jr., M.W., "Seismic Risk of a Co-Located Portfolio of Dams – Effects of Correlation and Uncertainty," paper presented at the 3rd International Week on Risk Analysis, Dam Safety, Dam Security, and Critical Infrastructure Management, Polytechnic University of Valencia, Valencia, Spain, 2011.

B.6.5 Probabilistic Aleatory Uncertainty Model for Dam Failures and Flooding

For DOE sites in the vicinity of rivers where dams affect downstream discharge, the frequency of flooding can be denoted by the following expressions:

$$\lambda(E>e)=\lambda_S(E>e)+\lambda_{SD}(E>e)+\lambda_{NO}(E>e)+\lambda_H(E>e) \quad \text{(Equation B-3)}$$

and

$$\lambda_H(E>e)=\lambda_{HF}(E>e)+\lambda_{HO}(E>e) \quad \text{(Equation B-4)}$$

Where,

$\lambda_S(E>e)=$ frequency distribution on flood elevation (E) exceeding a level, e, due to seismic dam failure.

$\lambda_{SD}(E>e)=$ frequency distribution on flood elevation (E) exceeding a level, e, due to sunny day dam failure.

$\lambda_{NO}(E>e)=$ frequency distribution on flood elevation (E) exceeding a level, e, due to releases from the dam as a result of faulty operation or structural failure of gate systems (not dam failures) during normal operations.

$\lambda_H(E>e)=$ frequency distribution on flood elevation (E) exceeding a level, e, due to hydrologic events.

$\lambda_{HF}(E>e)=$ frequency distribution on flood elevation (E) exceeding a level, e, due to hydrologic dam failure.

$\lambda_{HO}(E>e)=$ frequency distribution on flood elevation (E) exceeding a level, e, due to releases during hydrologic events not involving a dam failure.

These terms represent flood hazard curves at a site that are associated with different events. Each hazard curve is based on a probabilistic risk analysis. A schematic illustration of these flood hazard results is shown in Figure B-8.

An assessment of the frequency of dam failure should be performed on an SDC- and PDC-specific basis. An evaluation of the frequency of dam failure should consider the natural and anthropogenic structures that are involved in retaining control of a reservoir, a dam's construction materials, dam type, all external and intrinsic hazards the dam may be exposed to, dam operating characteristics and current condition of the dam. A methodology for evaluating the hydrologic, internal and earthquake failure probability of specific dams is contained in "Preliminary Safety Evaluation of Existing Dams" (McCann, et al., 1985). That reference also includes guidance for establishing risks of dams with degraded features.

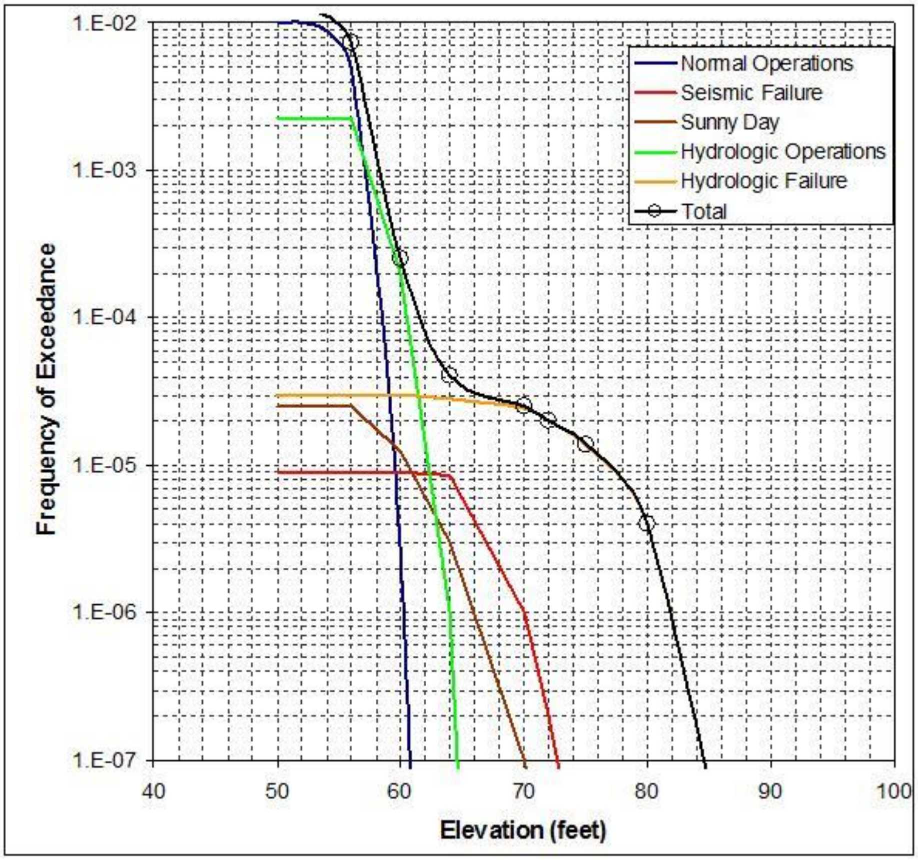

Figure B-8. Generation of a Mean Composite Flood Hazard Curve Associated with Dam Operations and Failure

B.6.6 Probabilistic Epistemic Uncertainty Model for Dam Failures and Flooding

The following epistemic uncertainties should be evaluated for the performance of dams: (1) hazards the dam is exposed to (e.g., seismic and extreme precipitation hydrological events); (2) structural reliability of the dam, its appurtenant structures and systems; and (3) modeling controlled and uncontrolled releases.

B.7 PFHA for Storm Surge Due to Hurricanes, Tropical Storms and Tropical Depressions

B.7.1 Overview

The assessment of storm surge flooding effects caused by extreme winds from hurricanes, tropical storms and tropical depressions has received considerable attention, given the significant risk these extreme meteorological phenomena pose for the Atlantic and Gulf Coast regions of the United States. Hurricane Katrina in 2005 and Hurricane Sandy in 2012 produced extraordinary storm surges, and serve as a reminder that such events, though infrequent, do occur. Storm surges can be amplified by astronomical conditions when hurricanes occur during the spring tide, as in the case of Hurricane Sandy. Pacific Coast hurricanes are rare and of smaller intensity compared to Atlantic and Gulf Coast hurricanes and thus do not pose significant storm surge risks to DOE facilities.

NRC has issued ISG on hurricane storm surge, seiche and tsunami (JLD-ISG-2012-06, Revision 0, 2013). NRC's NUREG/CR-7134, *The Estimation of Very Low Probability Hurricane Storm Surges for Design and Licensing of Nuclear Power Plants in Coastal Areas* (2012), looks at estimating hurricane storm surge events with low probabilities of occurrence and includes a first-order evaluation of the uncertainty in surge estimates.

B.7.2 Probabilistic Storm Surge Analysis

Modeling storm surge flood hazards resulting from hurricanes and tropical storms is generally carried out using the joint probability method (JPM).[22] This method, as applied in the Interagency Performance Evaluation Team (IPET) study for New Orleans, LA, is called the JPM-Optimum Sampling (OS) method,[23] which is well-suited to the probabilistic analysis of extreme storm surges.

A probabilistic approach for modeling hurricane storm surge has the following components:

- Probabilistic aleatory model to estimate the frequency of potential storm surge hazards;
- Physically-based models to estimate storm surge, including wave action;
- Mapping of the local terrain heights, shoreline orientation, and water body bathysphere depths to estimate storm surge elevations; and
- Probabilistic model of the sources of epistemic uncertainty in physically-based models and the uncertainty in estimating the parameters of the aleatory model.

Figure B-9 shows an overall schematic of the approach for physically-based modeling of storm surge and wave effects during hurricanes and tropical storms. This schematic was developed using the Advanced Circulation Model (ADCIRC) and Steady State Spectral Wave (STWAVE) codes used in the IPET post-Katrina evaluation for New Orleans, Louisiana.

Physically based modeling of storm surge has three basic components:

- **Wind-Field Model** –defines the storm track and time-varying wind field parameters in the PBL. This model defines a time series of wind fields and atmospheric pressure fields that drive surge and wave models;
- **Surge Model** - predicts the hydrodynamic response of the ocean to the wind and pressure field driving functions. The surge model predicts ocean or gulf surge levels and takes into account neap and spring tide effects; and

[22] Interim Staff Guidance, Japan Lessons-Learned Project Directorate "Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment, US Nuclear Regulatory Commission, JLD-ISG-2012-06, January 2013. G.R. Toro, D.T. Resio, D Divoky, A.W. Niedoroda, C. Reed, "Efficient Joint Probability Methods for Hurricane Surge Frequency Analysis", Ocean Engineering, Vol. 37(1), 125-134, 2010.

[23] U.S. Army Corps of Engineers (USACE). "Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System." Final Report of the Interagency Performance Evaluation Task Force. Volume VIII - Engineering and Operational Risk and Reliability Analysis, U.S. Army Corps of Engineers, 2007.

- **Wave Model** –estimates the breaking waves near the coast due to frictional forces and the resulting wave forcing, wave setup and run-up that occur onshore.

B.7.3 Hurricane, Tropical Storm and Tropical Depression Storm Surge Modeling

A number of different models are used to evaluate the storm surge and near-shore wave action caused by a hurricane. The models include:

1. **Sea, Lake and Overland Surges from Hurricanes (SLOSH)** – This code, developed by NOAA, may be used for wind fields typical of extra-tropical storms (JLD-ISG-2012-06, 2012). Additional information and details of SLOSH can be obtained by visiting http://slosh.nws.noaa.gov/sloshPub/. This code is also used by FEMA to develop flood levels in support of its flood insurance program.

   SLOSH can be run with many pre-generated grids typical of local bathyspheres. Figure B-10 presents observed surge heights versus surge height forecasts by the SLOSH model for thirteen storms in nine basins. Although SLOSH can produce reasonable surge estimates, it is limited by its grid size. A total of 570 tide gage, staff gage, and high water mark observations are shown with the corresponding SLOSH forecast. Generally, the aleatory uncertainty of the model is within ± 20 percent for significant surge heights (Jelesnianski, et al., 1992).

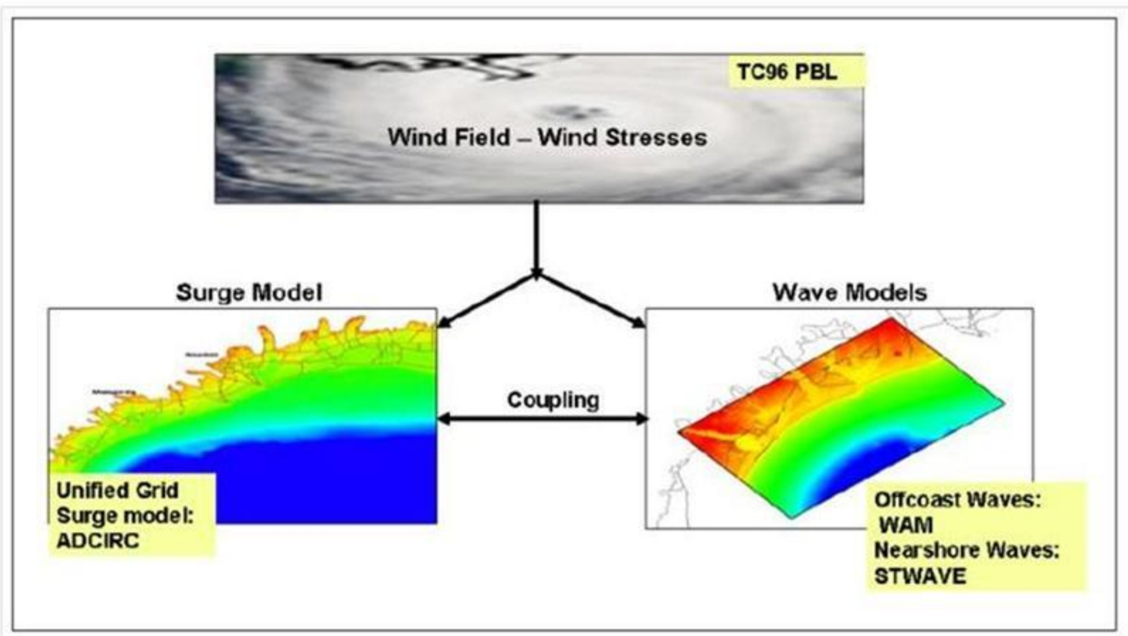

Figure B-9. Schematic for Modeling Hurricane Storm Surge and Wind Waves

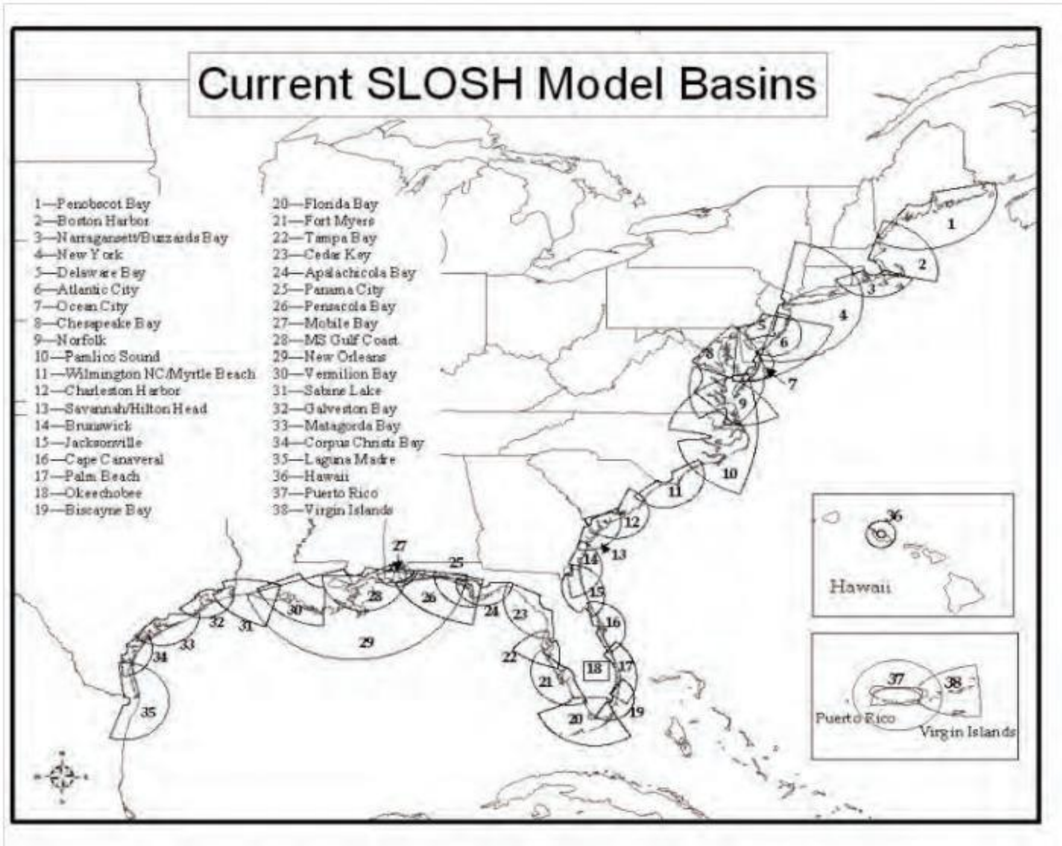

Figure B-10. Current SLOSH Model Basins[24]

2. **ADvanced CIRCulation (ADCIRC)**[25] – ADCIRC is a two-dimensional, depth-integrated, barotropic time-dependent long wave, hydrodynamic circulation model (see http://adcirc.org), used as part of the IPET evaluation of storm surge in New Orleans and coastal Louisiana.[26] It is a numerically intensive code and thus is very time-consuming to run even on mainframe computers. ADCIRC is typically used to estimate surge elevations where more precision is required. Accordingly, some organizations have supplemented SLOSH analyses with the more detailed ADCIRC program. ADCIRC coastal circulation and storm surge models comprise a system of computer programs for solving time-dependent, free-surface circulation and transport problems in either two- or three-dimensions. These programs utilize the finite element method in space allowing the use of highly flexible, unstructured grids. Typical ADCIRC applications have included modeling tides and wind-driven circulation, and analysis of hurricane storm surge and flooding. The IPET assessing the New Orleans

[24] G.R. Toro, D.T. Resio, D Divoky, A.W. Niedoroda, C. Reed, "Efficient Joint Probability Methods for Hurricane Surge Frequency Analysis", Ocean Engineering, Vol. 37(1), 125-134, 2010.

[25] This description of ADCIRC is drawn, in part, from the IPET report (USACE, 2007).

[26] Westerink, J.J., Luettich, R.A., Feyen, J.C., Atkinson, J.H., Dawson, C., Roberts, H.J., Powell, M.D., Dunion, J.D., Kubatko, E.J., and Pourtaheri, H., "A Basin to Channel Scale Unstructured Grid Hurricane Storm Surge Model Applied to Southern Louisiana," Monthly Weather Review, American Meteorological Society, 136(3):833–864. doi:10.1175/2007MWR1946.1, 2008.

Applied Research Associates, Inc., "HAZUS-MH Advanced Severe Storm Coastal Risk Assessment Methodology," prepared for National Institute of Building Sciences, 2006.

levee system used SLOSH analyses to identify candidate storms of interest and ADCIRC for the development of the peak surge estimate. (USACE, 2007)

ADCIRC has also been used in conjunction with a spectral wave code, STWAVE (Smith, et al., 2001). The water level and wind field from ADCIRC are provided in the appropriate form for STWAVE applications. STWAVE computes the high-frequency wave field, including the wave radiation stress associated with the storm (i.e., wind) and simulated water level. This wave radiation stress is then passed to the next ADCIRC simulation as an additional forcing term, in conjunction with the storm wind and pressure field. This model communication achieves a first-order coupling of the non-linear interaction between the wind-induced storm-surge and the wind-driven higher frequency wave.

3. **WAVEWATCH-III** – WAVEWATCH-III is a code for predicting deep-water wave parameters including significant wave height, mean wave period, dominant wave period, mean wave direction and surface stress acting at the air and water interface. It is the third generation wave model for computing deep-water wave spectra. It was developed by the National Centers for Environmental Prediction (NCEP) of the NWS Delft University of Technology, and the National Aeronautical and Space Administration (NASA) Goddard Space Flight Center.[27] WAVEWATCH-III is similar in its functionality to the other codes, though it does not provide the same level of modeling detail as ADCIRC. WAVEWATCH-III (see http://polar.ncep.noaa.gov/waves/wavewatch/) has been incorporated in the FEMA HAZUS-MH software[28] and it is used by NCEP to provide real-time hourly predictions of hurricane storm surge.

For each of these models, near-shore waves should be evaluated. Additional codes such as STWAVE[29] and a spectral wave prediction model (SWAN)[30] are used in conjunction with the deep-water surge models.

Developing a hurricane surge elevation hazard curve requires an integrative, interdisciplinary approach that incorporates state-of-the-art knowledge in hurricane science, meteorology, hydrology, and probabilistic methods. This process consists of the following seven steps:

1. Identify probabilistic data for representing storm parametric data available for the area of interest.
2. Select a stochastic set of simulated storm tracks affecting the region of interest.
3. Generate the frequency of occurrence of various storm parameters combinations in the JPM framework.

[27] Federal Emergency Management Agency, "Using HAZUS-MH for Risk Assessment," FEMA-433, 2004.

[28] Applied Research Associates, Inc., "HAZUS-MH Advanced Severe Storm Coastal Risk Assessment Methodology," prepared for National Institute of Building Sciences, 2006.

[29] Smith, J.M., Sherlock, A.R., and Resio, D.T., "STWAVE: Steady-state Spectral Wave Model User's Manual for STWAVE," Version 3.0, ERDC/CHL SR-01-1, U.S. Army Engineer Research and Development Center, Vicksburg, MS, 2001; Smith J. M., and Sherlock, A.R., "Full-plane STWAVE with Bottom Friction: II. Model Overview. System-wide Water Resources Program Technical Note," U.S. Army Engineer Research and Development Center, Vicksburg, MS, 2007.

[30] Booij, N., IJ.G. Haagsma, L.H. Holthuijsen, A.T.M.M. Kieftenburg, R.C. Ris, A.J. van der Westhuysen, and M. Zijlema, "SWAN Cycle III version 40.41: User manual, Delft University of Technology, The Netherlands, 2004.

4. Using storm parameters estimated in Steps 2 and 3 use the SLOSH code to identify potentially significant hazard scenarios. This screening process is a Monte Carlo simulation repeated tens of thousands of times.

5. Provide a detailed hydrodynamic simulation of the region and storms of interest. Hazard significant scenarios generated by the parameters identified in Steps 2 -4 are evaluated more thoroughly, using a suite of codes for more precisely computing the coupled hydrodynamic behavior between the near-shore water circulation and associated wind field (See Figure B-9).

6. Generate surge-height frequency curves by ranking the computed SLOSH/ADCIRC surge heights and the relative frequency. Care should be taken in using the overall curve for higher frequency challenges, where SLOSH predictions may be used.

7. Treat sources of epistemic uncertainty.

B.7.4 Probabilistic Aleatory Uncertainty Model for Storm Surges

A storm surge assessment is necessary to estimate the impact of specific hurricanes, tropical storms and tropical depressions. However, this assessment should also include an estimation of the frequency of hurricane, tropical storm, or tropical depression events and concomitant storm surge levels. The aleatory model is a stochastic model that takes into account spatial and temporal factors such as frequency of occurrence, magnitude, randomness of hurricanes, tropical storms and tropical depressions, variability of storm surge levels, and randomness of coastal tides. Applied to hurricane storm surge, this model estimates the frequency of exceedance of surge levels as a function of multiple parameters of the hurricane or tropical storm. In notational form, the frequency of occurrence of hurricane events can be expressed as shown in Equation B-5:

$$\nu_i = \nu_i(\Delta P, R_P, v_{\mathrm{f}}, \theta_l) \qquad \text{(Equation B-5)}$$

or, as a product of probabilities, as expressed in Equation B-6.

$$\nu_i(\Delta P, R_P, v_{\mathrm{f}}, \theta_l, x) = \Lambda_1\Lambda_2\Lambda_3\Lambda_4\Lambda_5 \qquad \text{(Equation B-6)}$$

Where,

Λ_1 = probability of the hurricane central pressure deficit, ΔP.

Λ_2 = conditional probability of storm radius, R_p, which is a function of the central pressure deficit.

Λ_3 = probability of the hurricane forward velocity, v_f.

Λ_4 = probability of the azimuthal approach/track direction at landfall, θ_l.

Λ_5 = frequency of occurrence of storms per year along the coast.

Given the occurrence of a hurricane (H_i), the peak surge elevation that occurs at a site, including wave effects, is random, and can be denoted by Equation B-7:

$S_{p,i} = S(H_i, W_i) + \varepsilon$ (Equation B-7)

Where,

$S(H_i, W_i)$ = peak elevation due to the hurricane surge and waves for event i.

ε = random term with zero mean and variance, Φ^2.

The variance is a function of the random factors not modeled, and therefore the variability is not explained by $S(H_i, W_i)$.

The peak surge elevations are determined from the physically-based storm surge and wave models described in the previous section. A number of factors contribute to the randomness term, "ε." These include the Holland B parameter,[31] if it is not specifically modeled, and the astronomical tide. Some of these sources of randomness are model aleatory variability and others are parametric aleatory variability (see Table B-2).

Equation B-8 is the mathematical form of an aleatory model for hurricane storm surge.

$$\lambda(S > s) = \sum \nu_i P(S_{p,i} > s \mid H_i)$$ (Equation B-8)

The summation is carried overall hurricane events as defined in Equation B-7.

Alternative methods to estimate the frequency of hurricane storm surge have been used. These methods inform traditional historical data by including paleohydrology of the coastal region to estimate frequency of the event. Historical data storm tracks are available on the NOAA website (noaa.gov) for significant storms as far back as 1855. These records are contained in the NOAA HURDAT North American Hurricane Database). Examples of paleohurricane analysis are included in Platzman, 1963.

B.7.5 Probabilistic Epistemic Uncertainty Modeling for Storm Surges

A number of sources of epistemic uncertainties are associated with the estimate of hurricane frequencies of occurrence and peak surge elevations. Sources of epistemic uncertainty include:

- Model uncertainty in the hurricane surge and wave models;
- Parametric uncertainty in the estimate of the parameters of each of the elements of the aleatory model (see Equations B-6, B-7 and B-8);
- Choice of datasets to estimate model parameters; and,
- Uncertainty in the distribution on central pressure deficits

The IPET report on Hurricane Katrina and NUREG/CR-7134 both discuss epistemic uncertainty. In the IPET study, it was estimated the model uncertainty in the ADCIRC/STWAVE estimate of peak surge

[31] See, for example, P. Vickery and D. Wadhera, "Statistical Models of Holland Pressure Profile Parameter and Radius to Maximum Winds of Hurricanes from Flight Level Pressure and H*Wind Data," Journal of Applied Meteorology and Climatology, Vol. 47, p. 2497, October 2008.

elevations, represented by the logarithmic standard deviation was approximately ten percent for New Orleans, Louisiana. At the time, this was considered an optimistic estimate because a considerable effort had been made to calibrate the ADCIRC model to the surge elevations from Hurricane Katrina and prior storms. In NUREG/CR-7134, the surge model uncertainty was estimated to be approximately 1.5 feet.

Estimates of parametric uncertainty are derived from the statistical analysis of available data. One such example is the parametric uncertainty in deriving parameters connecting hurricane parameters of radius, maximum winds, and central pressure. There is considerable scatter in the data (i.e., aleatory variability) and model uncertainty in the estimate of the parameters used to fit the data. Other examples of parametric model uncertainty include estimating the distribution of central pressure deficits and the rate of hurricane occurrence.

Other sources of epistemic uncertainty may require development of uncertainty models and the use of expert elicitation methods and workshops to interpret and evaluate these models and integrate their results. Examples of such sources are: (a) selection of alternative distributions to model various hurricane parameters (e.g., distribution on central pressure deficits), and (b) selection of datasets.

B.8 PFHA for Flooding Induced by Seiches

B.8.1 Overview

NRC's "Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment" (JLD-ISG-2012-06, January 2013) gives the following general description of seiche phenomena:

> Seiche is an oscillatory wave generated in lakes, bays, or gulfs as a result of seismic or atmospheric disturbances and with a period ranging from a few minutes to a few hours. The oscillatory modes for the body of water in question should be calculated from a variety of potential sources. Sources to consider include (1) local or regional forcing phenomena, such as barometric pressure fluctuations, strong winds, rapid changes in wind direction, surge associated with passage of local storms; and (2) distant but large forcing mechanisms such as distant storms, tsunami, or earthquake-generated seismic waves. For bodies of water with simple geometries, modes of oscillation can be predicted from the shape of the basin using analytical formulas. Most natural bodies of water have variable bathymetry and irregular shorelines and may be driven by a combination of forcing mechanisms (pp. 14-15).

Further information of a general nature can be found in meteorology and oceanography textbooks.

B.8.2 Seiche Modeling

Guidance for the computation of surge heights due to a seiche is contained in "Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment" (NRC, 2012) and other NRC and Corps of Engineers publications. Models such as Platzman's[32] may be used to estimate the maximum surge or seiche still water elevation for Great Lakes sites; coincident wind-generated waves and run-up are estimated as above. Methods for assessing seiche waves typically use one- and two-dimensional transient mathematical models of the Great Lake including the area near the facility. These methods take into

[32] G.W. Platzman, "The Dynamical Prediction of Wind Tides on Lake Erie," Tech Report #7, Dept. of Geophysical Science University of Chicago, 1963.

consideration the potential for the initiating event to occur in periodic resonance with the natural frequency of the bay or lake. Seiches most commonly occur in the United States at the shorelines of the Great Lakes.

B.8.3 Probabilistic Epistemic Uncertainty Modeling for Seiches

The probabilistic evaluation of seiche events can be carried out in much the same way as for hurricane storm surges. Typically, the driver for seiche events is an atmospheric pressure depression resulting from the passage of squall lines or extra-tropical storms. A frequency estimate is constructed by creating simulated storm tracks (based on regional history) and evaluating the effect of these conditions (using hydrodynamic models capable of handling seiche behaviors). The SLOSH model discussed above was developed to evaluate surges due to seiche events. These calculations can be adjusted to account for wind-wave effects and the associated wave run-up (Jelesnianski, et al., 1992).

B.9 PFHA for Flooding Due to Tsunamis

B.9.1 Overview

A number of different terrestrial and extraterrestrial mechanisms may generate a tsunami:

- Seismic events due to fault offsets;
- Submarine and sub-aerial landslides, seismically-initiated or otherwise;
- Pyroclastic flows and caldera collapses during volcanic eruptions;
- Impacts from ice falls, and
- Meteor impacts.

While a site could be exposed to a tsunami generated by any of these mechanisms, the most frequent cause is seismic events. Moreover, of the two more frequently occurring mechanisms, seismic effects produce the largest tsunamis. For this reason the discussion to follow focuses on probabilistic evaluation of such seismic events.

A Probabilistic Tsunami Hazard Analysis (PTHA) has the following components:

- Physically-based models to estimate tsunami waves, including on-shore run-up;
- A probabilistic aleatory model to estimate the frequency of tsunami flooding; and
- A probabilistic model of the sources of epistemic uncertainty in the physically-based models and the uncertainty in the parameters of the aleatory model.

B.9.2 Tsunami Modeling

A physically-based model of tsunami hazards has the following components:

- Seismic source characterization;

- Wave generation and propagation assessment; and
- Tsunami hazard quantification.

The first step in creating a PTHA is to gather supporting data. This effort will include a literature search, obtaining earthquake catalogs and other useful information from various agencies, expert elicitation, and conducting a workshop.

A number of alternative approaches are available to model tsunami waves generated by a seismic event:

- MOST – a suite of numerical programs that simulate all three phases of the tsunami: (1) generation by an earthquake; (2) propagation across the ocean; and (3) run-up.[33] The MOST code uses non-linear, shallow-water wave equations, including Coriolis force terms, expressed in a spherical coordinate system. Dispersion of tsunami waves, an effect of the dependence of wave celerity on the frequency of component waves, is addressed by taking advantage of the numerical dispersion inherent in a finite difference scheme.[34]

- TSUNAMI – N (near) - code that models the near-shore, shallow-water waves and far or deep-ocean wave generation and propagation. The near version code uses linear approximation in deep waters and shallow-water wave equations in shallow waters for computing run-up.

- TSUNAMI – F (far) - code that uses linear approximation of the non-linear, shallow-water wave equations to simulate propagation of the tsunami in the deep ocean, using a spherical coordinate system, but also includes coastal run-up simulations (Titov and Synolakis, 1997).

Other numerical modeling approaches are described in Thio, et al., 2007, Annaka, et al., 2007, and Thio, et al., 2010. In addition, empirical tsunami prediction models have also been proposed (Downes and Stirling, 2001).

B.9.3 Probabilistic Aleatory Uncertainty Model for Tsunamis

A probabilistic aleatory model for tsunami hazards can be formulated in the same manner as a PSHA for earthquake ground motion. The frequency of exceedance of tsunami run-up, R, can be expressed as $\lambda(R>r)$, as shown in Equation B-9:

$$\lambda(R > r) = \sum_{i=1}^{n_s} \nu_i \iint f_i(m) f_i(x \mid m) P(R > r \mid m, x) dm dx \qquad \text{(Equation B-9)}$$

[33] Titov V.V., "Numerical Modeling of Long Wave Runup," Ph.D. Thesis, University of Southern California, Los Angeles, California, 1997. http://search.proquest.com/science/docview/304370736; Titov V.V. and F.I. González, "Implementation and Testing of the Method of Splitting Tsunami (MOST) Model," NOAA Technical Memorandum ERL PMEL-112, NOAA/Pacific Marine Environmental Laboratory, Seattle, Washington, 1997; Titov V.V. and C.E. Synolakis, "Extreme Inundation Flows During the Hokkaido-Nansei-Oki Tsunami." Geophysical Research Letters, 24(11):1315-1318, 1997.

[34] Shuto, N. "Numerical Simulation of Tsunamis." In: Tsunami Hazard, E. N. Bernard (ed.), Kluwer Academic Publishers, Dordrecht, The Netherlands, 171-191, 1991.

Where,

$\nu_i =$ mean rate of occurrence of earthquakes on seismic source, i.

$f_i(m) =$ probability density function on earthquake magnitude for a seismic source, i.

$f_i(x \mid m) =$ probability density function on the location of the fault rupture on a seismic source, i.

$P(R > r \mid m, x) =$ conditional probability distribution on the tsunami run-up that would occur given an earthquake of magnitude m, that occurs at a location x.

The summation in Equation B-9 is carried out over the number of seismic sources that can generate a tsunami at a site. However, not explicitly shown in Equation B-9, but implied, is the fact that the tsunami wave generation is a function of the distribution of slip that occurs on a fault during an earthquake of a given Richter scale and depth. This slip distribution represents both aleatory and epistemic uncertainties.

The aleatory variability in the tsunami run-up that occurs, separate from the randomness of the earthquake, have been studied (Thio, et al., 2010). The authors of this report suggest that the aleatory variability in tsunami run-up estimates is a function of: (a) modeling aleatory uncertainty, (b) parametric aleatory uncertainty associated with the fault dip angle, and (c) the randomness in the slip distribution.

Figure B-11 shows the effect of this variability on the estimate of the tsunami hazard curve at a site. Also shown in Figure B-11 is the effect of truncation level on the tsunami hazard. The lognormal distribution can be truncated to reflect possible limits of the wave run-up. In the figure, truncation levels are expressed in terms of the number of standard deviations (ε = 2, 3, 4). At an annual frequency of exceedance of 10^{-4}, the tsunami wave height varies from approximately four meters when the aleatory variability is not considered, to approximately 15 meters at a truncation level of 4 standard deviations.

The total tsunami hazard at a site is the sum of the hazard associated with all independent tsunami generation mechanisms. This concept is illustrated in Figure B-12. Figure B-12 also shows the wave heights associated with storms and tides. The total hazard is the sum of the individual hazard curves.[35]

[35] Pacific Gas & Electric Company, "Methodology for Probabilistic Tsunami Hazard Analysis: Trial Application for the Diablo Canyon Power Plant Site," submitted to the PEER Workshop on Tsunami Hazard Analyses for Engineering Design Parameters, Berkeley CA, 2010.

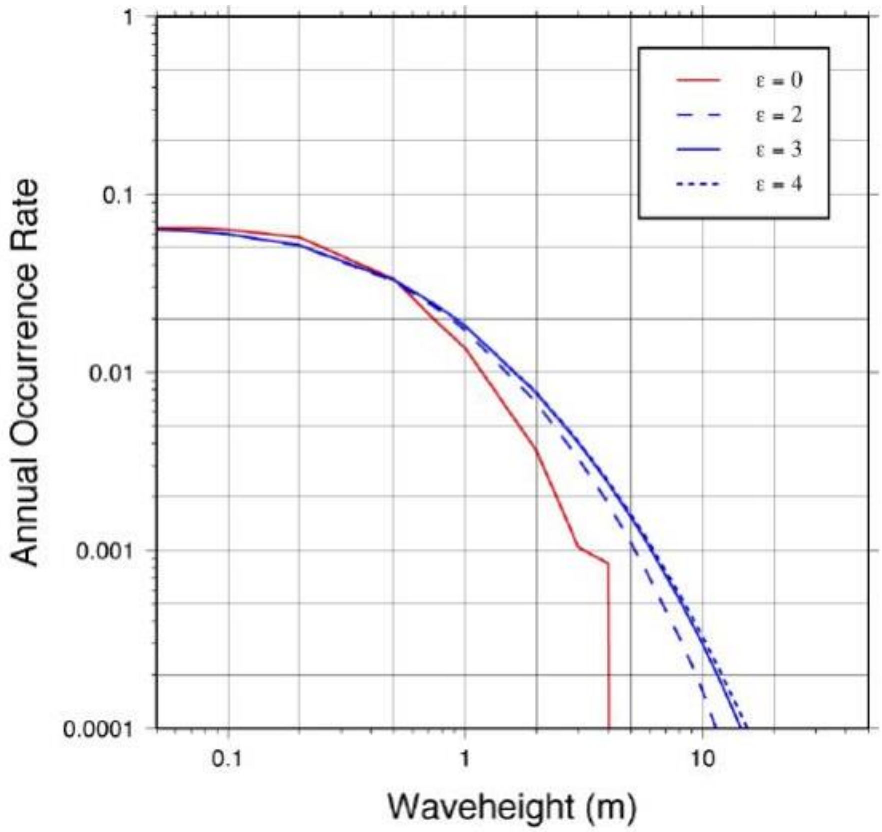

Figure B-11. Effect of the Aleatory Variability and Truncation on the Estimate of Tsunami Hazard at a Site[36]

[36] Thio, H.K., P. Somerville, and J. Polet, "Probabilistic Tsunami Hazard in California," PEER Report 2010/108 (331), Pacific Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, 2010.

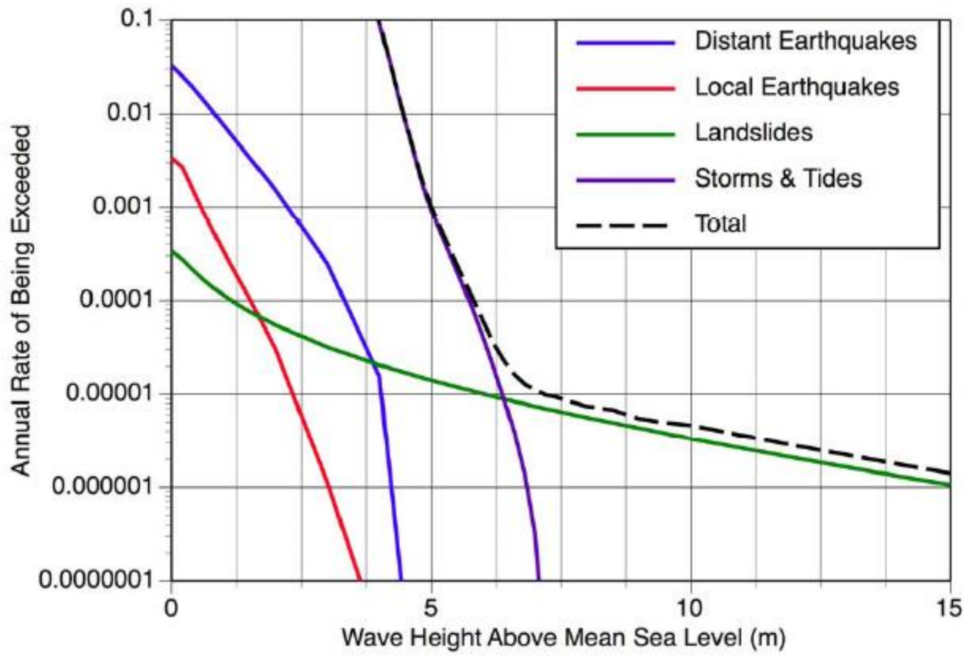

Figure B-12. Tsunami Hazard Curves Associated with Different, Independent Mechanisms[37]

B.9.4 Probabilistic Epistemic Uncertainty Model for Tsunamis

The process of evaluating sources of epistemic uncertainty is well-established in probabilistic seismic hazard analysis (PSHA) practices. In view of this, the evaluation of the epistemic uncertainties in the seismic source characterization part of a PTHA can follow these practices. Other parts of the tsunami hazard model (i.e., uncertainty in the tsunami generation and propagation models) have received less attention. However, available world-wide tsunami records can be used to evaluate model uncertainties. Components of the epistemic uncertainty analysis should include:

- Evaluation of the model uncertainty in the tsunami generation and propagation models;
- Evaluation of model and parametric uncertainty in the seismic source characterization; and
- Uncertainty in other tsunami generating mechanisms, such as landslide volume and velocity.

An example of the uncertainty in tsunami hazard estimates is shown in Figure B-13.

[37] Pacific Gas & Electric Company, "Methodology for Probabilistic Tsunami Hazard Analysis: Trial Application for the Diablo Canyon Power Plant Site," submitted to the PEER Workshop on Tsunami Hazard Analyses for Engineering Design Parameters, Berkeley CA, 2010. Available at http://peer.berkeley.edu/tsunami/wp-content/ uploads/2010/ 09/PGE_tsunami_Apr2010.pdf.

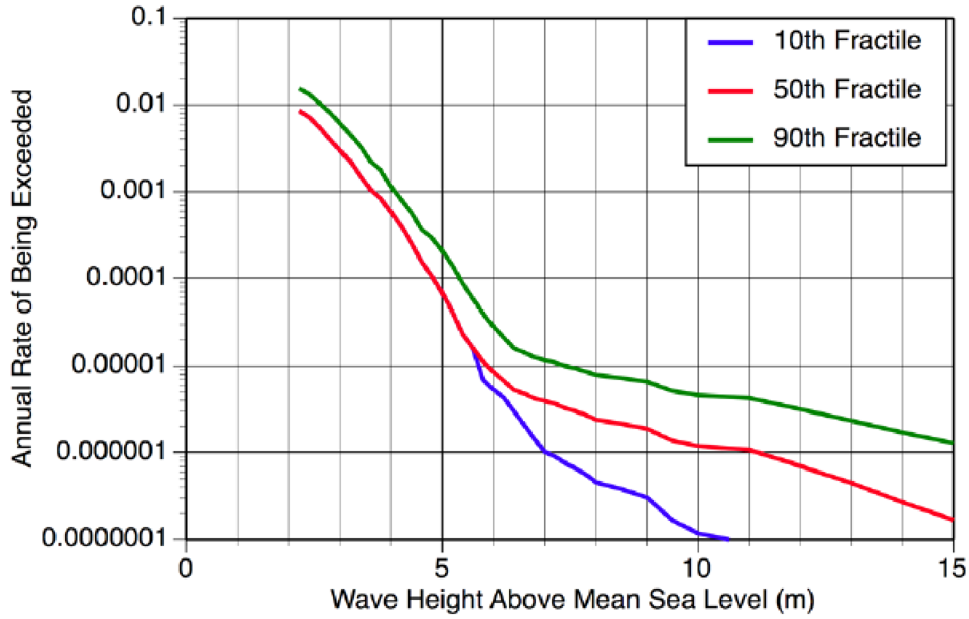

Figure B-13. Uncertainty in a Tsunami Hazard Estimate for a Site[38]

The process of characterizing seismic sources as part of PSHA studies is well-established and required by the NRC for ground motion studies (see NUREG-2117, 2011). The analyst performing the PTHA should characterize each seismic source in terms of:

- Source geometry location, depth, and dip;
- Style of faulting;
- Annual rate of earthquake occurrences;
- Maximum earthquake magnitude;
- Fault rupture length; and
- Fault displacement (i.e., slip) distribution.

Typically, there is considerable epistemic uncertainty in the characterization of a seismic source. In a probabilistic analysis, these uncertainties are explicitly identified, evaluated and quantitatively assessed. Fault tree methods are developed for a seismic source to model the sources of epistemic uncertainty in the characterization of each seismic source. Depending on the available data, there is often considerable uncertainty in the source characterization.

As described in Resio, et al., 2012, and Thio, et al., 2010, a number of hydrodynamic models can be used to model the generation and propagation of seismically-caused tsunami.

[38] Pacific Gas and Electric Company, Peer Workshop, 2010.

B.10 Potential Flood Damage to SSCs

The damage to SSCs and the threat to workers and the public vary depending on the type of flood hazard. In general, structural and non-structural damage can occur if a facility is inundated. Depending on the dynamic intensity of onsite flooding, severe structural damage and complete destruction of SSCs may result. In many cases, structural failure may be less of a concern than the damaging effects of inundation on facility contents and the possible subsequent dispersion of hazardous or radioactive materials. Site inundation can result in damage to safety equipment (e.g., ventilation system) required to maintain confinement.

Structural damage to buildings depends on the intensity of the flood and the local hydraulics of the site. Severe structural damage and collapse can occur when a flood causes multiple effects such as inundation, high flow velocity, transport of large debris, strong waves, and impact loads. Flood stage is perhaps the single most important characteristic of major floods. Flood stages below facility grade generally do not result in severe damage.

Structural failure of roof systems can occur when drains become clogged or are inadequate, and parapet walls allow water, snow, or ice to collect, thus impeding drainage. Exterior walls, both above and below grade, of reinforced concrete or masonry buildings can crack and possibly fail when subjected to flood-related hydrostatic forces.

Depending on the form and amount of material, the effects could be long-term and widespread should the contaminants enter the ground water or are deposited in populated areas.

Table B-4 summarizes the potential damage to buildings and flood protection devices that could result from various types of flood hazards.

Table B-4. Potential Flood Damage to SSCs

Flood Hazard	Potential Damage to SSCs
Submergence	Water damage to contents of the structure Loss of electric power system and component function Settlement of dikes, levees Levee overtopping
Hydrostatic loads	Cracking in structure walls and foundation damage Ponding on structure roofs that can cause collapse Failure of levees and dikes
Hydrodynamic loads	Erosion of embankments and undermining of seawalls Severe structural damage and erosion of levees

APPENDIX C: Additional Topics

a. History of DOE's Graded Approach to NPH Design

DOE's NPH design of SSCs has been based on a graded approach since June 1990, when University of California Radiation Laboratory (UCRL)-15910, *Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards*, was published. DOE-STD-1020-1994, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities,* which replaced UCRL-15910, also used a graded approach for NPH design of SSCs by specifying NPH requirements for various performance categories (PCs), each with a specified Target Performance Goal (TPG), defined as the target mean annual frequency of an SSC exceeding its specified limit state. Performance categorization of SSCs was performed using the criteria and procedure given in DOE-STD-1021-93, *Natural Phenomena Hazards Performance Categorization Guidelines for Structures, Systems, and Components*. Subsequently, DOE-STD-1020-2002 also used a graded approach based on the categorization criteria of the 2002 version of DOE-STD-1021-93.

Following the above trend, ANSI/ANS-2.26-2004 (R2010) adopted a similar graded approach for seismic design categorization of SSCs. In DOE-STD-1020-2012, DOE adopted, with certain modification of criteria, the graded approach and seismic design categorization method of ANSI/ANSI 2.26-2004 and seismic design criteria and method of ASCE/SEI 43-05, *Seismic Design Criteria for Structures, Systems and Components in Nuclear Facilities*; but the design and categorization for other NPHs (e.g., wind and flood) were unaltered. Thus, prior to the publication of DOE-STD-1020-2012, DOE followed two NPH categorization methods: (1) seismic for which SSCs were categorized into five seismic design categories (SDCs), and (2) other NPH (flood etc.) for which the SSCs were categorized into five PCs. DOE's adoption of ANSI/ANS-2.26-2004, Revision 2010, and ASCE/SEI 43-05 for seismic design was partly in recognition of the National Technology Transfer and Advancement Act of 1995 that requires (with certain exceptions) federal use of voluntary consensus standards developed by the industry.

In DOE-STD-1020-2012, the categorization method for seismic design, as set forth in ANSI/ANS-2.26-2004 (R2010) was extended, with certain modification of criteria, to other NPH designs, and the use of graded approach continued to be followed. The motivation for the graded approach was to enable design or evaluation of SSCs to be performed in a manner consistent with their importance to safety, importance to mission, and cost.

In ANSI/ANS-2.26-2004 (R2010) and ASCE/SEI 43-05, the term Limit State has been used to characterize deformation limits resulting from seismic loads. The term failure indicates that the deformation is greater than allowed deformation for a given limit state. For example, Limit State A for a building structure indicates that the building can perform its safety function even when it has large permanent deformations, when subjected to seismic loads and other applicable concurrent loads, whereas Limit State D would indicate that the building has essentially elastic deformations. Failure of Limit State D indicates that there are significant inelastic deformations and does not necessarily indicate collapse.

The term Limit State is applicable to seismic design only, and not to any other non-seismic NPHs. Deformation levels associated with SSC failure resulting from other non-seismic NPH loads are established based on permissible stresses, strains, or capacities specified in the industry codes or regulatory requirements applicable to the project. For example, for flood or tornado wind hazards evaluation of a building structure, SSC failure would be defined in terms of stresses, strains, or capacities specified in the building code.

b. Target Performance Goals (TPGs) Based on NDC

The NDC of an SSC is selected based on the consequence of SSC failure; the more severe the consequence, the higher the NDC. Design of SSCs to DOE-STD-1020 criteria associated with a given NDC will result in achieving a PG for the SSCs.

In determining what were considered to be appropriate TPGs, DOE evaluated data from two industry sources: (1) Seismic Probabilistic Risk Analyses (PRAs) of more than 20 nuclear power plants designed to stringent Nuclear Regulatory Commission (NRC) criteria, and (2) SME interpretation of seismic performance data for building structures designed to model building codes such as the Uniform Building Code (See *Guidelines for the Development of Natural Phenomena Hazards Design Criteria for Surface Facilities,* by Nelson, Hossain, and Murray, 1992).

The ratio between the annual probability of the design basis earthquake (DBE) and the estimated SSC failure rate for these two methods of design was estimated to measure inherent conservatism in the NPH analysis and SSC design methods. The term used in earlier versions of DOE-STD-1020 for this ratio was risk reduction factor (RRF). RRFs for seismic design were estimated to be more than one, indicating that when the SSC is subjected to a design basis seismic event, the probability of SSC failure is less than unity. However, there was no rigorous study to demonstrate that this is true for tornado, hurricane, and flood NPHs. The uncertainties in these hazard and demand estimates are judged to be much higher (see also Section 4.3.10b).

Consequently, for these NPHs, higher design basis return periods (DBRPs) were selected separately for each NDC to achieve the same TPGs. The TPGs for various categories applicable for all NPHs were given in Table B-1 of DOE-STD-1020-2002. These were the basis for TPGs recommended in ANSI/ANS-2.26-2004 (R2010) and ASCE/SEI 43-05 for seismic design only.

For SSC design purposes, PG values are not required to be calculated either in DOE-STD-1020-2012 or in its earlier versions. However, for seismic design of SSCs, the methodology and requirements given in DOE-STD-1020-2012 and DOE-STD-1020-2016 are aimed at achieving certain TPGs that are given in ANSI/ANS-2.26-2004 (R2010) and ASCE/SEI 43-05. These seismic TPGs are achieved by selecting DBE return periods and design and analysis criteria specified in DOE-STD-1020-2012 and DOE-STD-1020-2016. For other types of NPH, no such TPGs are specified. However, design basis NPH return periods and design criteria have been specified in DOE-STD-1020-2012 and DOE-STD-1020-2016 for most other NPHs, based on which one can estimate the PG values associated with each type of NPH.[39]

c. System Interaction

The effect of system interaction is an important consideration in determining the NDC of an SSC. Unlike DOE-STD-1021, in which the Performance Category of an SSC was determined before considering the system interaction effect, the ANSI/ANS-2.26-2004 (R2010) method of NDC determination has integrated the consideration of system interaction in a single step.

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[39] PG values for non-seismic NPHs, unlike those for seismic hazards, have not been studied with rigor. The design criteria used to mitigate non-seismic NPHs have the same level of conservatism as those for seismic design, but their design basis return periods are either the same or more than those for seismic NPH. So, the quantitative PG probability values for non-seismic NPHs, and hence the SSC failure rates, are likely to be equal or less than those due to seismic hazards.

As required by Section 3.6 of the Standard, capacities of SDC-1 and SDC-2 SSCs are determined using ASCE/SEI 7-10 requirements for Risk Category II and Risk Category IV, respectively. Furthermore, capacities of SDC-3 through SDC-5 SSCs are determined using ASCE/SEI 43-05 requirements. In determining the strength values of new or modified anchorage, the quality assurance requirements outlined in Chapter 10 of the Standard and Section 10 of this handbook need to be met.[40] If strength data from a manufacturer is used, it should be confirmed that the bolt is acquired under an appropriate quality assurance program. However, in general, to comply with the requirement in Section 3.6.2 of the Standard, seismic design and evaluation of SDC-1 and SDC-2 non-building structures and non-structural components, including mechanical and electrical equipment systems can be performed based on the requirements and guidance given in Sections 13 and 15 of ASCE/SEI 7-10. Specifically, the design of active mechanical and electrical equipment in ASCE/SEI 7-10 seismic design categories C through F conforms to the requirements in Section 13.2.2 of ASCE/SEI 7-10.

d. Seismic Performance

Anchorage and Supports

The presence of properly engineered anchorage is the most important single item affecting seismic performance of equipment. There are numerous examples of equipment sliding or overturning in earthquakes due to lack of anchorage or inadequate anchorage. These deficiencies can also threaten adjacent safety-related items or personnel through spatial interaction.

Engineered anchorage, required for all SDCs, is one of the most important factors affecting seismic performance of systems or components. Anchorage should have both adequate strength and sufficient stiffness to perform its safety function. Typical anchorages are: (1) cast-in-place bolts or headed studs; (2) expansion or epoxy grouted anchor bolts; and (3) welds to embedded steel plates or channels. The most reliable anchorage will be achieved by properly installed cast-in-place bolts or headed studs, undercut anchors and expansion anchors, or welding. Other expansion anchors are less desirable for vibratory environments (such as rotating machinery), for very heavy equipment, or for sustained tension supports. Epoxy-grouted anchorage is considered to be the least reliable of the alternatives, especially in elevated temperature or radiation environments.

Evaluation of facilities following earthquakes has demonstrated that ductile structures with properly anchored systems and components perform very well. This empirical evidence illustrates the importance of having properly engineered anchorage as part of the seismic design criteria. As a best practice, these criteria would encourage the use of larger and deeper embedment rather than minimal calculated anchorage. Use of cast-in-place, undercut anchors, expansion anchors should also be encouraged.

Adequate strength of equipment anchorage requires consideration of tension, shear, and shear-tension interaction load conditions. The strength of cast-in-place anchor bolts and undercut type expansion anchors should be based on the requirements in ASCE/SEI 7-10 and ACI 318-14, *Building Code Requirements for Structural Concrete and Commentary*, for SDC-1 and SDC-2 SSCs, and on the requirements in ASCE/SEI 43-05 and the most recent version of ACI 349-13, *Code Requirements for*

[40] Quality assurance and quality control problems related to anchors are more common and so are specially emphasized here. Quality assurance and quality control related data and documents and their traceability should be assured in accord with the requirements of Section 10 of the Standard. The quality assurance plan should emphasize the importance of documenting the load path.

Nuclear Safety Related Concrete Structures for SDC-3 through SDC-5 SSCs. For new design by the ACI-349 provisions, anchor designs should have a ductile-failure mode. For existing facility evaluation, some relaxation of acceptance criteria may be considered, provided detailed inspection and evaluation of the anchor bolt are performed in accordance with DOE/EH-0545, *Seismic Evaluation Procedure for Equipment in U.S. Department of Energy Facilities* and EPRI NP-5228, *Seismic Verification of Nuclear Plant Equipment Anchorage* (EPRI, 1991).

The strength of expansion anchor bolts can be based on strength values available from standard manufacturers' recommendations or sources such as site-specific tests or DOE/EH-0545 and EPRI NP-5228. Strength values typically include a safety factor of about four on the mean ultimate capacity of the anchorage. It is permissible to utilize strength values based on a lower factor of safety for evaluation of anchorage in existing facilities, provided detailed inspection and evaluation of anchors are performed as noted above. When an anchorage is modified or a new anchorage is designed, strength values should be determined based on applicable industry code requirements (IBC, ACI 349, and DOE/EH-0545).

Stiffness of equipment anchorage should also be considered, in accordance with these code requirements. Flexibility of anchorage can be caused by the bending of anchorage components or any other component of the equipment that is being anchored. Excessive eccentricities in the load path between the equipment item and the anchor are a major cause of base anchorage flexibility. Equipment base flexibility can allow excessive equipment movement and reduce its natural frequency, possibly increasing dynamic response. In addition, flexibility can lead to high stresses in anchorage components and failure of the anchorage or equipment.

Material Strength Properties for Capacity Evaluation

In accordance with Section 3.6 of the Standard, seismic design of SSCs applies capacities (C_C) that are based upon code specified minimum values. For concrete SSCs, C_C is calculated as specified in ACI 318-14, (for SDC-1 and SDC-2 SSCs) or in ASCE/SEI 43-05 and ACI-349-13 (for SDC-3 through SDC-5 SSCs). For SDC-1 and SDC-2 steel SSCs, the appropriate American Institute of Steel Construction (AISC) specifications are used, as required in IBC-2015. For SDC-3 through SDC-5 steel SSCs, ANSI/AISC N-690-2012, *Specification for Safety-Related Steel Structures for Nuclear Facilities*, is used as required in ASCE/SEI 43-05.

Special Considerations for Seismic Qualification of Systems and Components

Special considerations for assessing the seismic resistance capacity of equipment and distribution systems include:

1. Equipment supported on the ground or on the ground floor within a structure may experience the same earthquake ground motion as the structure. However, equipment or distribution systems supported within a structure respond to the motion of the supporting structure in a manner that can be significantly different from the ground motion.

2. Equipment or distribution systems may have either negligible interaction or significant coupling with the response of the supporting structure (see ASCE/SEI 43-05 and the most recent version of ASCE 4). For SDC-3 through SDC-5 SSCs, if the interaction is negligible, as determined by ASCE 4 methods and criteria, only the mass of the equipment needs to be included in the model of the building structure, and the equipment may be analyzed independently. If the equipment coupling

effects are not negligible, the equipment mass and stiffness properties should be modeled in the building structure model.

3. Equipment or distribution systems supported at two or more locations within a structure may be stressed due to both inertial effects and relative support displacements.

4. Many equipment items in DOE facilities are similar, in configuration and functional requirements, to those in industrial facilities throughout the world. As a result, a significant amount of data is available on the performance of equipment in past earthquakes and in shake-table qualification testing. Equipment which has performed well, based on such experience, may not require additional seismic analysis or testing, if it can be shown to be adequate based on the criteria and procedure given in DOE/EH-0545.

Use of Seismic Experience Data

For design of new systems and components, seismic qualification will generally be performed by analysis or testing as discussed in the previous sections. However, for existing systems and components, it is anticipated that many items will be judged adequate for seismic loadings on the basis of seismic experience data without analysis or testing. Seismic experience data was originally developed in a usable format by research sponsored by the nuclear power industry. In early 1982, the Seismic Qualification Utility Group (SQUG) (http://squg.mpr.com/) was formed for the purpose of collecting seismic experience data as a cost effective means to verifying the seismic adequacy of equipment in existing nuclear power plants. Sources of experience data included: (1) the numerous non-nuclear power plants and industrial facilities with equipment similar to that in nuclear plants, which have experienced major earthquakes; and (2) shake table tests, which had been performed to qualify safety-related equipment for licensing of nuclear plants. This information was collected and organized and guidelines and criteria for its use were developed. SQUG's *Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment* is the generic means for applying this experience data to verify the seismic adequacy of mechanical and electrical equipment, which may be used in a nuclear facility during and following the occurrence of a design level earthquake. Also relevant for the use of experience data to verify the seismic adequacy is the Senior Seismic Review and Advisory Panel (SSRAP) report (SSRAP Report, 1991).

The seismic adequacy of equipment and distribution systems is often as important as the adequacy of the building. In March of 1997, DOE issued "Seismic Evaluation Procedure for U.S. Department of Energy Facilities," designated DOE/EH-0545. This document was prepared to provide practical guidelines for the support and anchorage of many equipment items that are likely to be found in DOE facilities. It examines strengthening and upgrading of equipment to increase the seismic capacity in existing facilities. It was developed based on the SQUG 2001 GIP report, and the SSRAP report cited above. DOE/EH-0545 can also be used in new facilities with respect to support and anchorage.

Note the numerous restrictions and caveats on the use of this data are stated in the SSRAP report and the GIP. It is necessary to conduct either seismic analyses or shake table testing to demonstrate sufficient seismic capacity for those items that cannot be verified by seismic experience data or for items that are not obviously inherently rugged. Based on DOE's program on the application of experience data for the evaluation of existing systems and components, DOE/EH-0545 was prepared and specifically tailored to DOE-type facilities.

In order to utilize earthquake experience data to demonstrate seismic adequacy of equipment, DOE/EH-0545 procedure recommends that the following four criteria be satisfied:

1. The seismic motion at the equipment location is enveloped by the Generic Equipment Ruggedness Spectrum, Test Spectrum and Reference Spectrum (see Section 5.4.1 of DOE/EH-0545;

2. The equipment falls within the bounding criteria for a given class of similar equipment, which has survived strong earthquake shaking or past qualification tests;

3. The anchorage of the equipment is adequate to survive design level seismic loads; and

4. The equipment meets the inclusion or exclusion rules, also called caveats.

The use of earthquake experience data to verify the seismic adequacy of equipment demands considerable engineering judgment. For this reason, the peer review process should be used to examine use of these procedures.

For SDC-1 and SDC-2 SSCs, seismic evaluation of equipment or non-structural elements supported by a structure can be based on the total lateral seismic force as given in the ASCE/SEI 7-10. For SDC-3, 4, and 5 SSCs, the seismic evaluation of equipment and distribution systems can necessitate the development of floor response spectra representing the input excitation. Once seismic loading is established, seismic capacity can be determined by analysis, testing, or, if available, the use of seismic experience data. Whenever possible, seismic evaluation should be based on experience data.

The Seismic Qualification Reporting and Testing Standardization (SQURTS) program (EPRI, 2009) was initiated in the early 1990s by the EPRI to address obsolescence of components in nuclear power plants. This program applies the economies of scale of member utility owners and operators to share component seismic testing specifications, costs, and test results. EPRI manages a seismic test report database comprising SQURTS-performed test results and individual member test reports.

Seismic Interaction

During an earthquake, the seismic response of one SSC may affect the performance of another SSC. This sequence of events is called seismic interaction. Seismic interactions that could have an adverse effect on SSCs are considered in seismic design and evaluation of DOE facilities. Cases of seismic interaction to be considered include (1) structural failing and falling, (2) proximity, (3) flexibility of attached lines and cables, (4) flooding or exposure to fluids from ruptured vessels and piping systems, and (5) effects of seismically-induced fires.

ANSI/ANS-2.26 requires that if the seismic failure of a given SSC (the “Source SSC”) adversely affects the safety function of another SSC (the “Target SSC”), then the Source SSC should be designed to withstand a DBE level applicable for the SDC of the Target SSC, such that the Source SSC cannot adversely interact with the Target SSC. This requirement essentially renders the Source SSC to be designated an SDC level at least as high as the Target SSC. However, the Source SSC is not required to be designed to the same Limit State as the Target SSC. Its Limit State need only be such that it cannot adversely interact with the Target SSC during a DBE applicable for the Target SSC. Thus, if ANSI/ANS-2.26 is implemented, the Source SSC will be designed to withstand a DBE that is at least as high as the Target SSC. The source SSC, however, is not required, even though permissible, to meet the quality assurance (QA) requirements applicable to the target SSC, unless the direct safety function of the source SSC is such that it is at the same SDC level as the target SSC.

For an existing facility, an interaction problem can arise when a higher category (such as an SDC-4) SSC (target) is in danger of being damaged due to the failure of overhead or adjacent lower category (such as an SDC-1, 2, or 3) SSCs (source), which have been designed for lesser seismic loads than the higher category SSC (target). Lower category items interacting with higher category items or barriers protecting the target items need to be designed to prevent adverse seismic interaction. If there is a potential interaction adversely affecting the safety function of the target SSC, the SDC of the source SSC may need to be upgraded to be the same as that of the target SSC according to ANSI/ANS-2.26. However, the source SSC is not required to be designed to the same limit state as that of the target SSC. Limit state for the source SSC, according to ANSI/ANS-2.26, should be consistent with its own safety function.

Impact between structures, systems, or components in close proximity to each other due to relative motion during earthquake response is another form of interaction, which should be considered. If such an impact could cause damage or failure, a combined design approach should be used: sufficient separation distance to prevent impact together with adequate anchorage, bracing, or other means to prevent large deflections. Note that even if an impact occurs between adjacent structures or equipment, significant damage may not result. For example, rupture of a one-inch diameter pipe probably may not damage a twelve-inch diameter pipe regardless of the separation distance. However, the designer/evaluator needs to justify and document these cases to satisfy the seismic design requirements given in ASCE/SEI 43-05 and ASCE/SEI 7-10.

Design measures for preventing adverse performance from structural failing and falling and proximity seismic interaction modes include: (1) strength and stiffness; (2) separation distance; and (3) barriers. Sources may be designed to be strong enough to prevent falling or be sufficiently stiff to prevent large displacements such that adverse interaction does not occur. To accomplish this, the source item may sometimes need to be designed to more stringent structural integrity design criteria than are required for its own function. Maintaining function of the source item under this increased seismic design requirement may not be necessary. Source and targets can be physically separated at sufficient distance such that, under maximum seismic response displacements expected for target design criteria earthquake excitation, adverse interaction will not occur. Barriers can be designed to protect the target from source falling or source motions. In accordance with the requirements of ANSI/ANS-2.26, barriers should be designed to withstand impact of the source item without endangering the target SSC.

Insufficient flexibility to accommodate relative movement is another form of seismic interaction. This may occur when distribution lines such as piping, tubing, conduit, or cables are connected to an item important to safety. For safety SSCs, good practice mandates that sufficient flexibility of such lines be provided to assure functionality under maximum relative displacements.

Other forms of seismic interaction result if vessels or piping systems rupture due to earthquake excitation and cause fires or flooding, which could affect performance of nearby important or critical SSCs. In this case, such vessels or piping systems should continue to perform their function of containing fluids or combustibles, so they are elevated in category to the level of the targets that would be endangered by their failure.

Seismic Monitoring at DOE Facilities

This appendix is provided as guidance to assist DOE sites in meeting the seismic detection and recording requirement in DOE O 420.1C, Chg. 1, Attachment 2, Chapter IV, paragraph 3.e. Neither the DOE O 420.1C nor DOE-STD-1020-2016 provide detailed guidance on what types or densities/quantities of seismic monitoring instruments and records are appropriate for facilities of different sizes or with varying hazard levels. These are provided here, along with a brief and introductory description and use of various types of seismic monitoring instruments (additional guidance from ANSI/ANS-2.2-2016, *Earthquake Instrumentation Criteria for Nuclear Power Plants,* that is consistent with the guidance below, can also be used).

Types of Seismic Monitoring Instruments

Seismic monitors can be categorized into strong motion and weak motion types. A strong motion accelerograph (SMA) is best suited for measuring accelerations from strong potentially damaging earthquakes. It can measure accelerations in the ground as well as in the structural member of a facility. A SMA measures accelerations, as a function of time in three orthogonal (i.e., one vertical and two horizontal) directions, producing a time-history record of an earthquake. The horizontal dimensions are normally oriented north-south and east-west, or if the SMA is within a facility, it is oriented in alignment with the primary horizontal dimensions of the structure. SMAs begin recording after a trigger acceleration is reached, where 0.01g is a common set point. SMAs are useful for determining whether an earthquake has exceeded the design basis ground motion for a facility.

Weak motion seismometers measure small-magnitude ground accelerations more precisely than SMAs, but do not provide meaningful results for larger-magnitude seismic events. Weak-motion seismometers measure velocity or acceleration, and historically have been used to help determine distances to the earthquake epicenters and earthquake magnitudes. Most modern weak-motion seismometers measure movement in three directions, but some older equipment measure only in the vertical direction. Several DOE sites have older equipment, termed short-period seismometers, and they often measure velocity over a narrow high frequency range. Their use is essentially limited to determining earthquake magnitudes and epicenter distances. More modern weak motion seismometers, termed broadband seismometers, can measure accelerations over a wide frequency range (i.e., at least 0.2 to 50 Hz). Broadband seismometers reveal more about the physical processes occurring at the earthquake source and provide information about how the geology beneath the seismometer is responding to seismic input motions. These seismometers can provide helpful data for determining Kappa, a site-specific attenuation factor. Moreover, weak-motion seismometers are vital to establishing a long-term data base that provides input to future site-specific PSHAs.

Sites with Facilities Containing SSCs Below or Equal to SDC-2

A DOE site consisting of facilities with SSCs categorized no higher than PC-2 (per DOE-STD-1020-2002), or SDC-2 (per DOE-STD-1020-2016), is not required to have a site-specific PSHA (see Section 3 of ANSI/ANS-2.27-2008 (R2016). Therefore, such a site may not need its own broadband instruments for long-term data collection purposes. The requirement in DOE O 420.1C, Chg. 1, Attachment 2, Chapter IV, paragraph 3.e to detect and record the occurrence and severity of seismic events may be met through a cooperative agreement with a surrounding regional seismic network. The United States Geological Survey (USGS), a nearby university, or a State or local agency may maintain a seismic network that can provide sufficient information to meet the Order requirement. A written agreement with such an entity should be in place to demonstrate that the DOE site office or facility operator has the capability of obtaining requisite earthquake information. However, the DOE facility should have its own broadband instrument if no active seismic network station is located within 50 miles.

One or more SMAs should be available if a facility needs to remain functional after a seismic event. The SMAs will allow comparison of actual accelerations to design values, facilitating decisions on re-entry, re-occupancy and re-start. A SMA may also be helpful for assessing potential damage to high-value DOE assets, regardless of the hazards posed by the facilities.

Small Sites with Facilities Containing SDC-3 or Higher SSCs

A small DOE site (i.e., less than 1-2 square miles) containing one or more facilities with SSCs categorized as PC-3 (per DOE-STD-1020-2002) or SDC-3 (per DOE-STD-1020-2016), or higher, should maintain a seismic monitoring program or participate in a regional monitoring network such that the data necessary to perform a site investigation (see ANSI/ANS-2.27-2008 (R2016)) and site-specific PSHA (see ANSI/ANS-2.29-2008 (R2016), *Probabilistic Seismic Hazards Analysis*) can be collected and recorded. Such a network should have several (i.e., approximately ten) broadband seismometers within a 50-mile radius from the site boundary. The number and siting of the monitoring stations is influenced by the regional geology and tectonic activity and needs to provide representative data. Such a network is important for providing seismic catalog data and site-specific attenuation data to support future PSHA updates. A SMA should be located in the free-field near each PC-3/SDC-3, or higher facility, although a single station may be adequate for multiple facilities within a small site. Multiple free-field SMAs may be prudent for facilities that are close together but are known to have unique subsurface geology that could result in different surface responses.

Each facility containing PC-3/SDC-3 or higher SSCs with an enduring mission (i.e., remaining life greater than five years) should, depending on the structural configuration of the facility building, have one or more in-structure SMAs to aid in establishing whether seismic motions have exceeded the facility's design basis. If such a facility in a small site does not have an enduring mission, and within five years the small site will have no facilities above PC-2/SDC-2, then the site only needs to meet the monitoring expectations of Section 3.7.2.

Large Sites with Facilities Containing SDC-3 or Higher SSCs

A large DOE site (i.e., 2 square miles or larger) containing one or more facilities categorized as PC-3 (per DOE-STD-1020-2002) or SDC-3 (per DOE-STD-1020-2016), or higher, should maintain a seismic monitoring program or participate in a regional monitoring network, such that the data necessary to perform a site investigation (see ANSI/ANS-2.27-2008 (R2016)) and site-specific PSHA (see ANSI/ANS-2.29-2008) can be collected and recorded. Such a network should have several (i.e., approximately five) broadband seismometers within the site boundary. The network should also have approximately 15 broadband seismometers within a 50-mile radius from the site boundary. The number and siting of the monitoring stations is influenced by the regional geology and tectonic activity and need to provide representative data. Such a network is important for providing seismic catalog data and site-specific attenuation data to support future PSHA updates. A SMA should be located in the free-field near each location containing one or more PC-3/SDC-3, or higher, facilities. Co-locating some SMAs with broadband monitoring stations may be desirable. Multiple free-field SMAs may be prudent for facilities that are close together but are known to have unique subsurface geology that could result in different surface responses.

Each PC-3/SDC-3, or higher facility with an enduring mission (i.e., remaining life greater than five years) should, depending on the nature of the structure, have one or more in-structure SMAs to aid in establishing whether seismic motions have exceeded the facility's design basis. If the PC-3/SDC-3 or higher facilities do not have an enduring mission, and within five years the large site will have no facilities above PC-2/SDC-2, then the site only needs to meet the monitoring expectations of Section 3.7.2.

Seismic Monitoring Upgrade Plans

Some DOE sites have seismic monitoring programs that do not meet the seismic monitoring guidance of Sections 3.7.2 through 3.7.4. For example, existing weak motion seismometers may be a mix of short-period and broadband. The network descriptions here should be considered a vision for sites to work toward. Sites with an enduring mission should evaluate their network with respect to these criteria, and if upgrades are merited, should then develop a seismic monitoring upgrade plan to ensure that the network eventually meets these criteria. A long-term upgrade plan may result in the eventual replacement of short-period seismometers with broadband, installation of in-structure SMAs, or other upgrades.

Additional Seismic Monitoring Information

In 2012, the Electric Power Research Institute (EPRI) published *Seismic Instrumentation at Nuclear Power Plants*, EPRI # 1024889, available at www.epri.com. This document provides additional information on seismic monitoring systems, including considerations in locating seismic instruments.

e. Use of Fragility Analysis or Seismic Margin Study to Assist in a PISA or USQ Determination

Section 9.3.4 of the Standard mentions that a fragility analysis or seismic margin study may be performed to assist in the PISA and USQ determinations, and to justify continued operation of the facility. The following approaches are permitted as fragility or seismic margins studies:

- ASCE/SEI 43-05 is targeted to probabilistic performance goals. The ASCE/SEI 43-05 commentary has sensitivity analyses that demonstrate Target Performance Goals for SDC-3 through SDC-5 will be reasonably satisfied if one follows the standard's criteria.
- ASCE/SEI 43-05 also offers an alternate probabilistic criterion that involves demonstrating conditional failure probabilities for the DBE are less than about 1%, and for 1.5 times the DBE are less than about 10%. The 1.5 DBE criterion is useful in evaluating NPH events with a "cliff effect," such as seismically-induced soil settlement.
- Calculating a mean Annual Probability of Exceedance (APE) for absolute comparison to the Target Performance Goal in ASCE/SEI 43-05 may be useful:
  - One approach would be to perform a series of seismic analyses to gauge the median capacities and uncertainties.
  - An alternate approach would be to compute high confidence of low probability of failure (HCLPF) values using the methodology in EPRI NP-6041, *A Methodology for Assessment of Nuclear Power Plant Seismic Margin*, Revision 1), and to generate median capacities from these HCLPF by assuming various values of uncertainty.

Regardless of the methodology used, the engineer must provide a firm technical basis for the median capacities and variabilities considered in the analysis. In facilities with multiple failure modes, the failure modes should be combined using Boolean logic as the system level performance for weakly correlated failure modes will be greater than the worst performance of the individual failure modes.

Depending on the SSC being considered, the appropriate seismic analyses to develop fragilities or seismic margins may be elastic or nonlinear. Consistent with ASCE 43-05, nonlinear analyses can be either pseudo-static (i.e., pushover analyses) or nonlinear dynamic time history analyses. ASCE 43-05 offer deformation limits, for each Limit State, for use in nonlinear analysis. These deformation limits are targeted to preventing significant cyclic strength degradation, which provides reserve margin for aftershocks.