SAFETY • RESPONSIBILITY • SECURITY
SRS
SAVANNAH RIVER SITE

ENGINEERING STANDARD

STRUCTURAL DESIGN CRITERIA	STANDARD NO: 01060	PAGE 1 of 44
	DATE: 8/12/2010	REVISION: 10
	ESB TECH COMMITTEE: Civil/Structural	
	Approved by: Ken Stephens, Signature on File, Chairman (M&O), Engineering Standards Board; Noel Chapman, Signature on File, Chairman (LWO), Engineering Standards Board	

REVISION HISTORY

REV	DATE	DESCRIPTION OF REVISION
0	8/1/95	INITIAL ISSUE
1	10/1/96	Included DOE Order 420.1, the SBC, requirements for concrete anchors and chipping, deleted unused acronyms and references, and editorials.
2	07/28/97	Included site specific design spectra for PC-3 and PC-4, UBC 1997 seismic loads and loading combinations and editorials. Revised basic wind speed from "fastest mile" to "three seconds gust".
3	07/98	Added UBC seismic ductility provisions and earthquake load factor of 1.2 for new PC-3 and PC-4 structures, increase PC-3 design basis tornado speed to 178 mph, added automobile in the PC-3 tornado missile criteria, took out dates of reference National Codes and Standards, and added notes to some loading combinations.
4	09/99	Added reference criteria for stacks, removed the SBC, added dynamic settlement load factor of 1.2 for new PC-3 and PC-4 structures, added correlation between Functional Classification and Performance Categories, revised PC-1 and PC-2 design wind speeds, and revised PC-3 design basis spectra.
5	9/28/01	Revised to incorporate IBC in place of UBC, revised load combinations, revised tornado wind speeds, added ice loading criteria, and added Appendix A.
6	6/28/02	Incorporated changes from 2002 revision of DOE-STD-1020, revised PC-3 vertical seismic input spectra, added seismic requirements for systems and components, added tornado shelter requirements for PC-1 and PC-2 structures, included OSHA steel erection rules relating to design, and added test load requirement for embedded lift point tests, added clarification that PC are to be identified in design input documents, revised Table 7.1.4, clarified elevation reference in Section 5.2.7.
7	3/21/05	Aligned PC-1 and PC-2 requirements with IBC requirements. Allowed either ACI 318 or ACI 349 for PC-3 concrete design. Allowed either AISC-LRFD, AISC 335, ASCE 8, ANS/AISC N690 or ANS/AISC N690L for PC-3 steel design. Allowed either ANS/AISC N690 or ANS/AISC N690L for PC-4 steel design. Added ANSI/AISC 341 steel detailing requirements in lieu of FEMA 350 and FEMA 353 requirements. Reference model building codes for load combinations. Added provisions for sliding of unanchored PC-3 and PC-4 SSC. Enhanced PC-3 and PC-4 foundation design requirements. Updated PC-4 tornado requirements. Updated PC-3 and PC-4 snow load requirements. Added a tornado missile to tornado shelter requirements. Added ACI 360 for slabs on grade. Limited the applicability of PC-1 and PC-2 seismic accelerations in Appendix A.

REV	DATE	DESCRIPTION OF REVISION
		Updated references. Moved supplemental information to the Appendix. Reorganized Chapter 5 and renumbered Sections.
8	8/6/2006	Deleted all criteria for systems and components. Referenced SRS Engineering Standard 01061 for design criteria of systems, equipment and components. Deleted notes on Performance Category. Added Structural Requirements Matrix for various combinations of safety functions of structures, systems and components, which addresses the safety concern for the collocated worker. Deleted section 5.1.4.1, 5.1.4.2, 5.1.4.3, 5.2.2.4 and A5.2.2.4. Replaced section 5.1.4 and A5.1.4. Revised requirements for PC-3 and PC-4 foundation design. Revised PC-3 and PC-4 atmospheric ice load concurrent with wind load. Added a section on Seismic Structural Interaction. Clarified provisions on lift points.
9	8/11/2009	Incorporate DOE-STD-1189 requirements and updated code references to current editions
10	8/12/2010	The relationship between PC and SDC classification added, corresponding explanation added in the commentary, other minor editorial revisions.

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1.0 PURPOSE AND SCOPE

1.1 This document provides minimum structural design criteria for all new facilities and modifications to existing facilities, both permanent and temporary, at the Savannah River Site (SRS). Functional Classification and NPH criteria are determined as part of the hazard analysis process (see WSRC-SCD-11)

Existing structures may be analyzed for these criteria for the desired functional classification, safety function, Performance Category (PC) and Seismic Design Category (SDC). The goal of structural analysis of existing structures is to demonstrate compliance with current codes to the greatest degree practicable. Acceptance criteria for non-compliance with current criteria may be determined on a case-by-case basis.

1.2 The code year is established by the issue date of the project criteria documents. For modifications and additions to existing facilities, the Code of Record shall be established by the Design Authority in accordance with WSRC-TM-95-1 (Ref. 6.2.14.15). DOE Order 420.1B (Ref. 6.1.3) requires validation of existing designs, including designs with Natural Phenomena Hazards (NPH) assessments older than 10 years, against the current DOE natural phenomena hazard design criteria, or whenever significant changes in methodology or NPH inputs occur.

1.3 This Structural Design Criteria document applies to Performance Category 0 (PC-0) through Performance Category 4 (PC-4) and Seismic Design Category 1 (SDC-1) through Seismic Design Criteria 5 (SDC-5) structures. The NPH design criteria for Systems, Equipment and Components (SEC) are documented in SRS Engineering Standard 01061.

The relationship between the Performance Categories (PC) and the Seismic Design Categories (SDC) as defined by DOE-STD-1020 and DOE-STD-1189 is specified in the following equivalency table (Ref. 6.2.6.8)

Seismic Design Classifications

PC-1	is equivalent to	SDC-1, Limit State A
PC-2	is equivalent to	SDC-2, Limit State A
PC-3	is equivalent to	SDC-3, Limit State C For qualification of existing structures, Limit State C is applicable if the existing structure is capable of absorbing inelastic energy in accordance with ASCE 43 or DOE-STD-1020, otherwise Limit State D.
PC-4	is equivalent to	SDC-5, Limit State C

1.4 Notes on selected sections are given in Section 7.3, Appendix A. Appendix A provides background material, supplemental guidance, and comments on this Standard.

2.0 DOE ORDER AND STANDARDS APPLICABILITY

2.1 DOE Orders, DOE Guides and DOE Standards, as included in S/RID, applicable for the evaluation, modification or addition to SRS facilities are listed in Section 6.1.

2.2 Conflict between the (1) DOE Orders, DOE Guides and DOE Standards, as included in S/RID; (2) National Codes and Standards; and (3) this Engineering Standard; shall be brought to the attention of Engineering Standards Board's Civil/Structural Committee Chairman for resolution.

3.0 NATIONAL CODES AND STANDARDS APPLICABILITY

3.1 National Codes and Standards incorporated by reference in this document shall be the revision number and date at the time this document is invoked, or as otherwise noted. All references to ASCE 7 in this document pertain to ASCE 7-05, unless otherwise noted.

3.2 The design output documents shall specify the applicable Codes and Standards and their associated revision and date.

3.3 The applicability of the codes and standards given in this document is limited to the extent of the references in the text.

3.4.1 The International Building Code (IBC) (Ref. 6.2.9.1) is the basic structural building code at SRS unless otherwise noted in this document.

4.0 ACRONYMS & DEFINITIONS

4.1 ACRONYMS

The following is a list of acronyms and shortened titles used for the reference documents in this Engineering Standard.

AASHTO ....American Association of State Highway and Transportation Officials
ACI....................American Concrete Institute
AISC ....American Institute of Steel Construction
ANS ....American Nuclear Society
ANSI ....American National Standards Institute
APC...................Atmospheric Pressure Change
API ....American Petroleum Institute
AREMA ....American Railway Engineering and Maintenance-of-Way Association
ASCE ....American Society of Civil Engineers
ASD ....Allowable Stress Design
ASME ....American Society of Mechanical Engineers
AWWA ....American Water Works Association
CFR...................Code of Federal Regulation
CMAA...............Crane Manufacturers Association of America
CS ....Commercial Standard
DOE ....Department of Energy
DOE-STD ....Department of Energy Standard
FC ....Functional Classification
FEMA ....Federal Emergency Management Agency
HF ....Hazardous Facility
GS ....General Services
IBC....................International Building Code
IEEE..................Institute of Electrical and Electronics Engineers
LRFD ....Load Resistance and Factor Design
LS......................Hazardous Facility
NPH ....Natural Phenomena Hazard
OSHA................Occupational Safety and Health Administration
PC ....Performance Category
PS......................Production Support
R........................Response Modification Coefficient given in ASCE-7
R_a.......................Actual (Reduced) Response Modification Coefficient to be used in design
.... (Ra is defined in DOE Std. 1189)
S/RID ....Standards/Requirements Identification Document
SC ....Safety Class

SCDHEC...........South Carolina Department of Health and Environmental Control
SDC...................Seismic Design Category
SEC...................Systems, Equipment and Components
SI.......................Seismic Interaction
SLRS.................Seismic Load Resisting System
SRS...................Savannah River Site
SS......................Safety Significant
SSC...................Structures, Systems, and Components

4.2 DEFINITIONS

Key words and terms used in this document are defined as follows:

4.2.1 Natural Phenomena Hazard (NPH) - An act of nature (e.g., earthquake, wind, hurricane, tornado, flood, rain or snow precipitation, volcanic eruption, lightning strike, or extreme cold or heat) which threatens workers, the public, or the environment by potential damage to structures, systems, and components.

4.2.2 Performance Category (PC) - A classification based on a graded approach used to establish the non seismic NPH design and evaluation requirements for SSCs in accordance with DOE Standard 1021 (Ref. 6.1.2). The SSC Performance Category shall be identified in the design criteria input documents.

4.2.3 Seismic Design Category - A classification based on a graded approach (SDC-1 to SDC-5) used to establish the seismic NPH design and evaluation requirements for SSCs in accordance with ANSI/ANS 2.26 (Ref. 6.2.10.1). The SSC Seismic Design Criteria shall be identified in the design criteria input documents. This seismic design category is different from the seismic design category defined in the IBC (SDC-A to SDC-D). The IBC seismic design category will be noted with (IBC) to avoid confusion.

4.2.4 Limit State – The limiting acceptable deformation, displacement, or stress that an SSC may experience during or following an earthquake and still perform its safety function in accordance with ANS 2.26.

4.2.4.1 Limit State A – Large permanent distortion short of collapse and instability (uncontrolled deformation under minimal incremental load) but still able to perform its safety function and not impact the safety performance of other SCCs

4.2.4.2 Limit State B – Moderate permanent distortion but still able to perform its safety function. The acceptability of moderate distortion may include consideration of both structural integrity and leak-tightness.

4.2.4.3 Limit State C – Limited permanent distortion but still able to perform its safety function. The SSC is expected to undergo minimal damage during and following an earthquake such that no post earthquake repair is necessary. An SSC in this Limit State may perform its confinement function during and following an earthquake.

4.2.4.4 Limit State D – Essentially elastic behavior and able to perform its safety function during and following an earthquake. Gaseous, particulate, and liquid confinement is maintained. The SSC sustains no damage that would reduce its capability to perform its safety function.

4.2.5 Commercial Standard (CS) structure: Structures designed to the NPH and quality requirements of the International Building Code (PC-0, PC-1, PC-2, SDC-1, and SDC-2 categories)

4.2.6 Hazardous Facility (HF) structure: Structure, which due to the safety analysis, is credited with protecting the site worker or public from the effects of a nuclear or chemical accident (PC-3, PC-4, SDC-3, SDC-4, and SDC-5 categories)

4.2.7 Post-Seismic Differential Settlement – Unequal settling of soil under the foundation due to the effects of liquefaction and/or compression of soft zones after the seismic ground shaking has subsided.

5.0 REQUIREMENTS

The SSCs to be analyzed and designed in accordance with this standard are divided into two groups. The first of these is Commercial Standard (CS) structures. These structures include those with a Performance Category of PC-0, PC-1 or PC-2 or a Seismic Design Criteria of SDC-1 or SDC-2. Specific requirements for CS structures are provided in 5.1 and 5.3.

The second group is Hazardous Facility (HF) structures. These structures require a more robust design to protect both site personnel and the public from nuclear or chemical accident exposure. These structures have a NPH designation of PC-3, PC-4, SDC-3, SDC-4, or SDC-5. Specific requirements are provided in 5.2 and5.3.

Design requirements for a structure are driven by the Functional Classification (FC), safety function and Performance Category (PC), Seismic Design Category, and Limit State of the structure itself, and the Systems, Equipment and Components (SEC) it houses or supports. The Seismic Natural Phenomena Criteria matrix given in Table 7.1.2 provides the applicable structural design criteria in terms of SDC-1 through SDC-5 and the Limit State.

5.1 REQUIREMENTS FOR COMMERCIAL STANDARD STRUCTURES

CS structures shall be designed and constructed in accordance with the International Building Code (IBC), Ref. 6.2.9.1, as amended by the requirements of sections 5.1 and 5.3. These structures are identified by NPH designators PC-0, PC-1, PC-2, SDC-1, and SDC-2.

5.1.1 Classification of Structures for IBC Importance Factors

PC-1 and SDC-1 structures shall be classified as Occupancy Category I or II. PC-2 and SDC-2 structures shall be classified as Occupancy Category III or IV. Importance factors for structures for seismic, snow and wind loadings are provided in ASCE 7 for these categories. These categories shall supersede importance factors specified elsewhere in IBC and ASCE 7.

5.1.2 Design Loads

Design loads shall be per IBC except as modified below.

Note that PC-0 structures need not be designed for the effects of NPH loads.

5.1.2.1 Live Load (L) and Roof Live Load (Lr)

Live loads shall be as stipulated in IBC, (Ref. 6.2.9.1), AASHTO HB, (Ref. 6.2.1.1) or AREMA Manual (Ref. 6.2.5.1) as applicable. For structures exposed to wheel traffic, HS 20-44 (Ref. 6.2.1.1) truck loading shall be used, as a minimum, for wheel loading design. In areas subject to yard cranes, the crane wheel, track loading or outrigger loads during transportation, setup and lifts shall be considered.

For modifications or analysis of existing structures, loading based on the actual use of the structure may be used.

5.1.2.2 Rain Load (R) and Ponding Load (P)

Engineering Standard 01110 (Ref. 6.2.14.2) shall be used to quantify the rainfall event with 500 and 2000 year return periods for PC-1 and PC-2 structures, respectively. The effects of ponding shall be determined and included as the load P. The structural design of the roof system must satisfy design criteria for loads due to ponding that result from clogged/blocked drains and snow and ice loads. Ponding on the roof to the level of the secondary roof drainage system shall be considered.

The minimum drainage system design shall be for a 25-year, 6-hour rainfall event (4.5 inches total accumulation per Reference 6.2.14.8) at all performance categories.

5.1.2.3 **Wind Load (W)**

The basic wind speed (3-second gust) for SRS shall be taken as 100 mph for PC-1 and PC-2 structures. Wind loads shall be calculated using exposure "C" with wind importance factors defined in Section 5.1.1.

Atmospheric ice loads shall be determined in accordance with ASCE 7 for ice sensitive structures. Load combinations shall be modified in accordance with ASCE 7. Ice loads consist of the dead load of the ice with a concurrent wind load. The increase in projected area from the ice accretion on structural members shall be accounted for when determining the wind load on the members. The ASCE 7 structure category for importance factors is defined in Section 5.1.1.

5.1.2.4 **Earthquake Load (E, E_m)**

For SDC-1 and SDC-2 structures regulated by SCDHEC (Ref. 6.2.15.1), the SDC-2 requirements shall be used for earthquake load.

Seismic response coefficients are determined in accordance with IBC and ASCE 7 except the response modification coefficient (R) shall be as given in Table 7.1.2 for the required Limit State. Seismic ground motion design coefficients for SRS, that meet the IBC criteria, are developed in A5.1.2.4.

In the application of IBC earthquake requirements to building structures, Soil Structure Interaction (SSI) analysis is generally not required, and the In-structure Response Spectra (IRS) are implicitly provided through the formulation of seismic forces on components, F_P.
Per IBC 2009 Section 2205.2.1, steel structures in SDC-C (IBC) shall be designed and detailed in accordance with the provisions of AISC 341 if R > 3. Per IBC 2009 Section 2205.2.2, all steel structures in SDC-D (IBC) shall be designed and detailed in accordance with AISC 341 regardless of R except as permitted in ASCE 7, Table 15.4-1 and 15.4-2.

5.1.2.5 **Self Straining Force (T)**

Self straining forces required by IBC shall be considered. Buildings and structures shall be designed for the total and differential foundation settlements (T_Δ), resulting from the combined static and dynamic loads if the settlement has the potential to challenge the confinement function of the building or structure as credited in safety documentation or delineated in design criteria input documents. Examples of the potential to challenge the confinement function are given in the Appendix.

5.1.2.6 **Flood Load (F_a)**

The structure shall be designed for the flooding and wave action consequences (the Design Flood in IBC 2009 1612) associated with flooding events with return periods of 500 or 2000 years for PC-1 or PC-2 respectively. Loads resulting from flooding and wave action shall be considered for each SRS area in accordance with the flood hazard curves provided in Reference 6.2.14.12.

5.1.3 **Tornado Shelter Requirements**

Permanent PC-1 and PC-2 structures shall contain one or more safe rooms as Tornado Shelters. The total floor square footage of the safe rooms shall not be less than 5 sq. ft./person times the normal occupancy of the facility plus a provision for facility guests. The Tornado Shelter requirement shall be considered applicable to permanent facilities with both (1) a design life greater than 5 years; and (2) permanently assigned personnel.

The life safety design of Tornado Shelters in the interior of new PC-1 and PC-2 structures may be accomplished by choosing structural element sizes and details in accordance with FEMA 320 (Ref. 6.2.18.1), FEMA 361 (Ref. 6.2.18.2), or alternatively by designing the shelters for the following loads:

- 161 mph tornado wind speed, with exposure category C, and
- tornado missile: 2x4 timber plank 15 lb. @ 100 mph (horiz); max height 150 ft; 70 mph (vert).

Existing PC-1 and PC-2 structures may be evaluated as Tornado Shelters in accordance with the methodology used in Ref. 6.2.14.13.

5.1.4 **Seismic Structural Interaction (SI)**

The functional and spatial interaction of Systems, Structures and Components (SSC) and their effects on each other shall be designed so that the failure of a lower SDC shall not cause the failure, or adversely affect the function, of a higher safety related SDC SSC. SI includes equipment/structural seismic interaction commonly referred to as II/I.

5.1.5 **Other Structures**

The structures identified below shall be designed using the codes and standards given in the following table in lieu of IBC. The codes and standards in the following table shall be used in their entirety, including loads and load combinations. If an individual code or standard does not consider NPH loads, then the structure shall be designed to resist that specific IBC NPH load using IBC load combinations. The importance factor defined in Section 5.1.1 shall be used with the applicable code and standard.

Structure	Code/Standard
Highway Structures	AASHTO HB (Ref. 6.2.1.1)
Railway Structures	AREMA Manual (Ref. 6.2.5.1)
Stainless Steel Sections and Plates	ASCE 8 (Ref. 6.2.6.5)
Environmental Engineering Concrete Structures	ACI 350 (Ref. 6.2.2.5)
Concrete Chimneys	ACI 307 (Ref. 6.2.2.6) Note: All vertical reinforcement and dowel bars shall be fully developed.
Steel Stacks	ASME STS-1 (Ref. 6.2.12.1)
Shoring	IBC and OSHA 29 CFR Part 1926 (Ref. 6.2.11.1)
Crane Runway and Supporting Structures	CMAA Specification #70 (Ref. 6.2.8.1) or CMAA Specification #74 (Ref. 6.2.8.2)

5.2 REQUIREMENTS FOR HAZARDOUS FACILITY STRUCTURES

Hazardous Facility (HF) structures housing radiological and chemical hazards that have been identified with an NPH designation of PC-3, PC-4, SDC-3, SDC-4, or SDC-5 shall be designed and constructed in accordance with the requirements of 5.2 and 5.3.

5.2.1 **Design Loads**

Structures shall be designed for the loads prescribed in this standard and as supplemented by project specific criteria.

5.2.1.1 **Dead Load (D)**

Dead loads are loads that remain permanently in place and include (1) the self weight of the structure, cladding, roofing, ceilings, architectural finishes; (2) built-in partitions; (3) stairs; and (4) fixed equipment, systems and components.

5.2.1.2 **Live Load (L) and Roof Live Load (L_r)**

Live loads (L) are those loads produced by the use and occupancy of a building or other structure. Live loads on a roof (L_r) are those produced (1) during maintenance by workers, equipment, and materials and (2) during the life of the structure by movable objects such as temporary equipment. Also considered as live loads are the dynamic effects of operating equipment (such as cranes and pumps).

Live loads shall be as stipulated in ASCE 7, (Ref. 6.2.6.2), AASHTO HB, (Ref. 6.2.1.1) or AREMA Manual (Ref. 6.2.5.1) as applicable. For yard structures exposed to wheel traffic, HS 20-44 (Ref. 6.2.1.1) truck loading shall be used, as a minimum, for wheel loading design. In areas subject to yard cranes, the crane wheel, track loading or outrigger loads during lifts shall be considered.

For modifications or analysis of existing structures, loading based on the actual use of the structure may be used.

5.2.1.3 **Soil Load (H)**

Structures and elements of structures retaining soil shall be designed for (1) lateral earth pressure; (2) compaction loads; (3) any surcharge load; and (4) any hydrostatic pressure corresponding to the maximum probable groundwater level. The at-rest lateral soil pressure, K_o, shall be used unless the structure is flexible enough to develop active soil pressure, K_a.

5.2.1.4 **Fluid Load (F)**

The fluid load is the load resulting from the pressure of the fluid.

5.2.1.5 **Rain Load (R) and Ponding Load (P)**

Engineering Standard 01110 (Ref. 6.2.14.2) shall be used to quantify the rainfall event with 10,000 and 100,000 year return periods for PC-3 and PC-4 structures, respectively. The effects of ponding shall be determined and included as the load P. The structural design of the roof system must satisfy design criteria for loads due to ponding that result from clogged/blocked drains and snow and ice loads. Ponding on the roof to the level of the secondary roof drainage system shall be considered.

The minimum drainage system design shall be for a 25-year, 6-hour rainfall event (4.5 inches total accumulation per Reference 6.2.14.8) at all performance categories.

5.2.1.6 **Snow Load (S)**

The product of the Design Ground Snow Load and importance factor, $I \times p_g$, shall be taken as 10 psf for PC-3 structures and 15 psf for PC-4 structures. Snow loads shall be calculated in accordance with ASCE 7. An unheated building, or a building with a failed heating system, shall be assumed when determining the thermal factor C_t. The exposure factor, C_e, shall be conservatively estimated. Nearby structures, exposed mechanical equipment, distribution systems, stacks, etc. shall be considered in the exposure factor.

5.2.1.7 **Wind Load (W)**

Wind load design for buildings and other structures shall be determined in accordance with the procedures in ASCE 7, using exposure "C", with the basic wind speeds given in Table 7.1.1. The effect of the importance factor "I" is incorporated in the tabulated values for the basic wind speeds; "I" shall be taken as 1.0.

The design shall include consideration of wind driven missiles given in Table 7.1.1. The barrier thicknesses to preclude damage from a wind driven 2x4 missile are provided in DOE-STD-1020. Barriers other than those given in DOE-STD-1020 may be analyzed or designed per the missile barrier criteria given in ASCE-58 (Ref. 6.2.6.3).

Atmospheric ice loads, shall be determined in accordance with ASCE 7 for ice sensitive structures with the following modification to ASCE 7 Equation 10-5:

The term (2t) shall be replaced with 1.875 inches for PC-3 and 2.5 inches for PC-4.

Load combinations shall be modified in accordance with ASCE 7. Ice loads consist of the dead load of the ice with a concurrent wind load. The increase in the projected area from the ice accretion on structural members shall be accounted for when determining the wind load on the members. Wind speeds at 33 feet above grade with the ice loads shall be taken as 40 mph for PC-3 and 48 mph for PC-4. The importance factors, I_i and I_w, shall be taken as 1.0 for PC-3 and PC-4 structures. It should be noted that ice accretion and wind loads vary according to the height above or below 33 feet above grade.

5.2.1.8 **Tornado Load (W_t)**

Tornado wind load design W_t, shall be determined in accordance with the procedures in ASCE 7 (Ref. 6.2.6.2), using exposure "C", with the basic wind speed obtained from Table 7.1.1. The directionality factor, K_d, shall be taken as 1.0. The importance factor "I" shall be taken as 1.0.

Tornado driven missile criteria are given in Table 7.1.1. The barrier thicknesses to preclude damage from the 2x4 and 3" pipe tornado missiles are provided in DOE-STD-1020. Barriers other than those given in DOE-STD-1020 may be analyzed or designed per the missile barrier criteria given in ASCE-58 (Ref. 6.2.6.3).

Tornado wind pressure load and missile effects shall be combined for design and evaluation purposes, where required.

APC shall apply for enclosed structures as provided in DOE-STD-1020. Partially enclosed or open structures shall follow the provisions in ASCE 7.

5.2.1.9 **Earthquake Load (E)**

Earthquake loads shall be determined in accordance with DOE-STD-1189, ASCE 4 and ASCE 43. The site specific horizontal and vertical response spectra for SDC-3 are given in Table 7.1.3 and Figure 7.2.1. The SDC-4 spectra is determined by multiplying the SDC-3 spectra by a factor of 1.25. The spectra given in Table 7.1.3 and Figure 7.2.1 are considered "preliminary" in accordance with E7 Manual. To become "confirmed" spectra, SRS Geotechnical Engineering must review the facility specific soil conditions.

SDC-5 horizontal and vertical response spectra for SRS are not available at the current time. Contact SRS Geotechnical Engineering for location specific SDC-5 response spectra if required.

For SDC-3 and SDC-4 structures regulated by Ref. 6.2.15.1, the earthquake loads given in this standard shall be considered to meet the seismic loading requirements of SCDHEC.

5.2.1.10 **Self Straining Loads (T, T_o, T_a & T_Δ)**

The following types of self straining loads (T, unless otherwise noted) shall be considered when appropriate: thermal loads, moisture change, creep and shrinkage, and settlement, or combination thereof.

5.2.1.10.1 **Thermal Loads (T_o, T_a)**

The design of structures shall consider the effects of stresses and movements resulting from variations in temperature due to normal operation, T_o, and accident loads, T_a.

5.2.1.10.2 **Settlement (T_Δ) and Dynamic Settlement (E_Δ)**

Buildings and structures shall be designed for the total and differential foundation settlements resulting from the combined static and dynamic loads, T_Δ. The dynamic settlement, E_Δ, is due to dissipation of pore pressure and redistribution of soil stresses from the effects of a design basis earthquake. The dynamic settlement, E_Δ, is not taken concurrent with seismic inertial loads, E. The magnitude of foundation settlements shall be developed from geotechnical investigations.

5.2.1.10.3 **Moisture Change, Creep and Shrinkage**

Concrete and masonry structures shall be investigated for stresses and deformations induced by moisture change, creep and shrinkage.

5.2.1.11 **Accident Load (P_a)**

The consequences of a design basis accident resulting in internal pressurization shall be considered.

The consequences of accidental explosions (internal or external to the facility) and generated missiles shall be considered in the design or evaluation of facilities.

The consequences of potential heavy load drops shall be considered in the design or evaluation of structures.

5.2.1.12 **Pipe Reactions (R_o, R_a)**
Effects of pipe reactions on structures due to normal operation, R_o and accident conditions, R_a, shall be considered.

5.2.1.13 **Pipe Break Load (Y)**
Effects of pipe breaks on structures, including reaction, jet, and movement, shall be considered.

5.2.1.14 **Flood Load (F_a)**
The structure shall be designed for the flooding and wave action consequences associated with flooding events with return periods of 10,000, or 100,000 years for PC-3, or PC-4 respectively per DOE-STD-1020. Loads resulting from flooding and wave action shall be considered for each SRS area in accordance with the flood hazard curves provided in Reference 6.2.14.12.

5.2.2 **Structural Design**
The demand, calculated using the loads in Section 5.2.1 and the load combinations in the specific codes identified below, shall be less than the capacity calculated per ASCE 43 as amended by the requirements of sections 5.2.2.1 through 5.2.2.6.

Earthquake demand loads shall be calculated using dynamic analysis procedures of ASCE 4 (Ref. 6.2.6.1). Soil Structure Interaction (SSI) analysis shall be performed and In-structure Response Spectra (IRS) shall be obtained, when required, in accordance with requirements of ASCE 4.

5.2.2.1 **Concrete Structures**
Concrete structures shall be designed using the loadings in 5.2.1 and the requirements of ACI 349 as amended below as an exception to ASCE 43-05, Section 4.2.2 and 4.2.3.

The seismic loading defined in 5.2.1 shall be considered the Safe Shutdown Earthquake (SSE). Modify ACI 349-06 load combination equations as follows:

- Delete E_0 from equation (E_0 is not defined for DOE facilities)
- For new structures replace E_{SS} with (1.2E or $1.2E_\Delta$)
- For existing structures replace E_{SS} with (E or E_Δ)
- The factored Live Load, γL, shall represent the expected (mean) live load acting when the earthquake occurs and shall not be less than 0.25L.

The provisions for seismic design of ACI 349 (Chapter 21 in ACI 349-06) shall be used.

Anchorage design shall be in accordance with ACI 349-06 Appendix D. Site specific guidance is given in SRS Guide 03251-G. Alternately, the capacity of existing anchorage may be quantified using DOE/EH-0545.

SRS quality criteria shall be used in lieu of the quality provisions of ACI 349-06, Section 1.5.

5.2.2.2 **Steel Structures**
Sections 5.2.2.2.1 through 5.2.2.2.3 shall be used to design steel structures as applicable as an exception to ASCE 43-05, Section 4.2.4

5.2.2.2.1 **AISC 360 (LRFD and ASD)**
When using AISC-360, steel structures shall be designed using the loadings in 5.2.1, the load combinations of ASCE 7 Section 2.3 or 2.4, as amended below, and the requirements of AISC 360.

For the LRFD method, modify the load combinations in ASCE 7 Section 2.3.2, Equations 5 and 7 as follows:

- For new structures replace E with (1.2E or $1.2E_\Delta$)
- For existing structures replace E with (E or E_Δ)

- Replace 1.2D with 1.0D and 1.0L with 0.8L
- The factored Live Load, γL, shall represent the expected (mean) live load acting when the earthquake occurs and shall not be less than 0.25L.

Add the load combination $D+0.8L+R_o+T_o+W_t$ for tornado wind loads.

For the ASD method, modify the load combinations in ASCE 7 Section 2.4.1, Equations 5, 6 and 8 as follows:

- For new structures replace 0.7E with ($1.2x0.7E$ or $1.2x0.7E_\Delta$)
- For existing structures replace 0.7E with (0.7E or $0.7E_\Delta$)

The live load in Equation 6 shall represent the expected (mean) live load acting when the earthquake occurs and shall not be less than 0.25L.
Add the load combination $D+0.8L+R_o+T_o+0.7W_t$ for tornado wind loads.

5.2.2.2.2 **ASCE 8**

When using ASCE 8, stainless steel structures shall be designed using the loadings in 5.2.1, the load combinations of ASCE 7 Section 2.3 as amended below, and the requirements of ASCE 8.

Modify the load combinations in ASCE 7 Section 2.3.2, Equations 5 and 7 as follows:

- For new structures replace E with (1.2E or $1.2E_\Delta$)
- For existing structures replace E with (E or E_Δ)
- Replace 1.2D with 1.0D and 1.0L with 0.8L
- The factored Live Load, γL, shall represent the expected (mean) live load acting when the earthquake occurs and shall not be less than 0.25L.

Add the load combination $D+0.8L+R_o+T_o+W_t$ for tornado wind loads.

5.2.2.2.3 **ANSI/AISC N690**

When using ANSI/AISC N690, steel structures shall be designed using the loadings in 5.2.1 and the requirements of ANSI/AISC N690 as amended below.

For the LRFD method, modify equations as follows:

- Delete equation containing the Operating Basis Earthquake, (E_0 is not defined for DOE facilities)
- For new structures replace E_s with (1.2E or $1.2E_\Delta$)
- For existing structures replace E_s with (E or E_Δ)

For the ASD method, modify as follows:

- Delete equation containing the Operating Basis Earthquake, (E_0 is not defined for DOE facilities)
- For new structures replace E_s with (1.2E or $1.2E_\Delta$)
- For existing structures replace E_s with (E or E_Δ)

SRS Quality Criteria shall be used in lieu of the quality provisions of ANSI/AISC N690-06 Section NA5 and NM5.

5.2.2.2.4 **ANSI/AISC 341**

The Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341 shall be used for SDC-3, -4 and -5 steel structures when the inelastic energy absorption factor, F_μ, is taken greater than 1.0. Exception to this stipulation may be taken for qualification of existing structures provided F_μ is applied to a limited number of members in the structure and the connections of those members are capable of absorbing inelastic energy in accordance with ASCE 43 or DOE-STD-1020.

5.2.2.3 **Foundation Design**

Buildings and structures shall be designed to resist bearing failure, unacceptable settlement, sliding and overturning, and buoyancy. Soil properties required for design shall be developed from geotechnical investigations.

5.2.2.3.1 **Bearing Capacity**

When using ACI 349, the required soil bearing capacity for the foundation shall be computed using the loading combinations in ACI 349-06, Section 9.2.1 as amended below:

Modify ACI 349-06, Section 9.2.1, Equations 9-4, 9-6 and 9-9 as follows:

- Delete E_0 from equation 9-4 (E_0 is not defined for DOE facilities)
- For new structures, replace E_{SS} with 1.2E.
- The factored Live Load γL, shall represent the expected (mean) live load acting when earthquake occurs and shall not be less than 0.25L

The required soil bearing capacity shall be less than or equal to the design soil bearing capacity. The design soil bearing capacity is equal to the ultimate bearing capacity multiplied by a strength reduction factor. The ultimate bearing capacity and the strength reduction factor shall be based on the results of the geotechnical investigation.

Alternatively, when the allowable stress method is used, required soil bearing capacity for the foundation shall be computed using the loading combinations in ASCE 7, Section 2.4.1. For new structures, replace E in ASCE 7, Section 2.4.1 Equations 5, 6, and 8 with 1.2E. The required soil bearing capacity shall be less than or equal to the allowable soil bearing capacity. The allowable bearing capacity is equal to the ultimate bearing capacity divided by a safety factor of 3.

5.2.2.3.2 **Settlement**

Foundation settlement, including static and dynamic, total and differential settlements, shall be estimated based on facility specific soil conditions. Foundation settlement shall be considered as stated in Section 5.2.1.10.2.

5.2.2.3.3 **Sliding and Overturning**

Foundations shall be designed to resist sliding and overturning per requirements in ASCE 43 (Ref. 6.2.6.8).

5.2.2.3.4 **Buoyancy**

Foundations shall be designed to resist buoyancy. The required buoyancy resistance shall be greater than or equal to 110% of the flood load F_a.

5.2.2.4 **Deleted**

5.2.2.5 **Design of Unanchored Components**

Mobile and/or temporary components may be unanchored provided the requirements of either 5.2.2.5.1 or 5.2.2.5.2 are met.

5.2.2.5.1 **No Sliding or Overturning**

The nominal sliding/overturning resistance, R, shall be greater than 110% of the seismic, FE, sliding/overturning force. Seismic sliding/overturning forces shall be increased by 20% for new SSC. Similarly, the nominal sliding/overturning resistance shall be greater than 110% of the tornado, FT, sliding/overturning force. The nominal sliding/overturning resistance shall also be greater than 150% of the wind, FW, sliding/overturning force.

The nominal sliding resistance shall be the component's normal force times the static coefficient of friction. The normal force shall be the contact force between the component and the sliding plane, which is generally the component weight reduced to account for (1) vertical acceleration; (2) suction due to wind or tornado forces; or (3) buoyancy. For wind and tornado loading, 90% of the component weight shall be used.

For sliding, the coefficient-of-sliding-friction shall be set at the 95% exceedance level. For rocking, the coefficient-of-sliding-friction shall be set at the 5% exceedance level. Alternately, pure rocking (i.e., no slide-rock) can be assumed (Ref. 6.2.6.8). A concrete or steel rigid body sliding on a dry broom-finished concrete surface has a 95% exceedance level coefficient-of-sliding-friction of 0.3 and a 5% exceedance level coefficient-of-sliding-friction of 0.7.

5.2.2.5.2 **Potential Sliding or Overturning**

Alternately, the component may slide and/or overturn provided (1) the safety function of the component is maintained; and (2) the safety function of all adjacent components is maintained.

Median centered analysis techniques shall be used so as to predict "best-estimate" values of sliding distance and rocking angle. "Best-estimate" values of sliding distance or rocking angle may either be determined by time-history analysis, or by the "approximate methods" (Ref. 6.2.6.8). Seismic sliding/overturning forces shall be increased by 20% for new SSC.

These "best-estimate" values shall be increased by a factor of safety of 3 for sliding and 2 for rocking in order to obtain design values of sliding and rocking, except that the design value of sliding does not need to exceed 1.5 times the peak displacement of the input motion.

To avoid impact, sufficient clearances shall be provided to accommodate the design values of both sliding and rocking. However, these design values of sliding and rocking do not have to be combined. In addition, the design value of rocking angle shall not exceed the instability angle α defined by ArcTan (b/h), where b is the minimum horizontal distance from the edge of the body to the center-of-gravity, and h is height of the center-of-gravity height.

5.2.2.6 **Other Structures**

The structures/components below shall be designed with the applicable code/standard using the loadings in 5.2.1, the analysis procedures of the applicable code/standard, the dynamic analysis procedures of ASCE 4 for seismic loads, and the load combinations of the applicable code/standard, as amended below:

- Replace the seismic load E with (1.2E or $1.2E_{\Delta}$) for new structures
- Replace the seismic load E with (1.0E or $1.0E_{\Delta}$) for existing structures
- Replace the load factor on dead load, 1.2D, with 1.0D for seismic loading combinations where applicable
- Add the load combination $D+0.8L+R_o+T_o+W_t$ for tornado wind loads.

Procedures for straight wind load calculation in these codes shall be used for tornado wind load calculations.

Structure/Component	**Code/Standard**
Concrete Chimneys	ACI 307 (Ref. 6.2.2.6) Note: All vertical reinforcement and dowel bars shall be fully developed.
Steel Stacks	ASME STS-1 (Ref. 6.2.12.1) Note: The allowable stress may be increased by 20% for tornado pressure loads.
Crane runway and supporting structures	CMAA Specification #70 (Ref. 6.2.8.1) or CMAA Specification #74 (Ref. 6.2.8.2) or ASME NOG 1 (Ref. 6.2.12.2)

5.3 MISCELLANEOUS REQUIREMENTS

5.3.1 **Modifications to Existing Structures**

For additions or modifications to existing facilities, the effects of new loads transmitted to the existing structures shall be considered. Modifications and additions shall not, as a minimum, degrade the originally required performance of existing structures to the extent that they will not withstand NPH loads, provide confinement, or provide safe operation of essential facilities, protection of government property and the protection of life safety for occupants.

5.3.2 **Steel Erection Requirements**

The Occupational Safety and Health Administration (OSHA) issued Safety Standards for Steel Erection, 29 CFR 1926, Subpart R on January 18, 2001, which contractors and subcontractors at SRS are required to follow in accordance with 10 CFR 851 (Ref. 6.1.11). The subpart includes design requirements, among others, for columns to be anchored with a minimum of four bolts.

5.3.3 **Fire Enclosure Evaluations of Concrete Elements**

In structural evaluations of concrete elements for fire events, where necessary, the provisions found in ACI 216M (Ref. 6.2.2.1) shall be followed.

5.3.4 **Anchorage to Concrete**

Information useful for the design of cast-in-place anchors and post-installed anchors in concrete is provided in Engineering Guide 03251-G (Ref. 6.2.14.4). Information useful for the installation and testing of post-installed anchors in concrete is provided in 03252-G (Ref. 6.2.14.5).

For SDC-1 and SDC-2 systems and components, anchorage to concrete or masonry shall be designed for 1.3 times the prescribed seismic force in the connected part given in 13.3.1 of ASCE 7 unless it exceeds the force given in item a or c of 13.4.2 of ASCE 7.

5.3.5 **Coring, Chipping, and Drilling in Concrete**

Coring, chipping and drilling in concrete elements or structures shall be per Engineering Standard 03010 (Ref. 6.2.14.6) in accordance with the Structural Requirements Matrix in A5.0.

5.3.6 **Concrete for Confinement**

Concrete used for confinement shall consider the strategies given in Ref. 6.2.6.6 and provisions of ACI 350, Ref. 6.2.2.5.

5.3.7 **Lift Points**

- Rigging attachment and lift points are integral to the item being lifted, and include those for structural elements such as cell covers, plugs, etc. that will be lifted into place during construction or that will be routinely lifted during their operating life. Structural assemblies are “below-the-hook” but above the item being lifted including embedments, and include spreader beams, frames, etc. Rigging attachment and lift points, and structural assemblies for lifting shall be designed in accordance with the applicable material code (ACI, AISC, etc.) with the following clarifications and additional requirements: Rigging attachment and lift points shall be designed for the factored dead load of the item times a dynamic load factor (DLF). Concrete anchorage of rigging attachment and lift points shall be designed for the above load in accordance with ductile or non-ductile provisions of ACI 318 Appendix D, or ACI 349 Appendix D, as applicable.
- Structural assemblies that are subjected to periodic lift tests (load test) at 125% of the actual component weights in accordance with the SRS Hoisting and Rigging Manual WSRC-TM-90-7 (Ref. 6.2.14.16) shall be designed for 1.25 times the rated lift load plus the factored dead load of the assembly, times a dynamic load factor (DLF), i.e., DLF x (1.25 x rated lift load + code factor x dead load of the structural assembly).
- The dynamic load factor (DLF) shall be taken as 2.0 unless otherwise justified. See CMAA # 70 and 74 (Ref. 6.2.8.1 and 6.2.8.2). The load case with DLF need not be combined with NPH events.

Procured load rated items for hoisting and rigging shall meet a factor of safety of 5 on ultimate strength or 3 on yield strength in accordance with the SRS Hoisting and Rigging Manual WSRC-TM-90-7 (Ref. 6.2.14.16).

5.3.8 **Fragility Analysis**

Existing structures that do not meet the deterministic limits of this standard may be further evaluated using failure limits and variabilities calculated for the given structures in accordance with DOE-STD-1020 and ASCE 43.

6.0 REFERENCES

6.1 DOE ORDERS AS INCLUDED IN S/RID, AND DOE STANDARDS AND MEMORANDA

6.1.1 DOE-STD-1020-02- NPH Design and Evaluation Criteria for DOE Facilities, January 2002.

6.1.2 DOE-STD-1021-93 - NPH Performance Categorization Guidelines for Structures, Systems, and Components, Change Notice 1, January 1996.

6.1.3 DOE Order 420.1B - Facility Safety, December 2005.

6.1.4 DOE G 420.1-2, Guide for the Mitigation of Natural Phenomena Hazards for DOE Nuclear Facilities and Non-Nuclear Facilities, March 28, 2000.

6.1.5 A Memorandum of January 22, 1998 from DOE to the Office of Nuclear Safety Policy and Standards: H. Chander, Newsletter (Interim Advisory on Straight Winds and Tornados).

6.1.6 DOE-STD-1090-2007 – Hoisting and Rigging, Change Notice 1, December 2007.

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

6.1.8 DII 420.1.1.A – Natural Phenomena Hazards (NPH) Mitigation, DOE-SR Directive Implementation Instructions, 3/29/99.

6.1.9 DOE G 420.1-1, Nonreactor Nuclear Safety Design Criteria And Explosives Safety Criteria Guide For Use With Doe O 420.1, March 28, 2000.

6.1.10 DOE –STD-1189-2008 Integration of Safety into the Design Process, March 2008

6.1.11 10 CFR 851 Worker Safety and Health Program

6.2 CODES AND STANDARDS

6.2.1 American Association of State Highway and Transportation Officials (AASHTO)

6.2.1.1 Standard Specification for Highway Bridges (AASHTO HB)

6.2.2 American Concrete Institute (ACI)

6.2.2.1 ACI 216M Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies

6.2.2.2 Deleted

6.2.2.3 Deleted

6.2.2.4 ACI 349 Code Requirements for Nuclear Safety Related Concrete Structures

6.2.2.5 ACI 350 Code Requirements for Environmental Engineering Concrete Structures

6.2.2.6 ACI 307 Code requirements for Reinforced Concrete Chimneys

6.2.2.7 ACI 318 Building Code Requirements for Structural Concrete and Commentary

6.2.2.8 Deleted

6.2.3 American Institute of Steel Construction (AISC)

6.2.3.1 ANSI/AISC N690 Specification for Safety-Related Steel Structures for Nuclear Facilities

6.2.3.2 Deleted

6.2.3.3 AISC 360 Specifications for Structural Steel Buildings

6.2.3.4 Deleted

6.2.3.5 Deleted

6.2.3.6 ANSI/AISC 341 Seismic Provisions for Structural Steel Buildings

6.2.4 **American Petroleum Institute (API)**

6.2.4.1 API 650 Welded Steel Tanks for Oil Storage

6.2.4.2 API 620 Design and Construction of Large, Welded, Low Pressure Storage Tanks

6.2.5 **American Railway Engineering and Maintenance-of-Way Association (AREMA)**

6.2.5.1 AREMA Manual for Railway Engineering, Volume I and II

6.2.6 **American Society of Civil Engineers (ASCE)**

6.2.6.1 ASCE 4 Seismic Analysis of Safety Related Nuclear Structures

6.2.6.2 ANSI/ASCE 7 Minimum Design Loads for Buildings and Other Structures

6.2.6.3 ASCE 58 Structural Analysis and Design of Nuclear Plant Facilities

6.2.6.4 Deleted

6.2.6.5 SEI/ASCE 8 Specification for the Design of Cold-Formed Stainless Steel Structural Members

6.2.6.6 Concrete Watertight Structures and Hazardous Liquid Containment, Robert Hengst, ASCE Press, 1994

6.2.6.7 Deleted

6.2.6.8 ASCE 43 Seismic Design Criteria For Structures, Systems and Components In Nuclear Facilities And Commentary

6.2.7 **American Water Works Association (AWWA)**

6.2.7.1 AWWA D100 Welded Steel Tanks for Water Storage

6.2.8 **Crane Manufactures Association of America (CMAA)**

6.2.8.1 CMAA Specification #70, Electrical Overhead Traveling Cranes

6.2.8.2 CMAA Specification #74, Specifications for Top Running and Under Running Types of Single Girder Electric Overhead Traveling Cranes

6.2.9 **International Code Council (ICC)**

6.2.9.1 International Building Code (IBC)

6.2.9.2 AC 156, Acceptance Criteria for Seismic Qualification by Shake Table Testing of Nonstructural Components and Systems

6.2.10 **American Nuclear Society**

6.2.10.1 ANSI/ANS 2.26 Categorization of Nuclear Facility Structures, Systems, and Components for Seismic Design

6.2.11 **Occupational Safety and Health Administration (OSHA)**

6.2.11.1 29 CFR Part 1926 OSHA Safety and Health Standards

6.2.12 **American Society of Mechanical Engineers**

6.2.12.1 ASME STS-1 Steel Stacks

6.2.12.2 ASME NOG 1 Rules For Construction Of Overhead And Gantry Cranes

6.2.13 **Institute of Electrical and Electronics Engineers**

6.2.13.1 IEEE 344-1987, IEEE Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations

6.2.14 **SRS Standards, Guides, Reports and Calculations**

6.2.14.1 Engineering Standard 01061, Qualification of Systems, Equipment and Components for Natural Phenomena Hazards

6.2.14.2 Engineering Standard 01110, Civil Site Design Criteria

6.2.14.3 SRS Seismic Response Analysis and Design Basis Guidelines, WSRC-TR-97-0085, Rev. 0, R. C. Lee, M. E. Maryak and M. D. McHood, March 1997

6.2.14.4 Engineering Guide 03251-G, Cast-In and Post-Installed Anchors in Concrete

6.2.14.5 Engineering Guide 03252-G, Installation and Testing of Concrete Anchors

6.2.14.6 Engineering Standard 03010, Coring, Chipping and Drilling in Concrete

6.2.14.7 Engineering Guide, 11520-G, Evaluation of Seismic Spatial Interactions Between Facility Structures, Systems, and Components

6.2.14.8 Tornado, Maximum Wind Gust, and Extreme Rainfall Event Occurrence Frequencies at the Savannah River Site (U), WSRC-TR-98-00329, September 1998.

6.2.14.9 Deleted

6.2.14.10 Tornado Hazard Assessment, Memo from McDonald-Mehta Engineers to Brent Gutierrez, November 9, 1997.

6.2.14.11 "Revised Envelope of the Site Specific PC-3 Surface Ground Motion", Memo from Brent Gutierrez to Lawrence Salomone and Fred Loceff, September 9, 1999.

6.2.14.12 Flood Hazard Recurrence Frequencies for A-, K- and L-Areas and Revised Frequencies for C-, F-, E-, S-, H-, Y-, and Z-Areas, WSRC-TR-2000-00206, June 30, 2000.

6.2.14.13 Evaluation of SRS Buildings for Tornado Shelter Protection, WSRC T-TRT-G-00001, Revision 2, April 1, 1998.

6.2.14.14 Engineering Guide 15060-G, Application of ASME B31.3

6.2.14.15 Responsibilities and Requirements, SRS Engineering Standards Program, WSRC-TM-95-1

6.2.14.16 Savannah River Site Hoisting and Rigging Manual, WSRC-TM-90-7

6.2.14.17 Generation of SRS SDC-1 to SDC-5 Response Spectra, T-CLC-G-000312

6.2.14.18 SCD-11, Consolidated Hazard Analysis Process (CHAP) Program & Methods Manual

6.2.15 **State of South Carolina, Department of Health and Environmental Control (SCDHEC)**

6.2.15.1 Final Regulation, Department of Health and Environmental Control, Chapter 61, 61-104, Hazardous Waste Management Location Standards, Statutory Authority: 1976 Code Section 44-56-30, -35, et. seq.

6.2.16 **U.S. Nuclear Regulatory Commission (USNRC)**

6.2.16.1 NUREG-0800, Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants, U.S. NRC

6.2.17 **DOE National Labs**

6.2.17.1 "Final Draft, Development Of A Probabilistic Tornado Wind Hazard Model For The Continental United States," Hazards Mitigation Center Lawrence Livermore National Laboratory, July 20, 2000.

6.2.18 **Federal Emergency Management Agency, FEMA**

6.2.18.1 Taking Shelter From the Storm: Building a Safe Room Inside Your House, FEMA 320, First Edition, Federal Emergency Management Agency, October 1998, includes FEMA 320 A, In-Residence Shelter Construction Drawings.

6.2.18.2 Design and Construction Guidance for Community Shelters, FEMA 361.

7.0 TABLES, FIGURES AND APPENDIX A

7.1 TABLES

7.1.1 Non Seismic Natural Phenomena Hazards Criteria

7.1.2 Seismic Natural Phenomena Hazards Criteria

7.1.3 SDC-3 Site Specific Spectra

7.2 FIGURES

7.2.1 Site Specific Spectra, SDC-3 Horizontal and Vertical Spectra, 5% Damping,

TABLE 7.1.1 – Non Seismic Natural Phenomena Hazards Criteria

	Performance Category	PC-3	PC-4
	Performance Goal Annual Probability of Exceedance	$1x10^{-4}$	$1x10^{-5}$
WIND	Annual Hazard Exceedance Probability	$1x10^{-3}$	$1x10^{-4}$
	Three Second Wind Speed, mph	133	160
	Missile Criteria	2x4 timber plank 15 lb. @ 50 mph (horiz); max height 30 ft.	2x4 timber plank 15 lb. @ 50 mph (horiz); max height 50 ft.
TORNADO	Annual Hazard Exceedance Probability	$2x10^{-5}$	$2x10^{-6}$
	Three Second Wind Speed, mph	180	220
	Atmospheric Pressure Change (APC)	70 psf @ 31 psf/sec	120 psf @ 45 psf/sec
	Missile Criteria	2x4 timber plank 15 lb. @ 100 mph (horiz); max height 150 ft; 70 mph (vert) 3 in dia std steel pipe, 75 lb @ 50 mph (horiz); max height 75 ft; 35 mph (vert) 3000 lb automobile @ 19 mph rolls and tumbles	2x4 timber plank 15 lb. @ 150 mph (horiz); max height 200 ft; 100 mph (vert) 3 in dia std steel pipe, 75 lb @ 75 mph (horiz); max height 100 ft; 50 mph (vert) 3000 lb automobile @ 25 mph rolls and tumbles
FLOOD	Annual Hazard Exceedance Probability	$1x10^{-4}$	$1x10^{-5}$
	Roof Design	See Section 5.2.1.5 and 5.2.1.6	
	Design Basis Flood	See Section 5.2.1.14	

TABLE 7.1.2 – Seismic Natural Phenomena Hazards Criteria

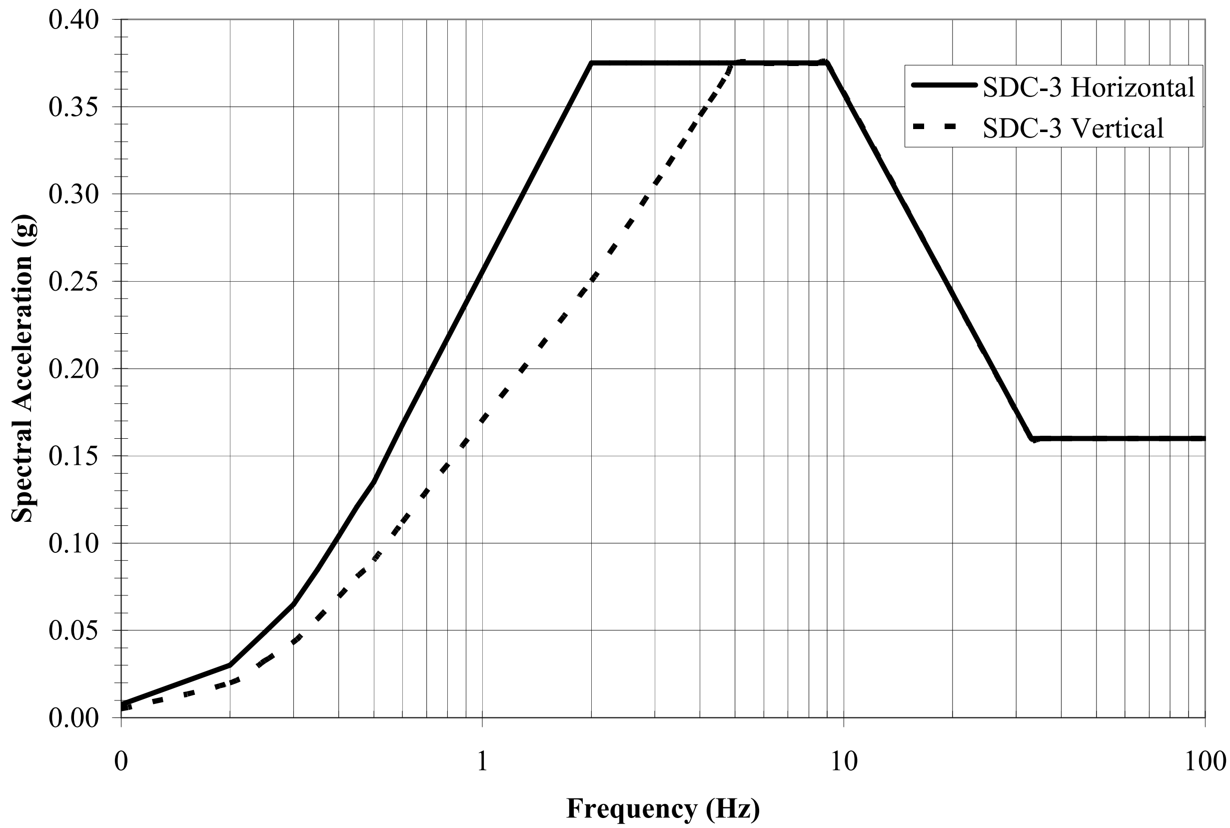
			Limit State				Analysis		
			A	**B**	**C**	**D**			
SDC - Seismic Design Category	Annual Hazard Exceedance Probability	Hazard Return Period	Large Permanent Distortion Allowed - Collapse Prevention	Moderate Permanent Distortion Allowed - Consideration for structural integrity and leak-tightness may be required	Minor Permanent Distortion Allowed - No post earthquake repair necessary	Essential Elastic Behavior - No damage reducing capacity	Soil-Structure Interaction	In-Structure Response Spectra	Post Seismic Differential Settlement
			Significant damage	*Generally repairable damage*	*Minimal damage*	*No damage*			
1	~6.E-04	1,667	<u>ASCE 7</u> Occupancy Category I or II I = 1 $R_a = R$	<u>ASCE 7</u> Occupancy Category I or II I = 1 $R_a = 0.80$ R But $R_a \geq 1.0$	<u>ASCE 7</u> Occupancy Category I or II I = 1 $R_a = 0.67$ R But $R_a \geq 1.0$	<u>ASCE 7</u> Occupancy Category I or II I = 1 $R_a = 1.0$	NR	NR	Only if confinement is required following a seismic event
2	4.E-04	2,500	<u>ASCE 7</u> Occupancy Category III or IV I = 1.25 (Cat III), I = 1.5 (Cat IV) $R_a = R$	<u>ASCE 7</u> Occupancy Category III or IV I = 1.25 (Cat III), I = 1.5 (Cat IV) $R_a = 0.80$ R But $R_a \geq 1.0$	<u>ASCE 7</u> Occupancy Category III or IV I = 1.25 (Cat III), I = 1.5 (Cat IV) $R_a = 0.67$ R But $R_a \geq 1.0$	<u>ASCE 7</u> Occupancy Category III or IV I = 1.25 (Cat III), I = 1.5 (Cat IV) $R_a = 1.0$	NR	NR	
3	4.E-04	2,500	<u>ASCE 43</u> F_μ from Table 5-1 for Building Elements (F_μ=1.0 to 6.0) F_μ from Table 8-1 for Equipment and Distribution Systems (F_μ=1.25 to 2.0)	<u>ASCE 43</u> F_μ from Table 5-1 for Building Elements (F_μ=1.0 to 4.0) F_μ from Table 8-1 for Equipment and Distribution Systems (F_μ=1.15 to 1.5)	<u>ASCE 43</u> F_μ from Table 5-1 for Building Elements (F_μ=1.0 to 2.5) F_μ from Table 8-1 for Equipment and Distribution Systems (F_μ=1.0 to 1.25)	<u>ASCE 43</u> F_μ from Table 5-1 for Building Elements F_μ=1.0 F_μ from Table 8-1 for Equipment and Distribution Systems F_μ=1.0	Yes	Yes	Yes
4	4.E-04	2,500	<u>ASCE 43</u> F_μ from Table 5-1 for Building Elements (F_μ=1.0 to 6.0) F_μ from Table 8-1 for Equipment and Distribution Systems (F_μ=1.25 to 2.0)	<u>ASCE 43</u> F_μ from Table 5-1 for Building Elements (F_μ=1.0 to 4.0) F_μ from Table 8-1 for Equipment and Distribution Systems (F_μ=1.15 to 1.5)	<u>ASCE 43</u> F_μ from Table 5-1 for Building Elements (F_μ=1.0 to 2.5) F_μ from Table 8-1 for Equipment and Distribution Systems (F_μ=1.0 to 1.25)	<u>ASCE 43</u> F_μ from Table 5-1 for Building Elements F_μ=1.0 F_μ from Table 8-1 for Equipment and Distribution Systems F_μ=1.0	Yes	Yes	Yes
5	1.E-04	10,000	<u>ASCE 43</u> F_μ from Table 5-1 for Building Elements (F_μ=1.0 to 6.0) F_μ from Table 8-1 for Equipment and Distribution Systems (F_μ=1.25 to 2.0)	<u>ASCE 43</u> F_μ from Table 5-1 for Building Elements (F_μ=1.0 to 4.0) F_μ from Table 8-1 for Equipment and Distribution Systems (F_μ=1.15 to 1.5)	<u>ASCE 43</u> F_μ from Table 5-1 for Building Elements (F_μ=1.0 to 2.5) F_μ from Table 8-1 for Equipment and Distribution Systems (F_μ=1.0 to 1.25)	<u>ASCE 43</u> F_μ from Table 5-1 for Building Elements F_μ=1.0 F_μ from Table 8-1 for Equipment and Distribution Systems F_μ=1.0	Yes	Yes	Yes

Note: For Systems, Equipment, and Component analysis and design see Engineering Standard 01061.

Table 7.1.3 SDC-3 Site Specific Spectra
(PC-3 is equivalent to SDC-3)

	Spectral Acceleration	
Frequency (Hz)	**Horizontal (g)**	**Vertical (g)**
0.10	0.0075	0.00500
0.20	0.030	0.02000
0.30	0.065	0.04333
0.40	0.104	0.06933
0.50	0.135	0.09000
0.60	0.1677	0.11180
2.00	0.375	0.25000
2.50	0.375	0.28044
3.00	0.375	0.30531
3.50	0.375	0.32634
4.00	0.375	0.34456
4.50	0.375	0.36063
5.00	0.375	0.37500
9.00	0.375	0.37500
33.00	0.160	0.16000
100.00	0.160	0.16000

Figure 7.2.1 Site Specific Spectra, SDC-3 Horizontal and Vertical Spectra, 5% Damping,
(PC-3 is equivalent to SDC-3)



7.3 APPENDIX A NOTES

Notes on selected sections in this Standard are given in this Appendix. They provide background and commentary material, supplementary guidance and additional information on requirements in this Standard. Appendix A references begin with the letter "A". References from the text of this Standard are also used in the Appendix.

A1.0 PURPOSE AND SCOPE

A1.2 Specific codes years are sometimes noted in the standard in order to bring attention to a particular section of a referenced code. The code year referenced corresponds to the most recent code edition at the time of issue of the present revision to the standard. This does not preclude the use of this standard with future code editions. To use a future code edition, the specific sections referenced should be replaced with the equivalent section of the future code edition.

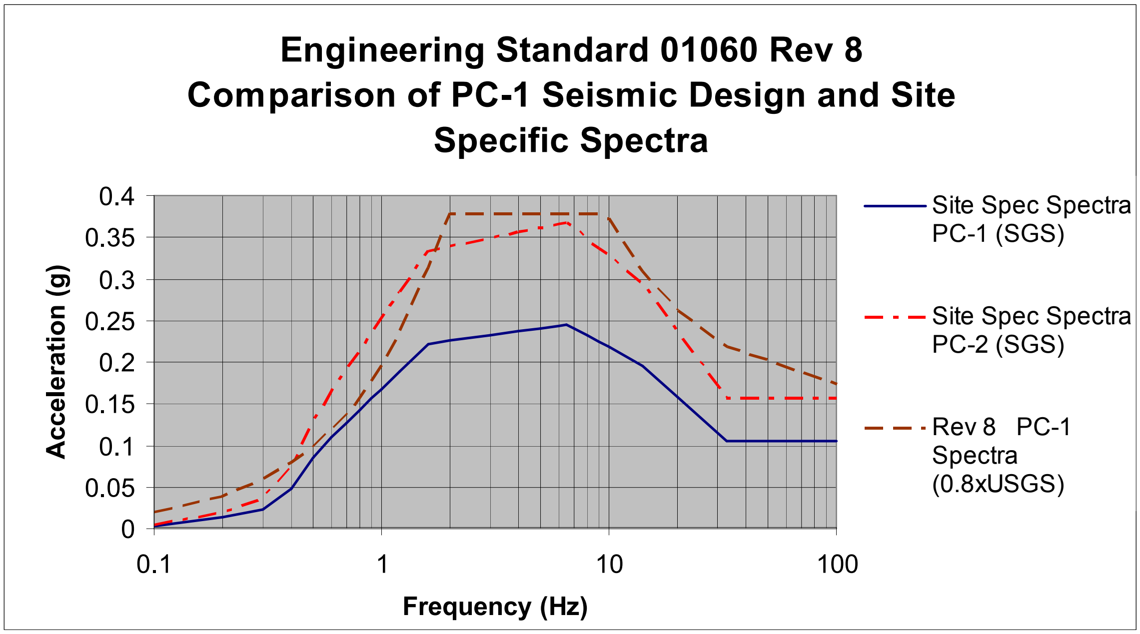
A1.3 The following describes the basis and development of the seismic criteria specified in this Standard. It also provides justification for the PC/SDC equivalencies specified in Section 1.3 of the Standard, most notably equating PC-2 to SDC-2 LS A rather than LS B as ASCE 43 does. For design purposes, the user is referred to Section A5.1.2.4 for SDC-1 and SDC-2 spectra equations and Table 7.1.3 for the SDC 3 spectrum.

A1.3.1 DOE-STD-1020-94 specified that PC-2 shall have two bump-ups over PC-1, 1) the seismic hazard exceedance probability is lower for PC-2, and 2) PC-2 requires an Importance Factor greater than 1.0 (I = 1.25).

A1.3.2 The generally accepted understanding of the Importance Factor is that it reduces the R factor in the base shear calculation, R being a measure of the allowed energy absorption from inelastic deformation. By reducing the R value less damage is expected, thereby moving the response to a less damaging Limit State.

A1.3.3 Items 1 and 2 above are, therefore, consistent with the table in the commentary of ASCE 43, showing the relationship of PC-1 and PC-2 relative to a greater seismic demand and less damage response.

A1.3.4 In the development of Engineering Standard 01060, it was recognized that the design spectra based on the USGS mapped acceleration values and the model building code equations were considerably greater than site-specific spectra developed by SGS, especially for the frequency range of interest. Provided below is a graphical comparison of site-specific spectra for PC-1 and PC-2 (developed in 1998, see Ref A7) along with the PC-1 design spectra specified in Revision 8 of Engineering Standard 01060. (Note that STD-01060, revision 8 was based on DOE-STD-1020-02, not 1020-94.) The Rev 8 PC-1 spectrum includes a reduction factor of 80% from the spectra generated purely on the USGS mapped values, i.e., the maximum reduction allowed when site-specific spectra are available in accordance with the IBC.



Note that the PC-1 design spectrum envelops the PC-2 site-specific spectrum, except for the frequency range between 0.4 Hz and approximately 1.7 Hz. The range of frequencies where the PC-1 design spectrum does not envelop the PC-2 site-specific spectrum is below the fundamental frequencies of both the building structure and the mechanical/electrical systems typically encountered at a nuclear facility. The exceptions to this include the building SSI frequencies, which are not a consideration for PC-2 structures, and tank/vessel sloshing effects. If applicable, liquid sloshing typically occurs below 0.4 Hz, a region where the design spectra envelop the site-specific spectra. For most design applications involving PC-2 SSCs, an equivalent static analysis method that bases the seismic design loads on peak acceleration data (with a suitable dynamic load factor) is an acceptable simplified approach that negates the need to determine the system natural frequencies. Hence, using the design spectra is conservative relative to the PC-2 site-specific spectra

A1.3.5 Based on the above, the early revisions of Engineering Standard 01060 essentially raised the PC-1 input motion to a return period roughly equivalent to PC-2, but the Importance Factor was maintained at 1.0. The PC-2 input motion was then made equivalent to PC-1, but the seismic Importance Factor was specified so as to be consistent with the model building code Essential Facility (initially 1.25 and later 1.5).

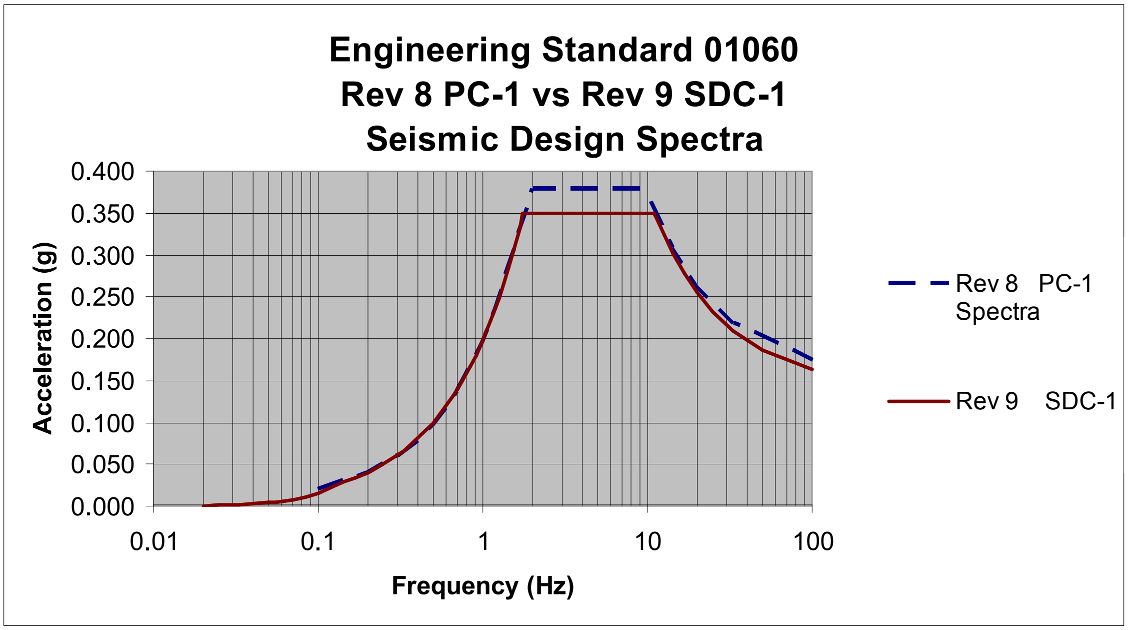
A1.3.6 DOE-STD-1020-02 subsequently changed the definition of PC-2 relative to PC-1. It specified that PC-2 will have a single bump-up over PC-1, i.e., an Importance Factor of 1.5. DOE-STD-1020-02 is the basis for Engineering Standard 01060, revision 8.

A1.3.7 In Engineering Standard 01060, revision 9, ASCE 43-05 methodology for seismic load specification was adopted along with additional data from DOE-STD-1189 resulting in a graded approach to the application of input motion (SDC) and damage control (Limit State) for the lower seismic demand SSC (SDC-1 and SDC-2).

A1.3.8 The table in Appendix A of DOE-STD-1189 unfortunately identified the Importance Factor, I, as the means for increasing the seismic input level from SDC-1 to SDC-2. To achieve the desired Limit State, it also specified a different factor as being applied to the Response Modification Coefficient (R value). For Limit State B (the goal for PC-2 SSCs as identified by the commentary table C1-1 in ASCE 43), the factor is 0.8 (or 1/1.25) which is consistent with the Importance Factor identified in DOE-STD-1020-94 but not DOE-STD-1020-02.

A1.3.9 The commentary table in ASCE 43-05 is consistent with DOE-STD-1020-94, but not DOE-STD-1020-02. Additionally, the table in Appendix A of DOE-STD-1189 further complicates the issue by mixing the use of Importance Factor with another factor applied to the R value.

A1.3.10 An additional change from STD-01060, revision 8 to revision 9 was the use of later USGS seismic acceleration map values for SRS. A comparison of the resulting basic spectra (Revision 8 PC-1 vs. Revision 9 SDC-1) is provided below.



Note that there is a reduction in the range of peak acceleration of approximately 8%, but the amplitudes at other frequencies are nearly identical.

A1.3.11 The design spectra for SDC-3 in Engineering Standard 01060, revision 9, have been held at the PC-3 spectra, awaiting further clarification from the Central and Eastern United States (CEUS) seismic hazard study that is currently being conducted by EPRI.

A1.3.12 There is no one-for-one correlation in F_μ values between ASCE 43 and DOE-STD-1020. DOE-STD-1020 provides a single value of F_μ for 26 types of structural components. ASCE 43 lists 41 types of structural components, with an F_μ identified for each Limit State A through D. A cursory review indicates that for components common to both lists, DOE-STD-1020 identifies F_μ values that fall between Limit States B and C. The conservative approach would be to classify DOE-STD-1020 PC-3 equivalent to Limit State C, consistent with the table in Appendix A of ASCE 43.

A1.3.13 Based on the historical evolution of seismic design criteria described above, a comparison between the seismic structural criteria of DOE-STD-1189 (as implemented at SRS beginning with STD-01060 Rev.9) with the criteria of DOE-STD-1020 (as implemented at SRS up through STD-01060 Rev.8) implies the following seismic criteria equivalency:

PC-1 = SDC-1, LS-A
PC-2 = SDC-2, LS-A
PC-3 = SDC-3, LS-C, New construction: Apply LS-C.

Re-qualification of existing facilities: Apply LS-C, provided the existing structure is capable of absorbing inelastic energy in accordance with ASCE-43 or DOE 1020 requirements. Alternatively, apply LS-D.

A5.0 REQUIREMENTS

Structural requirements are identified in the project criteria documents. This document is to include the NPH requirements identified as the Performance Category, Seismic Design Category, and the acceptable Limit State. The NPH categorization and acceptable Limit State are developed as part of the hazard analysis and documented in accordance with SCD-11.

A5.1 Structural Requirements for Commercial Standard SSCs

SSCs with a NPH designation of PC-0, PC-1, PC-2,SDC-1 or SDC-2 are designed in accordance with the IBC code, as amended by the requirements of Sections 5.1 and 5.3.

IBC refers to specific ASCE standards to develop demands and specific material codes (ACI, AISC, etc.) to develop capacities. It is not acceptable to substitute alternate standards or material codes because the alternate code/standard may not provide the same level of safety. For example the IBC does not mention, hence it does not allow, ACI 349.

A5.1.1 Classification of Structures for IBC Importance Factors

The wind load importance factors are defined in DOE-STD-1020 as 1.0 for both PC-1 and PC-2. DOE-STD-1020, Table 3-2 also recommends wind speeds of 100 and 107 mph for PC-1 and PC-2 structures which equate to a wind importance factor of 1.0 for PC-1 and 1.145 for PC-2 assuming a basic wind speed of 100 mph. To be consistent with the IBC wind design methodology, an importance factor of 1.15 is adopted for PC-2 with a basic wind speed of 100 mph. This agrees with ASCE 7 Table 6-1 with PC-1 representing Category II and PC-2 representing Category IV. Consistent with ASCE 7, Table 6-1, a wind importance factor of 1.15 is also used for Occupancy Category III.

DOE-STD-1020 does not explicitly define the snow importance factor. The ratio of rainfall accumulation for PC-2 and PC-1 structures ranges from 1.11 to 1.24 for various rainfall durations in SRS STD 01110. Thus, the PC-2 rainfall is approximately 1.2 times the PC-1 rainfall. Assuming that snow is proportional to rainfall intensity yields a PC-2 snow importance factor of 1.2, which is consistent with the snow importance factor of 1.2 for Category IV buildings in ASCE 7 Table 7-4. Consistent with ASCE 7, Table 7-4, a snow importance factor of 1.1 is used for Occupancy Category III. The resulting categories, use groups and importance factors are summarized in the table below.

Consistent with DOE-STD-1189, seismic importance factors are per ASCE 7. It is noted that per DOE-STD-1189, ASCE 7-05 Occupancy Category IV (I_E = 1.5) shall be used if there is a radiological release consequence of concern to the public or the environment resulting from an unmitigated failure of the SSC.

DOE Category	IBC Occupancy Category	Importance Factors				
		Snow	Wind	Ice Load		Seismic
				Wind Pressure	Ice Accretion	
		I_S	I_W	I_W	I_i	I_E
PC-1, SDC-1	II	1.0	1.0	1.0	1.0	1.0
PC-2, SDC-2	III	1.1	1.15	1.0	1.25	1.25
PC-2, SDC-2	IV	1.2	1.15	1.0	1.25	1.5

A5.1.2.2 Rain Load (R) and Ponding Load (P)

The rain load and ponding criteria are derived from DOE-STD-1020. Site specific rainfall accumulations are provided in SRS STD 01110.

A5.1.2.3 Wind Load (W)

See the discussion in A5.1.1.

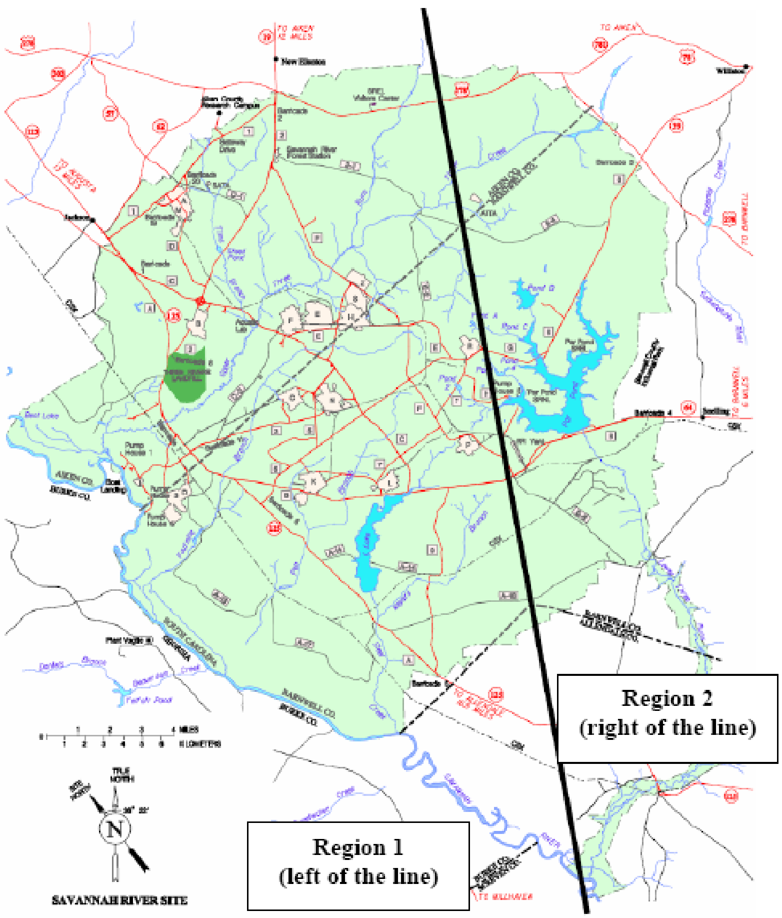
For the determination of atmospheric ice loads, a conservative maximum value of nominal ice thickness, t, for the entire Savannah River Site, obtained from ASCE 7 maps, is 0.75 in. It is permissible to use a facility specific nominal ice thickness per ASCE 7 maps.

A5.1.2.4 Earthquake Load (E, Em)

IBC earthquake load parameters are developed in T-CLC-G-00312.

Seismic Design Spectra

Due to the large area covered by SRS and the proximity of SRS to Charleston, SRS is divided into two regions as shown below. All of the SRS production areas (A, B, C, D, E, F, H, K, L, M, N, P, R, S, T, Z) are in Region 1 (left of the line). Also included in Region 1 are Barricade 1(Jackson), Barricade 2 (New Ellenton), L-Lake Dam, and the SREL Visitor's Center. The facilities in Region 2 (right of the line) include ATTA, Par Pond Dam, the RR Yard, Barricade 3 (Williston) and Barricade 4 (Barnwell).



The IBC design response spectra control points are given in the table below for SDC-1 with an importance factor of 1.0. Equations for the design response spectra for the two regions at SRS are given below. See T-CLC-G-00312 for digitized values.

IBC Design Response Spectra Control Points, 5% damping (SDC-1)

	SRS Region 1 (left of the line)	SRS Region 2 (right of the line)
S_{DS}	0.35g	0.366g
S_{D1}	0.199g	0.206g
T_0	0.091 sec	0.090 sec
T_S	0.570 sec	0.561 sec

For SRS Region 1 (left of the line), the SDC-1 Design Response Spectra (5% damped) is given by the following equations:

$S_a = 0.6 \text{ x } (S_{DS}/T_0)\ T + 0.4 \text{ x } S_{DS} = 2.31g \text{ x } T + 0.14g$ .... For $T < T_0$

$S_a = S_{DS} = 0.35g$ .... For $T_0 \leq T \leq T_S$

$S_a = S_{D1} / T = 0.199g / T$ .... For $T_S < T \leq T_L$

$S_a = (S_{D1})(T_L) / T = (0.199g)(8) / T^2 = 1.592g / T^2$ .... For $T > T_L$

For SRS Region 2 (right of the line), the SDC-1 Design Response Spectra (5% damped) is given by the following equations:

$S_a = 0.6 \text{ x } (S_{DS}/T_0)\ T + 0.4 \text{ x } S_{DS} = 2.44g \text{ x } T + 0.146g$ .... For $T < T_0$

$S_a = S_{DS} = 0.366g$ .... For $T_0 \leq T \leq T_S$

$S_a = S_{D1} / T = 0.206g / T$ .... For $T_S < T \leq T_L$

$S_a = (S_{D1})(T_L) / T = (0.206g)(8) / T^2 = 1.648g / T^2$ .... For $T > T_L$

Note that for SDC-2 an importance factor of 1.5 is used.

Seismic Base Shear

The seismic base shear is proportional to the spectral acceleration at the structure's natural period, the importance factor, I_E, and weight. The seismic base shear is inversely proportional to the inelastic response modification factor, R. Note that the R values applicable to SDC-1 and SDC-2 structures are significantly larger than the F_μ values used in SDC-3 and higher analysis.

Both R and F_μ reduce the structures loading to account for inelastic behavior. R values are larger than $F\mu$ because the R value represents a life safety limit state while the $F\mu$ represents a confinement limit state. The life safety limit state would require that concrete walls remain standing but may be extensively cracked; i.e. they may not maintain pressure differential with normal HVAC. Contrarily, the confinement limit state would allow concrete walls to be cracked; but the cracks would be small enough to maintain pressure differential with normal HVAC. A detailed discussion of seismic performance is contained in ASCE 43 and ASCE 4.

Seismic Design Category
IBC Seismic Design Categories, which are different than the SDC notation used in this document and ASCE 43, are defined in IBC 2009 Tables 1613.5.6(1) and (2) as a function of the Occupancy Category, S_{DS} and S_{D1}. For SDC-1 structures in Region 1 (left of the line) with Occupancy Category I or II, the IBC Seismic Design Category is C. For SDC-2 structures in Region 1 (left of the line) with Occupancy Category IV, the a IBC Seismic Design Category is D. Due to the value of S_{D1}, for SDC-1 and SDC-2 structures in Region 2, the IBC Seismic Design Category is D per IBC 2009 Table 1613.5.6(2). On a facility specific basis, SDC-1 structures in Region 2, may classified as IBC Seismic Design Category C using IBC 2009 Table 1613.5.6(1) alone if the provisions of IBC 2009 §1613.5.6.1 are satisfied.

Alternate Earthquake Load Parameters
As an alternate to the design response spectra derived by use of the S_{DS} and S_{D1} given in A5.1.2.4 location specific design response parameters may be developed for a given facility. Spectral acceleration, S_a, for any period for the location specific design response spectra shall meet other requirements of ASCE 7, Chapters 11 and 21.

SCDHEC Regulated Structures
Structures regulated by SCDHEC (Ref. 6.2.15.1) are required, under seismic provisions for Hazardous Waste Management locations, to maintain confinement for earthquake ground motions with a ten percent probability of occurrence in two hundred and fifty years. The mean return period for such ground motion is also about 2,500 years.

A5.1.2.5 Self Straining Force (T)
IBC requires that other loads, such as the self straining force (T), be considered when applicable. The self straining force arises from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement or combinations thereof.

Dynamic settlement due to dissipation of pore pressure and redistribution of soil stresses from the effects of a design basis earthquake may be specified, depending on the local site conditions. The magnitude of foundation settlements shall be developed from geotechnical investigations. Earthquake induced foundation settlements need not be combined with seismic inertial loads.

Analytical experience has shown that cast-in-place reinforced concrete and structural steel buildings are generally capable of accommodating several inches of differential settlement without collapse. Differential settlement for such buildings need not be explicitly considered.

Contrarily, precast concrete and lightly reinforced masonry buildings could experience severe local, through-the-wall, cracking which may lead to a loss of confinement when subject to several inches of differential settlement. If a precast concrete structure is used for an airborne confinement boundary, differential settlement could potentially overstress the joint connections between adjacent precast wall panels which may lead to loss of confinement or even collapse. Differential settlement of this structural system must be explicitly considered.

A5.1.3 Tornado Shelter Requirements
The requirement for Tornado Shelters in PC-1 and PC-2 structures arises out of a Memorandum from DOE-SR to WSRC (Ref. A10).

The Tornado Shelter requirement for a facility may be omitted if a tornado shelter, which is suitable for the normal occupancy of the facility, exists in the vicinity of the facility. A walking distance, typically covered in the minimum time between a tornado warning and a potential tornado, shall be considered to be the vicinity. The tornado shelter in the vicinity could be existing or new.

Trailers are not considered permanent structures for the purposes of the Tornado Shelter requirement.

The 161mph design tornado wind speed is based on site specific studies and is less than the 200 mph regional design wind speed in FEMA 361. The 15 lb. 2x4 tornado missile is specified by FEMA 361.

A5.1.4 Seismic Structural Interaction

In order to assure no seismic Structural Interaction (SI), the lower SDC (or FC) SSC is evaluated for the higher SDC (or FC) demand loads for no collapse criteria. The higher SDC (or FC) is not assigned to the lower level SDC (or FC) SSC. The interaction load determination may be performed by a bounding analysis but need not follow the rigor of the higher SDC SSC, and the no collapse analysis may go beyond code allowable capacities.

A5.1.5 Other Structures

The codes and standards identified in this section have loads and load combinations that generally fall into two broad groups (1) ASCE 7 based loads; and (2) specialized structure/component specific loads. Note that IBC and ASCE 7 loads and loading combinations are compatible. Thus, codes with ASCE 7 based loads are generally consistent with IBC loads.

Other codes, such as ACI 307 and ASME STS-1, contain a state of the industry approach to determine wind forces for stacks and chimneys. The approach used in these two codes starts with the IBC basic wind speed and the importance factor defined in Section 5.1.1. The wind forces developed by these two codes are more representative than wind loads developed using IBC.

We anticipate that future versions of all the codes and standards in Section 5.1.5 will eventually develop seismic criteria which are compatible with IBC. The Design Authority may develop project specific specifications, which exceed the requirements of this standard, to ensure compatibility with anticipated code updates.

Useful additional information on anchor bolt design for steel stacks may be derived from two ASCE references [A13 and A14].

A5.2 Requirements for Hazardous Facility Structures

Operating and NPH loads are defined in this section. NPH requirements for PC-3, PC-4, SDC-3, SDC-4, and SDC-5 structures are based on DOE-STD-1020 and ASCE 43. Dynamic analyses conforming to ASCE 4 are required for seismic loads.

Structural capacities may be developed using either ACI 318, ACI 349, AISC 360, or ANSI/AISC N690. The load combinations referenced by the specific capacity code are utilized to ensure consistency between load combinations, factors of safety, load factors and strength reduction factors. Seismic load combinations are modified as required to be consistent with DOE-STD-1020. Tornado wind load combinations are added as required.

A5.2.1.3 Soil Load (H)

The effects of soil compaction can cause localized soil pressures to exceed the at-rest soil pressure. Soil compaction loads are considered surcharge loads.

A5.2.1.6 Snow Load (S)

DOE STD 1020 does not have specific criteria pertaining to snow load and the flood criteria is adapted for use. The flood criteria is based on a 10,000 year return period for PC-3 structures and a 100,000 year return period for PC-4 structures. ASCE 7 provides ground snow load with a mean return period of 50 years. SRS Standard 01110 contains site specific rainfall hazard information for 500 to 100,000 year return periods. The rainfall data in SRS Standard 01110 is extrapolated to 50 years and shown in Figure A5.2.1.6-1.

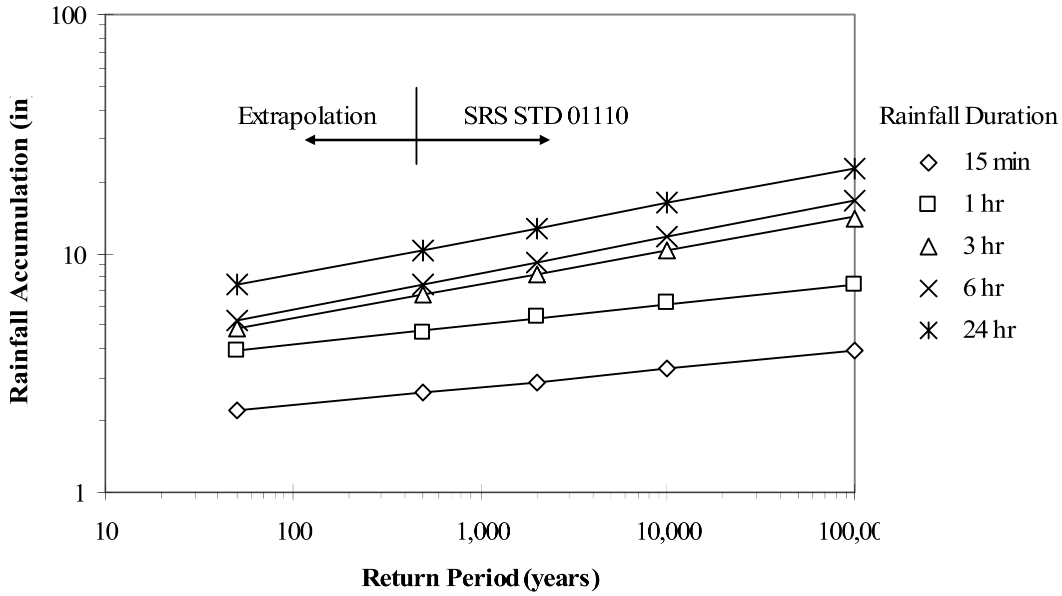


Figure A5.2.1.6-1 SRS STD 01110 Rainfall Data

Considering each rainfall duration separately, the maximum ratio of the 10,000 year rainfall to the 50 year rainfall is 2.26. Likewise the maximum ratio of the 100,000 year rainfall to the 50 year rainfall is 3.20.

The slope of the rainfall hazard curve is assumed to be similar to the slope of the snow hazard curve. Thus, the 2.26 multiplier between a 50 year event and 10,000 year event is assumed to be applicable to both PC-3 rain and snow.

The ASCE 7 ground snow load for SRS is 5 psf as specified in ASCE 7 Figure 7.1. However, SRS is near 10 psf snow contour on Figure 7.1. ASCE 7 Table C7-1 contains city specific ground snow loads that list Augusta GA with a 7 psf snow load and Columbia SC with an 8 psf snow load. Since SRS is much closer to Augusta, the 50 year SRS ground snow load is increased to 7 psf. Note that IBC specified 5 psf ground snow load is sufficiently accurate for PC-1 and PC-2 analyses.

The 7 psf ground snow load is factored in the following table to achieve snow loads with 10,000 and 100,000 year return periods. Load combinations treat snow as a service level event and apply a load factor of 1.6. Dividing the 10,000 and 100,000 year ground snow loads by the load factor yields 9.9 psf and 14 psf snow loads for PC-3 and PC-4 respectively. The snow loads are rounded to the nearest 5 psf.

	PC-3	PC-4
Return Period	10,000	100,000
Hazard Multiplier	2.26	3.20
Ground Snow Load based on a load factor of 1.0	7 x 2.26 = 15.8 psf	7 x 3.20 = 22.4 psf
Ground Snow Load based on a load factor of 1.6	15.8/1.6= 9.9 psf	22.4/1.6=14.0 psf
Design Ground Snow Load, $I \times p_g$	10 psf	15 psf

A5.2.1.7 Wind Load (W)

Extreme winds in the Savannah River Site (SRS) area, excluding tornado winds, are associated with tropical weather systems, thunderstorms, or strong winter storms.

The PC-3 and PC-4 design basis wind speeds given in Section 5.2.1.7 correspond to the 10^{-3} and 10^{-4} annual probability of exceedance wind speed in Ref. A4. The wind speeds in Ref. A4 are given in fastest-mile wind speeds and are converted to three-second gust wind speeds using Figure C6-2 of ASCE 7-02 in Ref A9.

The design basis wind speeds given in Section 5.2.1.7 are larger than the values for estimated maximum three-second gust straight wind speeds based on a site specific investigation reported in Reference 6.2.14.8. The straight-line (non-tornadic) wind speeds (three second gusts) for SRS for return periods of 50, 100, 1,000 and 10,000 years are found to be about 83, 88, 107 and 127 miles per hour. The estimates for straight wind are generated from a Fisher-Tippet Type I extreme value (Gumbel) distribution function using historical wind speed (gust) data from SRS and the surrounding region. The period of record ranged from 25 to 47 years. The methodology is consistent with the Department of Energy criteria for site hazard characterization (Ref. A1).

Reference 6.1.5 contains Interim Advisory on Straight Winds and Tornados which identified the PC-3 and PC-4 wind speeds as 1.3 and 1.5 times the 100 mph PC-1 wind speed.

All three sets of recommendations are summarized in the following table. Maximum values are taken conservatively as the design basis wind speeds.

	PC-3	PC-4
Annual Probability of Exceedance	10^{-3}	10^{-4}
Reference A4, modified by Reference A9	133 mph	160 mph
Reference 6.2.14.8	107 mph	127 mph
Reference 6.1.5	130 mph	150 mph
Design Basis Wind Speed	133 mph	160 mph

The ice maps developed for ASCE 7 are based on a mean recurrence interval of 50 years. The 2 factor in ASCE 7 Equation 10-5 is used to convert the mapped nominal ice thickness value to a 500 year return period, the same return period as used for wind loads.

Assuming the required annual probability of hazard exceedance (APE) for ice is the same as that for wind, the APE for PC-3 and PC-4 ice loads is 1E-3 and 1E-4, respectively. In accordance with ASCE 7 Table C10-1, for an APE of 1E-3, the multiplier is 2.3 on the mapped nominal thickness of 0.75 inches, giving a thickness of 2.3 x 0.75 = 1.725. This value is rounded up to 1.875 inches so as not to be lower than the value obtained for PC-2 (2 x 0.75 x 1.25 = 1.875). While ASCE 7 Table C10-1 does not give a factor for an APE of 1E-4, the data can be extrapolated to obtain a factor of 3.33. This is used to obtain a value of 2.5 inches (3.33 x 0.75) for PC-4 criteria.

This criterion is based on the maximum mapped value of nominal ice thickness, t, for SRS of 0.75 inches. It is permissible to use a facility specific nominal ice thickness per ASCE 7 maps in conjunction with the above methodology.

A5.2.1.8 Tornado Load (Wt)

Tornado Speed

Ref 6.2.17.1 develops the tornado wind speed as 169 mph and 214 mph at annual frequencies of $2x10^{-5}$ and $2x10^{-6}$ respectively for the Savannah River Site. Ref 6.2.17.1 is based on the fastest ¼ mile wind speed, which is converted to three-second gust wind speeds using Figure C6-2 of ASCE 7-02. The resulting three second gust wind speeds of 173.4 mph and 217 mph are rounded up to 180 mph and 220 mph, respectively, for PC-3 and PC-4 tornado events.

Missiles
The design basis straight-line wind and tornado missiles given in Table 7.1.1, except for the PC-3 tornadic automobile missile, are based on the DOE-STD-1020 (Ref. 6.1.1) requirements for PC-3 and PC-4 DOE sites. McDonald (Ref. A2) provides a rationale for the PC-3 and PC-4 design basis missiles criteria for DOE sites in relation to the observed data for tornadoes, and results of simulation and analytical studies. Sections 2.7 and 3.8.4 of the McDonald report (Ref. A2) summarize the basis for the DOE missiles and for the corresponding speeds, respectively.

In the comprehensive report (Ref. A2) , McDonald observes that the design basis missile criteria for DOE sites are different from the United States Nuclear Regulatory Commission (US NRC) criteria for nuclear power plants. Section 1.4 of the report (Ref. A2) addresses specifically the US NRC Standard Review Plan (SRP, Ref. 6.2.16.1) 3.5.1.4 criteria. The DOE criteria are more appropriate, the report states, because the missiles are selected based on field observations, level of risk is different from nuclear power plants, and the design basis tornado speeds are significantly lower than those in US NRC criteria.

The PC-3 tornadic missile of an automobile weighing 3,000 pounds, not given in DOE-STD-1020 (Ref. 6.1.1), represents large heavy missiles that roll and tumble along the ground (Ref. A2). Its design basis speed is based on the recommendation of McDonald-Mehta on a specific proposed project at Savannah River Site (Ref. 6.2.14.10). The US NRC (1975) criteria contain missile criteria for an automobile weighing 4,000 pounds (Ref. A2).

The global effects of tornado wind pressure load and the local effects of missile impact loads may not occur at the same time, in which case their effects need not be combined for design and evaluation purposes. For example, if the missile is ejected after the tornado attains its maximum speed near its maximum radius, then the tornado wind pressures and the atmospheric pressure change effects are negligible at the time of the missile strike. However there may be other conditions when their effects may have to be combined where necessary.

Atmospheric Pressure Change (APC)
Calculations for the APC parameters are based on the methodology given in References A3 and A4. Values of the parameters for the PC-3 and PC-4 design basis tornado speeds of 180 and 220 miles per hour (mph) are given below (Ref. A5).

	PC-3	**PC-4**
Vmax = Design Basis Tornado, mph	180	220
Vt = Maximum Translational Speed, mph	50	50
Vro = Maximum Rotational Speed, mph = Vmax - Vt	130	170
Vθ = Maximum Tangential Speed, mph = 0.89 x Vro	115.7	152.3
APC = Maximum Atmospheric Pressure Change, psf = $0.00238 \times (V\theta \times 5{,}280/3{,}600)^2$ rounded up to	68.5 70	117.1 120
Rmax = Maximum Radius, ft	165	195
Rate of Pressure Drop, psf/sec = APC x (Vt / Rmax) x (5,280/3,600)	31	45

In accordance with DOE-STD-1020, at peak tornado wind speeds the APC may be reduced by ½ of the maximum value.

A5.2.1.9 Earthquake Load (E)
A major change in seismic design introduced by ANS 2.26 and ASCE 43 is dividing the seismic demand requirements into two parts, load and response. The load portion is reflected in the return period of the Seismic Design Criteria (SDC) and the associated change in ground acceleration values. The response portion is reflected in acceptable damage to the SSC defined by the specification of a Limit State (LS). The combination of SDC (3,4, or 5) and Limit State (A, B, C, or D) determines the Seismic Design Basis (SDB) and acceptance criteria for designing SSCs.

The Limit States are selected to ensure the desired safety performance in an earthquake, the limiting acceptable deformation, displacement, or stress that an SSC may experience during or following an earthquake and still perform its safety function. Four Limit States are identified and used by this standard and ANSI/ASCE/SEI 43

Limit State A: An SSC designed to this Limit State may sustain large permanent distortion short of collapse and instability (i.e., uncontrolled deformation under minimal incremental load) but shall still perform its safety function and not impact the safety performance of other SSCs. Examples of SSCs that may be designed to this Limit State are as follows:

(1) building structures that must function to permit occupants escape to safety following an earthquake;
(2) systems and components designed to be pressure retaining but may perform their safety function even after developing some significant leaks following an earthquake.

Limit State B: An SSC designed to this Limit State may sustain moderate permanent distortion but shall still perform its safety function. The acceptability of moderate distortion may include consideration of both structural integrity and leak-tightness. Examples of SSCs that may be designed to this Limit State are as follows:

(1) building structures that cannot be damaged to the extent that the ability to perform their safety function is lost. Such structures include fire stations, hospitals, or other emergency response structures;
(2) systems and components designed to be pressure retaining but may perform their safety function even after developing some minor leaks following an earthquake (i.e., either they do not contain hazardous material, or the leakage rates associated with minor leaks do not exceed the consequence level of the assigned SDC category).

Limit State C: An SSC designed to this Limit State may sustain minor permanent distortion but shall still perform its safety function. An SSC that is expected to undergo minimal damage during and following an earthquake such that no post earthquake repair is necessary may be assigned this Limit State. An SSC in this Limit State may perform its confinement function during and following an earthquake. Examples of SSCs that may be designed to this Limit State areas follows:

(1) glove boxes containing radioactive or hazardous material;
(2) confinement barriers for radioactive or hazardous materials;
(3) heating ventilation and air-conditioning systems that service equipment or building space containing radioactive or hazardous material;
(4) active components that may have to move or change state following the earthquake.

Limit State D: An SSC designed to this Limit State shall maintain its elastic behavior. An SSC in this Limit State shall perform its safety function during and following an earthquake. Gaseous, particulate, and liquid confinement by SSCs is maintained. The component sustains no damage that would reduce its capability to perform its safety function. Examples of SSCs that may be designed to this Limit State are as follows:

(1) containments for large inventories of radioactive or hazardous materials;
(2) components that are designed to prevent inadvertent nuclear criticality;
(3) SSCs that perform safety functions that may be impaired due to permanent deformation (e.g., valve operators, control rod drives, high-efficiency particulate air (HEPA) filter housings, turbine or pump shafts, etc.);
(4) SSCs that perform safety functions that require the SSC to remain elastic or rigid so that it retains its original strength and stiffness during and following a DBE to satisfy its safety, mission, or operational requirements (e.g., relays, switches, valve operators, control rod drives, HEPA filter housings, turbine or pump shafts, etc.).

For examples of application of limit states to SSCs see ANS 2.26 Appendix B.

With the implementation of DOE-STD-1189, the design basis ground motion is now determined based on the provisions of ASCE 43-05. Development of a new SRS ASCE 43 design spectra has been put on hold awaiting an input rock motion spectra from an Electric Power Research Institute (EPRI) study of seismic hazard for the Central/Eastern United States (CEUS). DOE has provided support to this effort and will subscribe to the issued results. The EPRI study is progressing to support the licensing of new nuclear power plants and is targeted for completion in 2009. This work will have a technical basis supported by the national geotechnical community, thereby removing the potential for disagreements and controversy as was associated with the issuance of the 1999 PSHA. Once the CEUS seismic hazard model is developed and input rock spectra issued, SRS design spectra will be developed for inclusion in this standard. The time required to develop the SRS design spectra could be as much as an additional year. Until the new spectra is developed, the spectra used for PC-3 structures will continue to be used for SDC-3 structures. The use of this spectrum is consistent with the requirements of ASCE 43 as shown in T-CLC-G-00312. It is believed that the PC-3 spectra is conservative relative to what will be produced for the SDC-3 spectra when then new rock spectra will be available from the CEUS study.

Per ASCE 43, the difference between the SDC-3 and SDC-4 spectra is the use of a 0.8 or 1.0 scale factor, respectively. Consistent with this methodology and the discussion above, until the new spectra is developed, the SDC-4 spectrum for SRS is determined by multiplying the SDC-3 spectra by a factor of 1.25 (i.e.1.0/0.8).

The SDC-5 design basis response spectra is omitted from the body of this revision of the Standard in anticipation of a determination of the seismic hazard curve for SRS which will include additional data. In the interim, the SDC-5 design basis horizontal response spectra is taken as the PC-4 spectra given in Revision 5 of this Standard and presented in Table A5.2.1.9-2 and Figure A5.2.1.9-2. It is provided for information purposes only and should be used only on a case-by-case basis and after concurrence by Geotechnical Engineering and DOE-SR. The SDC-5 design basis vertical response spectra is not determined at this time. There is no certainty that the V/H ratio for soil profiles at SRS for an SDC-5 earthquake would not exceed unity. Furthermore there appears to be no significant need at the present time for a site wide SDC-5 design basis ground input spectra.

A5.2.2 Structural Design

Material codes are identified in this section which contain load combinations required to calculate demand along with capacities used to demonstrate that the demand is less than the capacity. Supplemental load combinations are provided in Section 5.2.2 to address NPH loads not specifically addressed by the material codes. In general ASCE 43 is used as the guiding document. However, since ASCE 43 references a previous generation of design codes, exceptions are made to allow the use of the latest generation of material design codes

There may be some situations where a specific material code may not contain explicit provisions for a design load. It is not acceptable to omit loads that are not explicitly required by a specific design code.

Information useful to selecting material codes for design is contained in DOE G 420.1-1.

A5.2.2.1 Concrete Structures

For low-rise shear wall structures, ASCE 43 Section 4.2.3 provides an alternate equation for shear capacity because it was judged the ACI 349 equations were too conservative. The ACI 349 committee is currently studying this equation. Use of the ACI 349 capacity equation is more conservative and should be used for new design, however the ASCE 43 equation may be used where additional capacity is needed provided adequate justification is given.

The dead and live loads in load combinations include the effects of static differential settlement.

A5.2.2.2 Steel Structures

The capacity of steel structures is to be determined in accordance with the provisions of ASCE 43. However, ASCE 43 is based on the previous generation of steel codes including the AISC N690-94, AISC 335 (ASD 9), AISC 690L-03, and AISC LRFD (LRFD 3), These steel codes have all been superseded by the new combined codes referenced in this standard. In order to use the updated provisions of the new steel codes an exception must be made to ASCE 43 section 4.2.4, Structural Steel.

AISC 360, ASCE 8, or ANSI/AISC N690 may be used for the design of a PC-3, SDC-3 and SDC-4 structures while only ANSI/AISC N690 may be used for the design of a PC-4and SDC-5 structures.

AISC N690 consists of exceptions to the AISC 360 code as well as the applicability of ASCE 8 and is generally intended for elastic design Therefore, AISC N690 does not direct application of the seismic provisions of AISC 341, stating that the provisions are generally not applicable and that the AISC 341 sections 6 and 7 shall be appropriately considered when designing for inelastic behavior. Since, ASCE 43 allows for the use of N690 with inelastic behavior (i.e. $F_{\mu} > 1.0$), seismic detailing must be provided in these cases.

AISC N690 specifies the strength of stainless steel members, assemblies and connections be determined in accordance with sections 3, 4, and 5 of ASCE 8. While the ASCE 8 criteria is specifically developed for cold-formed members it is judged to be applicable for light (less than ½ " thick) extruded and hot rolled stainless steel sections. Note that N690 does not limit the application of ASCE 8 to only cold formed stainless steel structures.

A5.2.2.3 Foundation Design

The Geotechnical Engineer shall determine the ultimate bearing capacity, allowable bearing capacity, and strength reduction factor. These properties shall be determined based on the site-specific geotechnical investigation. Generally, at SRS, allowable bearing capacity is one third of the ultimate bearing capacity and the strength reduction factor is between 0.5 and 0.8. Allowable bearing capacity for NPH loading and certain temporary dynamic loading may be increased by 33%.

The Geotechnical Engineer shall also estimate foundation settlements, including static and dynamic settlements, total as well as differential settlements. Settlements shall be estimated based on expected loads and site specific soil conditions.

The sliding resistance may include up to 50% of the passive pressure acting on the embedded portion of the foundation greater than 2 feet deep. The full passive pressure acting on a foundation is an ultimate capacity and is predicated on deformation. Up to 100% of the passive pressure (less the portion for the upper 2 feet) may be utilized if the horizontal deformation required to mobilize the passive resistance is explicitly considered.

Horizontal deformation of the building embedment is required to develop the full passive pressure acting on the foundation (Ref A17). Deformation can range from 2 to 4% of the embedment for sands and from 10 to 15% of the embedment for clays. At this deformation the foundation has slid and it is not appropriate to combine the static sliding resistance due to friction and the full passive resistance. The horizontal deformation required to develop 50% of the passive resistance is much smaller. Thus, the contribution of passive pressure is limited to 50% unless the deformation is explicitly considered.

A5.2.2.4 Deleted

A5.2.2.5 Design of Unanchored Components

Requirements for the design of unanchored components are derived from ASCE 43 (Ref. 6.2.6.8) which is compatible with DOE-STD-1020. ASCE 43 also contains analytical methods to quantify the amount of sliding.

Exercise caution when using coefficients of friction contained in engineering handbooks:

- Ensure that the surface finish in the handbook is compatible with the actual application. For example, Marks Handbook identifies a static coefficient of friction for hard steel on hard steel of 0.78 in one table but does not clearly note that the values are for dry, degreased, uncoated steel without an oxide film – a rare condition in an operating nuclear facility. Adding the oxide film alone reduces the coefficient of friction by a factor of about 3.
- Handbook values may represent the results of a single test, the average of a small number of test, or the range of a small number of test. These values seldom correspond to the 95% exceedance level. Coefficients of friction with a 95% confidence level are required to meet the DOE-STD-1020 performance goals.
- Reference A18 contains an extensive study of the coefficient of friction between structural steel surfaces with different surface preparations. The coefficient of variation (COV = Standard Deviation / Average) for friction between steel sections with clean mill scale is 0.21 or less, while the COV between as-received hot-dipped galvanized steel sections is 0.38 or less. Given that the 95% exceedance level is roughly 1.64 standard deviations below the average: then the 95% exceedance level is (1-1.64*0.21)=0.65 times the average coefficient of friction for steel with clean mill scale and 0.37 times the average coefficient of friction for hot-dipped galvanized steel. This example illustrates that there is considerable variability in the coefficient of friction and the use of average values can be grossly unconservative.

A5.3 Miscellaneous Requirements

A5.3.2 Steel Erection Requirements

29CFR 1926, Subpart R contains additional requirements for column stability, connection details, steel joists, roof and floor openings, slip resistance, prohibition of shop placement of shear connectors and systems-engineered metal buildings.

A5.3.4 Anchorage to Concrete

IBC Section 1911 and 1912 requirements for anchorage to concrete are not required for PC-0 SSC.

A5.3.7 Lift Points

For below-the-hook requirements see also ASME B30.20, “Below-the-Hook Lifting Devices” (Ref A20).

A5.3.8 Fragility Analysis

Alternative analyses can range from a demonstration that the “basic intention” of DOE-STD-1020 and ASCE43 has been met to a full fragility analysis which demonstrates that the annual probability of failure is less than the DOE-STD-1020 and ASCE 43 performance goal.

The basic intention of DOE-STD-1020 as stated in Section 2.4.3 “Basic Intention of Dynamic Analysis Based Deterministic Seismic Evaluation and Acceptance Criteria,” is to achieve less than a 10% probability of unacceptable performance for a SSC subjected to 150% of the Design/Evaluation Basis Earthquake. ASCE 43 augments this basic intention by requiring:

1. less than about a 1% probability of unacceptable performance for the Design Basis Earthquake Ground Motion, and
2. less than about a 10% probability of unacceptable performance for a ground motion equal to 150% of the Design Basis Earthquake Ground Motion.

The additional criterion required by ASCE 43 is require for seismic evaluations.

Table A5.2.1.9-1 - SDC-3 Site Specific Spectra
(PC-3 is equivalent to SDC-3)

Frequency (Hz)	Horizontal Spectral Acceleration (g)	Vertical motion per ASCE 4	
		Spectral Acceleration (g)	Smoothed Spectral Acceleration (g)
0.10	0.0075	0.00500	0.00500
0.20	0.030	0.02000	0.02000
0.30	0.065	0.04333	0.04333
0.40	0.104	0.06933	0.06933
0.50	0.135	0.09000	0.09000
0.60	0.1677	0.11180	0.11180
2.00	0.375	0.25000	0.25000
2.25	0.375	0.25000	0.26607
2.50	0.375	0.25000	0.28044
2.75	0.375	0.25000	0.29344
3.00	0.375	0.25000	0.30531
3.25	0.375	0.26959	0.31623
3.50	0.375	0.28772	0.32634
3.75	0.375	0.30460	0.33575
4.00	0.375	0.32040	0.34456
4.25	0.375	0.33523	0.35283
4.50	0.375	0.34922	0.36063
4.75	0.375	0.36245	0.36800
5.00	0.375	0.37500	0.37500
9.00	0.375	0.37500	0.37500
33.00	0.160	0.16000	0.16000
100.00	0.160	0.16000	0.16000

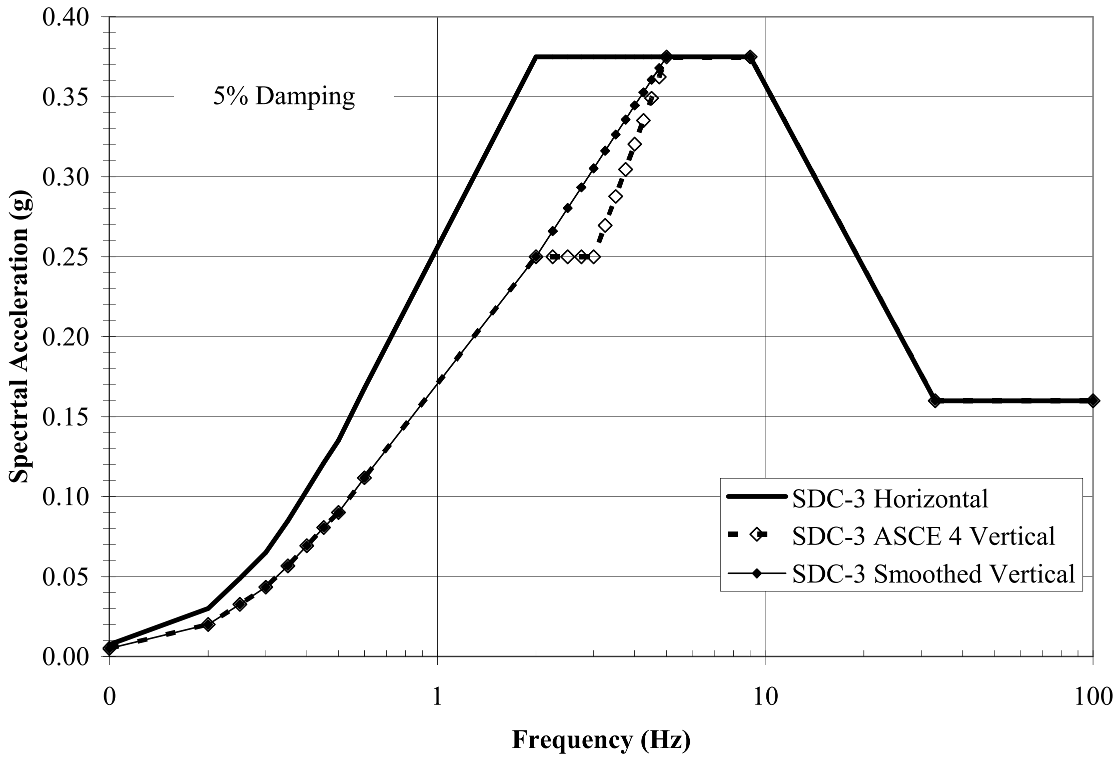


Figure A5.2.1.9-1 Site Specific Spectra, SDC-3 Horizontal and ASCE 4 and Smoothed Vertical Spectra
(PC-3 is equivalent to SDC-3)

Table A5.2.1.9-2 – SDC-5 Site Specific Spectra, Horizontal 5% Damping

******Provided for Information Only******

Reference 6.2.14.3

Frequency (Hz)	Horizontal Spectral Acceleration (g)
0.1	0.0067
0.2	0.0311
0.3	0.0632
0.4	0.1519
0.5	0.3411
0.7	0.4813
1.5	0.760
7	0.655
18	0.350
33	0.227
100	0.227

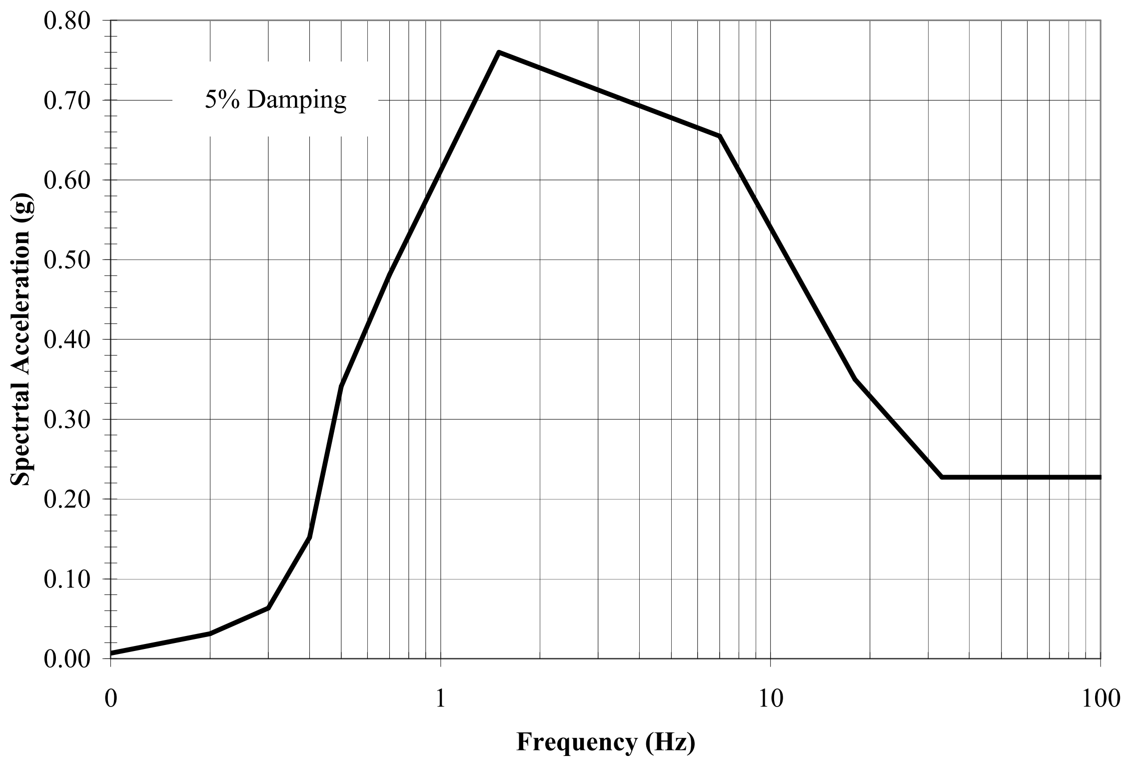


Figure A5.2.1.9-2 SDC-5 Site Specific Spectra, Horizontal 5% Damping

******Provided for Information Only******

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A12 "Generation of PC-3 Horizontal 0.5 % Damping Design Basis Spectra, Compatible with SRS Standard 01060, Revision 5 (U)". Engineering Calculation T-CLC-G-00168.

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