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Design-Load Basis for LANL Structures, Systems, and Components

Los Alamos
NATIONAL LABORATORY
EST.1943

The World's Greatest Science Protecting America

LA-14165
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Isabel Cuesta

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Design-Load Basis for LANL Structures, Systems, and Components

by

Isabel Cuesta

Abstract

This document supports the recommendations in the Los Alamos National Laboratory (LANL) Engineering Standard Manual (ESM), Chapter 5–Structural providing the basis for the loads, analysis procedures, and codes to be used in the ESM. It also provides the justification for eliminating the loads to be considered in design, and evidence that the design basis loads are appropriate and consistent with the graded approach required by the Department of Energy (DOE) Code of Federal Regulation Nuclear Safety Management, 10, Part 830.

This document focuses on (1) the primary and secondary natural phenomena hazards listed in DOE-G-420.1-2, Appendix C, (2) additional loads not related to natural phenomena hazards, and (3) the design loads on structures during construction.

1 Introduction

1.1 Purpose

The purpose of this document is to support the recommendations in the Los Alamos National Laboratory (LANL) Engineering Standard Manual (ESM), Chapter 5–Structural, and provide the basis for the loads, analysis procedures, and codes to be used in the ESM. It also provides the justification for eliminating the loads to be considered in design and support that the design-basis loads are appropriate and consistent with the graded approach required by the Department of Energy (DOE) Code of Federal Regulation Nuclear Safety Management, 10, Part 830.

1.2 Background

The main objective of DOE-O-420.1A is to establish facility safety requirements for the Department of Energy, including the National Nuclear Security Administration. These requirements are as follows:

- Nuclear safety design,
- Criticality safety,
- Fire protection,
- Natural phenomena hazards mitigations, and a
- System engineer program.

DOE-O-420.1A establishes the Natural Phenomena Hazards (NPH) program as "all DOE nuclear and nonnuclear facilities must be designed, constructed, and operated so that public, workers, and environment are protected from the adverse impacts of NPH." DOE-G-420.1-2 classifies the NPH into the two following categories:

- Primary Natural Phenomena Hazards (direct natural phenomena) and
- Secondary Natural Phenomena Hazards (indirect natural phenomena, caused by a primary NPH).

The Order also requires that the Structures, Systems, and Components (SSCs) must be designed, constructed, and operated to withstand the effects of natural phenomena hazards as necessary to ensure the following:

- The confinement of hazardous material,
- The operation of essential facilities,
- The protection of government property, and
- The protection of life safety for occupants of DOE buildings.

These requirements are also valid for additions or major modifications to existing SSCs. Also in DOE-G-420.1-2, Section 6.4.1, it is stated that a very thorough assessment of historical seismicity, geology, geotechnology, meteorology, and hydrology is required for the most hazardous facilities. All potential sources of severe natural phenomena must be identified, and their potential effect at the site must be evaluated. Investigations to establish the potential for soil failure, such as liquefaction and fault displacement, are required.

1.2.1 Graded Approach

LANL facilities are diverse enough to warrant a graded approach (e.g., some are office buildings but others have hazardous radioactive and chemical materials). DOE 10 CFR 830.3 determines this grading process as a function of the following parameters:

- The relative importance to safety, safeguards, and security,
- The magnitude of any hazards involved,
- The live-cycle stage of the facility,
- The programmatic mission of the facility,
- The particular characteristics of the facility,
- The relative importance of radiological and nonradiological hazards, and
- Any other relevant factor.

1.2.2 Performance Category

The NPH Guide DOE G 420.1-2 references Performance Categories (PCs). Performance goals are expressed as the mean annual probability of exceedance of acceptable behavior limits of structures and equipment as a result of the effects of natural phenomena. Five PCs have been established in this DOE guideline ranging from conventional buildings (PC-1) to nuclear-type facilities (PC-4). Another performance category, PC-0, is also considered for structures that do not require design for NPH (e.g., sidewalks). DOE-STD-1021 provides criteria for selecting the PCs of SSCs in accordance with DOE-O-420.1A for the purpose of mitigating NPH in all DOE facilities.

The concept of PC with corresponding target probabilistic performance goals has been developed to assist in applying the graded approach to NPH design and evaluation. The SSC in LANL are assigned PC-1, PC-2, PC-3, and PC-4 (out of the five existing performance categories), depending upon its safety importance. Each PC is assigned a target performance goal in terms of the probability of unacceptable damage resulting from specific natural phenomena. The unacceptable level of damage is related to the safety function of the SSC during and after the occurrence of NPH.

According to DOE-G-420.1-2, for *PC-1 SSCs*, the primary concern is preventing major structural damage, collapse, or other failure that would endanger personnel (life safety).

The design and evaluations of PC-1 SSCs are based on current building codes (IBC 2003, Seismic Use Group I).

PC-2 SSCs are meant to ensure the operability of essential facilities or to prevent physical injury to in-facility workers.

Design of PC-2 SSCs should result in limited structural damage from design-basis natural phenomena events to ensure minimal interruption to facility operation and repair following the event. The design and evaluations of PC-2 SSCs are similar to the design criteria for essential facilities in current building codes (IBC 2003, Seismic Use Group III).

DOE-STD-1020-2002 establishes that PC-1 and PC-2 SSCs are to be designed following the most recent model building code, which is the International Building Code (IBC) 2003. The IBC 2003 refers to ASCE 7-02 for supplemental evaluation of the design loads.

PC-3 SSCs are those for which failure to perform their safety function could pose a potential hazard to public health and the environment because radioactive or toxic materials are present and could be released from the facility as a result of that failure.

Design considerations consist on limiting the facility damage as a result of design-basis natural phenomena events so that hazardous materials can be controlled and confined, occupants are protected, and the functioning of the facility is not interrupted. PC-3 NPH provisions are similar to those used for the reevaluation of commercial plutonium facilities.

PC-4 SSCs are also those for which failure to perform their safety function could pose a potential hazard to public health and the environment because radioactive or toxic materials are present *in large quantities* and could be released as a result of that failure. The quantity of hazardous materials and energetics is similar to a large Category-A reactor (> 200 MW_t).

Design considerations for this category are to limit facility damage from design-basis natural phenomena events so that hazardous materials can be controlled and confined, occupants are protected, and essential functions of the facility are not interrupted. PC-4 seismic provisions are similar to those used for the reevaluation or design of civilian nuclear power plants, where off-site release of hazardous material must be prevented.

1.3 Scope

SSCs should be designed, constructed, and operated to withstand the effects of natural phenomena as necessary to ensure the confinement of hazardous material, the operation of essential facilities, the protection of government property, and the protection of occupants of LANL buildings.

This document will select defensible and appropriate design loads for LANL SSCs. Using information from historical records, regional geological maps, and other investigations, models are developed to estimate the likelihood of natural phenomena of various magnitudes impacting a site.

1.4 Plan of Development

This document is made up of four main chapters. The first chapter documents the primary natural phenomena hazards listed in DOE-G-420.1-2, Appendix C. The second chapter documents the secondary natural phenomena hazards listed in DOE-G-420.1-2, Appendix C. The third chapter refers to all the additional loads not related to natural-phenomena hazards. Finally, the fourth chapter refers to the design loads on structures during construction.

1.5 Acknowledgements

Numerous individuals contributed to the writing and to the understanding of the natural phenomena hazards mentioned in this document. I particularly would like to acknowledge and thank Effiok Etuk, Jamie Gardner, Tom Houston, Robert P. Kennedy, Alexis Lavine, Tobin Oruch, Glen Pappas, Michael Salmon, and Douglas Volkman for their contributions and insight. This work has been funded through the Department of Energy.

1.6 References

Concrete/Masonry

ACI 318-02	Building Code Requirements for Structural Concrete
ACI 349-01	Code Requirements for Nuclear Safety-Related Concrete
ACI 530/ASCE 5/TMS 402-02	Building Code Requirements for Masonry Structures

Steel

AISC ASD	Manual of Steel Construction, Allowable Stress Design, 9th Edition, 1989
AISC LRFD	Manual of Steel Construction, Load & Resistance Factor Design, 3rd Edition, 2001
AISC LRFD/ASD	Specification for Structural Steel Buildings, 2005
AISC N690	Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, 1994, with Supplement N. 1, ANSI/AISC N690-1994s1, April 2002
AISC/ANSI N690L	LRFD Specification for Safety-Related Steel Structures for Nuclear Facilities, December 17, 2003
AISC/ANSI 341	Seismic Provisions for Structural Steel Buildings, 2002

Industry Standards and Guidance

ASCE 4-98	Seismic Analysis of Safety-Related Nuclear Structures
ASCE 7-02	Minimum Design Loads for Buildings and Other Structures
ASCE 24-98	Flood Resistant Design and Construction
ASCE 32-01	Design and Construction of Frost-Protected Mustow Foundations
ASCE 37-02	Design Loads on Structures During Construction
ASCE XXX-03	Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities and Commentary, draft

Building Code

IBC 2003	International Building Code

DOE Documents

DOE 10 CFR 100	Reactor Site Criteria
DOE 10 CFR 830	Nuclear Safety Management, 2001
DOE 40 CFR 264	Standards for Owners and Operators of Hazardous Waste Treatment, Storage, and Disposal Facilities
DOE 29 CFR 1926	Safety and Health Regulations for Construction
DOE G 420.1-1 -00	Nonreactor Nuclear Safety Design Criteria and Explosives Safety Criteria Guide for use with DOE O 420.1, Facility Safety
DOE G 420.1-2 -00	Guide for the Mitigation of Natural Phenomena Hazards for DOE Nuclear Facilities and Nonnuclear Facilities
DOE M 440.1-1-96	Explosive Safety Manual
DOE O 420.1A -04	Facility Safety
DOE O 6430.1A-89	General Design Criteria
DOE O 5480.28	Natural Phenomena Hazards Mitigation
DOE-STD-1020-94	Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities (superseded by DOE-STD-1020-02)

DOE Documents (Cont.)

DOE-STD-1020-02	Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities
DOE-STD-1021-93	Natural Phenomena Hazards Performance Categorization Guidelines for Structures, Systems, and Components, Reaffirmed with Errata, April 2002
DOE-STD-1022-94	Natural Phenomena Hazards Performance site Characterization Criteria Reaffirmed with Errata, April 2002
DOE-STD-1023-95	Natural Phenomena Hazards Assessment Criteria, Reaffirmed with Errata, April 2002
DOE-STD-1024-92	Guidelines for use of Probabilistic Seismic Hazard Curves at DOE sites (Change notice #1, 1996)
DOE-STD-1066-99	Fire Protection Design Criteria
DOE-STD-1088-95	Fire protection for Relocatable Structures
DOE-STD-3014-96	Accident Analysis for Aircraft Crash into Hazardous Facilities
DOE-NRC RG 1.132	Site Investigations for Foundations of Nuclear Power Plants
DOE-NRC RG 1.165	Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motions

2 Primary Natural Phenomena Hazards

2.1 *Earthquakes*

2.1.1 Regional Geology

The Los Alamos National Laboratory is located in northern New Mexico and lies within the Rio Grande rift, which is an area of active crustal extension that spreads from southern Colorado to northern Mexico.

2.1.2 Seismic Source Zones: The Rio Grande Rift

Seismic source zones are area sources defined by their unique tectonic, geologic, and seismologic characteristics. Within the LANL region, there are four zones as shown in Table 2-1: the Rio Grande rift, the Colorado Plateau transition zone, the Great Plains, and the Southern Rocky Mountains. Background (random) earthquakes that are not associated with distinct tectonic features (e.g., faults or folds) are assumed to occur within theses seismic source zones. Of the four local seismic zones, the Rio Grande rift source zone has the highest potential hazard for background earthquakes at LANL (Wong et al., 1996).

Table 2-1. Seismic Source Zones (from Wong et al., 1995)

Seismic Source Zones	**Background (random) Earthquake Maximum Magnitude (M_W*)**
Rio Grande Rift	6.3 ± 0.3
Colorado Plateau Transition	6.0 ± 0.3
Southern Rocky Mountains	6.3 ± 0.3
Great Plains	6.0 ± 0.3

*M_W = Moment Magnitude

The Rio Grande rift is an active seismotectonic source zone in the western United States and has a seismic and volcanic history spanning the last 30 million years. The rift is an area of east-west crustal extension and is expressed on the earth's surface as a series of generally north-south-striking, elongated basins extending from Leadville, Colorado, to northern Mexico (e.g., Baldridge et al., 1984). In Northern New Mexico, the rift includes the San Luis basin, the Española basin, the Santo Domingo basin, and the Albuquerque basin.

Crustal extension in the Rio Grande rift continues today and is evident through high heat flow, hot springs, continued seismicity, geodetic observations, and recent lava flows.

Historic seismicity in the basins of the Rio Grande rift is characterized by widespread, abundant microseismic events, temporally punctuated by larger earthquakes that range from the limits of human perceptibility to potentially seriously damaging earthquakes of an approximate magnitude of 7.0. Significant history earthquakes in the rift include the magnitude 7.2 Sonora earthquake in 1887 and two approximate magnitude-6.0 earthquakes in Socorro in 1906. The 1918 Cerrillos earthquake occurred 50 km southeast of LANL and had an estimated M_L = 5.5.

2.1.3 Faults: The Pajarito Fault System

Los Alamos lies near several major boundary faults of the Rio Grande rift in north-central New Mexico. The western margin of the Rio Grande rift in the Los Alamos area is locally defined by the Pajarito fault system. The Pajarito fault system is an approximately 41-km-long, generally north-striking system of normal faults. The three major faults of this system are the Pajarito fault, the Rendija Canyon fault, and the Guaje Mountain fault (see Figure 2-1). All of these faults have significant displacement, in some places larger than 150 meters, on the 1.2 million-year old Bandelier Tuff, and paleoseismic studies suggest three Holocene (last 11,000 years) paleoseismic events on the Pajarito fault system.

The Pajarito fault is the main down-east rift-bounding fault, and is made up of a complex zone of primarily north- to northeast-striking normal faults and folds located along the west margin of LANL. The antithetic (down-west) Rendija Canyon fault is located ~3 km east of the Pajarito fault, strikes north to northeast, and dips steeply to the west. South of Los Alamos Canyon, near the northern boundary of LANL, the Rendija Canyon fault splays to the southwest into a broad zone of faulting and folding, with displacement diminishing to the southwest. The Guaje Mountain fault is located 1 to 2 km east of the Rendija Canyon fault, and is similar to it in orientation, structural style, and sense of slip. The Guaje Mountain fault has not been mapped south of Bayo Canyon. Table 2-2 shows the rupture length, best-estimate maximum magnitude, style of faulting, and slip rate for each of the three faults (Wong et al., 1995). Wong et al., however, did not recognize any Holocene events on the Pajarito fault. Recent work indicates three Holocene paleoseismic events on the Pajarito fault that may imply slip rates roughly six times higher than those of Wong et al.

Because the Pajarito fault is rift derived, and the Rendija Canyon fault and Guaje Mountain fault are antithetic to it, the hypothetical possibility exists that the extensional sweep of the Pajarito fault at depth defines the terminal boundary of the Rendija Canyon and Mountain faults.

2.1.4 Seismic Hazard Investigations

For sites containing PC-1 and PC-2 SSCs, DOE-STD-1022 states that previous site-specific probabilistic seismic-hazard studies, if available, or information provided in the model building codes, such as the IBC code, or national consensus standards can be used. The selection is made by the code, employing seismic zone maps and a specified frequency spectrum. However, IBC and DOE-STD-1023 allow specific site investigation and development of a site-specific seismic hazard assessment for use in the design. The SSCs must be evaluated or designed for the greater of the site-specific values or the model code values unless site-specific values are lower and can be justified.

For sites containing PC-3 or PC-4 SSCs, a site-specific characterization of the seismic-related hazards is required. The investigation depends on the performance category, geologic, and seismologic environment, and the local soil conditions at the site.

DOE-STD-1023 describes methods for conducting a Probabilistic Seismic Hazard Analysis (PSHA) to produce a seismic hazard curve to be used in selecting the design-basis earthquake (DBE) for PC-3 SSCs. Any site whose site-specific hazard curves exceed the USGS curves (for similar site conditions) should continue to use these site-specific curves. For sites containing facilities with SSCs in PC-3 and 4, a site-specific NPH assessment must be conducted in accordance with the applicable DOE standard.

Table 2-2. Pajarito Fault System (data from Wong et al., 1995 and LANL Seismic Hazards Team, unpublished)

Fault	Rupture Length (km)	Best Estimate Maximum Moment Magnitude (M_w)	Style of Faulting	Slip Rate Best Estimate (Range) (mm/yr)
Pajarito	41	6.9 ± 0.3	Normal	0.09 (0.01–0.95) Quaternary 0.6 (?) Holocene
Rendija Canyon	~10	6.5 ± 0.3	Normal	0.02 (0.01–0.25)
Guaje Mountain	~9	6.5 ± 0.3	Normal	0.01 (0.01–0.14)

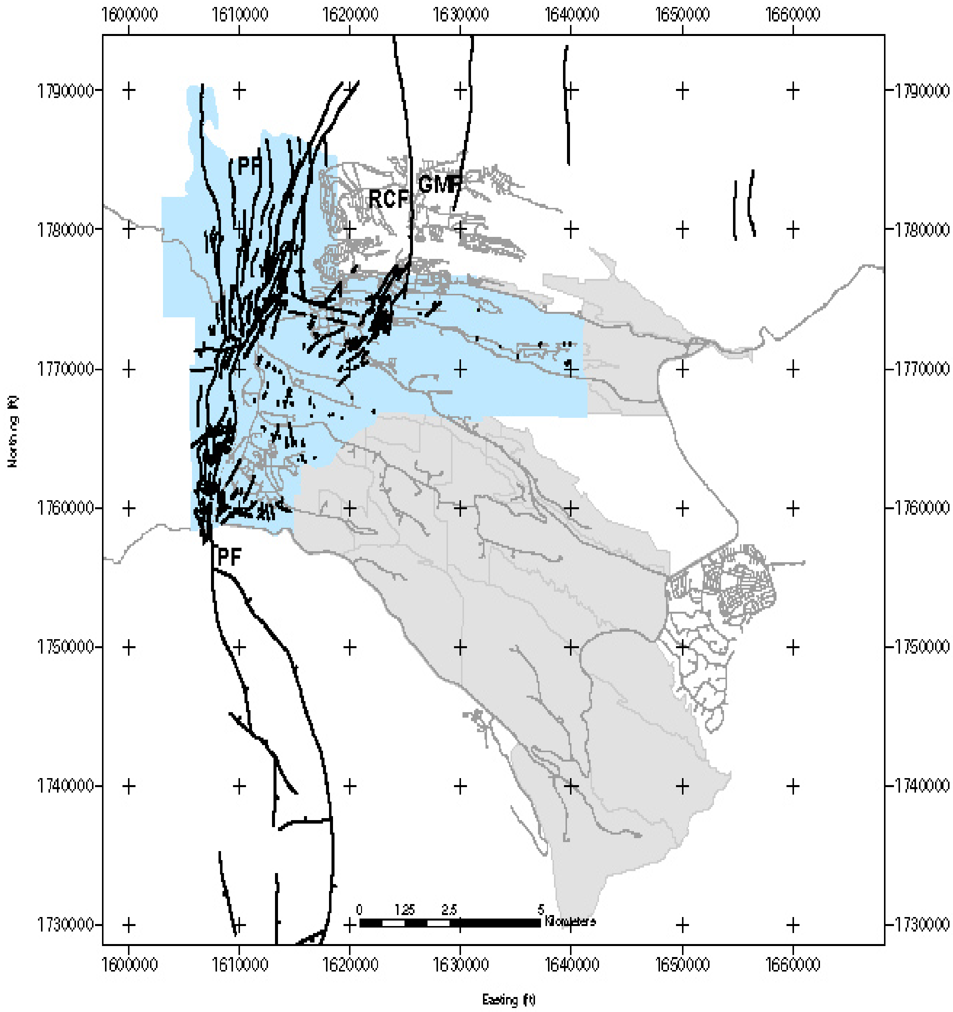


Figure 2-1. Map of the Pajarito Fault System in the Area of LANL. Shaded gray area is LANL, shaded blue area is the area of detailed geologic mapping, and the black lines are faults and related structures from Gardner and House (1987), Gardner et al. (1998, 1999, 2001), Lewis et al. (2002), Lavine et al. (2003b), and in-progress mapping.

DOE-STD-1022-94 refers to 10 CFR Part 100 for criteria guidelines for the site characterization of PC-3 and PC-4 SSCs. One of the factors of consideration is the geologic and seismic siting criteria. Appendix A of this CFR Part points out the required investigation for seismic ground motion and surface faulting. The investigations required to determine whether and to what extent nuclear power plants need to be designed for surface faulting are presented in 10 CFR 100. NRC RG 1.165 provides general guidance on procedures to conduct geological, geophysical, seismological, and geotechnical investigations. In this document, four areas of investigation are addressed:

- 200-mile radius within the site to identify seismic sources. Regional geological and seismological investigations are not expected to be extensive nor in great detail, but should include literature reviews, the study of maps, and remote sensing data.
- 25-mile radius within the site. Geological, seismological, and geophysical investigations in greater detail than the regional investigations to identify and characterize the seismic and surface deformation potential of any capable tectonic sources and the seismic potential of seismogenic sources.

- 5-mile radius within the site. Detailed geological, seismological, geophysical, and geotechnical investigations of the site to evaluate the potential for tectonic deformation at or near the ground surface and to assess the ground motion transmission characteristics of soils and rocks in the site vicinity (if appropriate). Investigations should include monitoring by a network of seismic stations.
- 0.5-mile radius within the site. Very detailed geological, seismological, geophysical, and geotechnical investigations to assess specific soil and rock characteristics as described in NRC RG 1.132.

2.1.4.1 Seismic-Hazard Investigations in Los Alamos

The first seismic-hazards study at LANL was conducted in 1972 by Dames and Moore. This report was a geologic, foundation, hydrologic, and seismic investigation of the Plutonium Facility at TA-55. Following this early study, Gardner and House (1987) reviewed and compiled all existing studies and data relevant to seismic hazards at LANL to evaluate the need for a new seismic hazards study in light of more modern methodologies. This report stated that an earthquake of Richter Magnitude 6.5–7.8 in the Pajarito fault system could be expected and that existing data indicated that the fault system must be considered "capable" in the definitions of 10 CFR 100. They also concluded that because of the seismic properties of the Bandelier Tuff and the surface geology of Los Alamos being so variable, the responses of different sites within the Laboratory should be analyzed individually for design purposes. This report contains a partially annotated bibliography with all the relevant studies to seismic hazards at LANL up to 1987.

The need to comply with DOE O 5480.28 on NPH Mitigation resulted in a comprehensive seismic hazards evaluation, which was completed in 1995 and conducted for LANL by Woodward-Clyde Inc. of Oakland, California. This four-year program evaluated the earthquake potential and ground-shaking hazard at LANL. In this study, 25 faults and four seismic source zones were identified as seismic sources potentially significant to LANL. The source zones account for the hazard from "background" earthquakes that do not rupture into the surface.

Between 1996 and 2004, a number of paleoseismic studies have been conducted at LANL to improve the data used in the Woodward-Clyde study for both surface rupture and ground-motion hazards. These have included a number of trenches along the Pajarito and Guaje Mountain faults (McCalpin, 1998, 1999; Reneau et al., 2002; Gardner et al., 2003) and detailed geologic mapping of the northern and western parts of LANL and the Pajarito fault escarpment west of LANL (McCalpin, 1997; Gardner et al., 1998, 1999, 2001; Lewis et al., 2002; Lavine et al., 2003b; Lewis et al., in preparation; Lavine et al., in preparation). These more recent efforts have led to a better understanding of fault geometry and kinematics and paleoseismic activity.

DOE-STD-1020-94 established the use of return periods of 500, 1,000, 2,000, and 10,000 years for PC-1, PC-2, PC-3, and PC-4, respectively. A Probabilistic Seismic Hazard Analysis using a logic-tree approach was performed using geologic data and a seismic source characterization as the input data. Site-specific probabilistic peak horizontal and vertical accelerations were determined for the return periods of 500, 1,000, 2,000, and 10,000 years, as shown in Table 2-3 for each PC.

Based on the PSHA, the uniform hazard spectra, and design spectra for each return period were calculated, and synthetic time histories were generated for use in seismic design and seismic safety analyses of LANL facilities. The PSHA is currently being reassessed based on more recent paleoseismic studies.

Table 2-3. Site-Specific Probabilistic Peak Ground Accelerations (Wong et al., 1995)*

Performance Category	Return Period (yr)	Mean Annual Probability of Exceedance, P_H	Design Horizontal PGA (g)	Design Vertical PGA (g)
PC-1	500	20×10^{-4}	0.15	0.11
PC-2	1000	10×10^{-4}	0.22	0.19
PC-3	2000	5×10^{-4}	0.31	0.27
PC-4	10000	1×10^{-4}	0.57	0.58

*Before DOE-STD-1020-2002

Vertical and horizontal hazard-response spectra were determined for the following Technical Areas: 55, 46, 41, 21, 18, 16, 3, and 2 and for the four return periods 500, 1,000, 2,000, and 10,000 years.

DOE O 420.1A requires implementing in all standards the most current model building codes, such as IBC 2003 and the current industry standards for PC-1 and PC-2 facilities.

Since the publication of DOE-STD-1020-94, several new documents have been published that made DOE-STD-1020-94 outdated, as shown in DOE-STD-1020-2002: (1) the 1997 National Earthquake Hazards Reduction Program (NEHRP) introduced new seismic maps for evaluating the seismic hazard, (2) the UBC 97, BOCA, and SBCCI were replaced by IBC 2000 (previous version of IBC 2003 that adopted the 1997 NEHRP), and (3) DOE O 420.1A and DOE G 420.1-2 were approved and adopted the use of IBC 2000 for PC-1 and PC-2 facilities.

The site-specific seismic hazard study for LANL was developed in 1995 and needs to be reviewed and updated per DOE-STD-1023-95 about every 10 years.

2.1.5 LANL Earthquake Loads

2.1.5.1 PC-1 and PC-2 SSCs Seismic Provisions

DOE-STD-1020-2002 establishes that PC-1 and PC-2 SSC are to be designed following the most recent model building code, which is the IBC 2003. The IBC 2003 refers to ASCE 7-02 for supplemental evaluation of the design loads. IBC Sections 1613-1623 and ASCE 7-02 Section 9 correspond to the seismic provisions.

IBC 2003 provides seismic hazard maps defined in terms of the Maximum Considered Earthquake (MCE) ground motions. These maps are associated with a 2,500-year return period earthquake. The graded approach is maintained by using two thirds (2/3) of the MCE and a unity importance factor for PC-1 and 2/3 × MCE and 1.5 importance factor for PC-2 facilities (see Table 2-4).

Table 2-4. LANL Site-Specific Probabilistic Accelerations*

Performance Category	Return Period (year)	Mean Annual Probability of Exceedance, P_H	Design Horizontal PGA (g)	MCE PGA (g)
PC-1	2,500[1]	4×10^{-4}	0.22	0.34
PC-2	2,500[1]	4×10^{-4}	0.22	0.34

Note: PGA = Peak Ground Acceleration.
*After DOE-STD-10210-2002.
[1]Based on 2% Exceedance Probability in 50 years.

IBC Section 1614.1 refers to an alternative seismic design using only ASCE 7-02. IBC seismic provisions can be disregarded when structures are designed in accordance with the provisions of ASCE 7 Sections 9.1 through 9.6, 9.13, and 9.14. However, it is recommended to use the IBC 2003 seismic provisions in combination with ASCE 7 as needed. The main differences between the code and the standard are the following:

- redundancy (IBC Section 1617.2 vs ASCE 7 Section 9.5.2.4),
- the seismic-force-resisting systems (IBC Section 1617.6 vs ASCE 7 Section 9.5.6-8)
- earthquake loads–design detailing requirements and structural components load effects (IBC Section 1620 vs ASCE 7 Section 9.5.2.6)
- architectural, mechanical, and electrical components (IBC Section 1621 vs ASCE 7 Section 9.6)
- nonbuilding structures (IBC Section 1622 vs ASCE 7 Section 9.14)
- seismically isolated structures (IBC Section 1623 vs ASCE 7 Section 9.13)

For all the sections mentioned above, the IBC 2003 modifies ASCE 7-02 and should be followed.

2.1.5.2 PC-1 and PC-2 Design Response Spectra

Earthquake loads determined by IBC 2003 seismic design methodology are based on the following spectral response parameters:

- S_S = mapped MCE spectral-response acceleration at short periods for Site Class B.
- S_1 = mapped MCE spectral-response acceleration at 1-s period for Site Class B.

LANL design spectral response accelerations are defined for Site Class B at 2% probability of exceedance in a 50-year building life (2,500 year average return period) from the maps in IBC and the 1996 mapped values in the USGS web site. The 2,500-year mapped rock spectral accelerations at LANL (35º52'N and 106º19'W) are:

- $S_S = 0.60$ g
- $S_1 = 0.19$ g

LANL site conditions are typically considered to be Site Class D. Site Class D is a stiff soil profile where average shear wave velocities over the top 100 feet are between 600 and 1,200 ft/s. Based on IBC 2003, the site specific factors F_a and F_v are 1.32 and 2.04, respectively. Therefore, the soil-modified MCE spectral response accelerations at short and at 1-s period are:

- $S_{MS} = F_a \times S_S = 1.32 \times 0.60\text{ g} = 0.79\text{ g}$
- $S_{M1} = F_v \times S_1 = 2.04 \times 0.19\text{ g} = 0.39\text{ g}$

The MCE defined by IBC has a 2% probability of exceedance in 50 years (approximately 2,500 yr return period). These values may be compared to LANL site-specific ground response spectra for 2,500 years. The LANL seismic hazard study (Wong et al. 1995) reports a peak response spectrum of about 0.81g and 0.39 g at 1-s period.

The design spectral response accelerations at short periods and at a 1-s period are defined as 2/3 the MCE by IBC Section 1615.1.3:

- S_{DS} = 5% damped design spectral response acceleration at short periods (0.2 s) = 2/3 S_{MS}
- S_{D1} = 5% damped design spectral response acceleration at 1-s period = 2/3 S_{M1}

Therefore, using the LANL site-specific values of MCE, the design spectral response accelerations are

- $S_{DS} = 2/3 \times 0.81\text{ g} = 0.54\text{ g}$
- $S_{D1} = 2/3 \times 0.39\text{ g} = 0.26\text{ g}$

The design response spectrum $S_a(g)$ also has the following parameters:

- $T_o = 0.2 \times S_{DS}/S_{D1} = 0.1$ s
- $T_s = S_{D1}/S_{DS} = 0.48$ s
- PGA = $0.4 \times S_{DS} = 0.22$ g

The horizontal design response spectrum can be determined as follows:

$$S_a(g) = \begin{cases} 0.22 + 3.36T & T < 0.1s \\ 0.54 & 0.1\,s \le T < 0.48\,s \\ \dfrac{0.26}{T} & T \ge 0.48\,s \end{cases}.$$

The vertical design response spectrum in accordance with IBC is given implicitly by adding a scaled dead load as shown:

$$E_v = \pm\, 0.2 \times S_{DS} \times D = \pm\, 0.11D,$$

where D is the Dead load.

Figure 2-2 shows the graphical representation of the design response spectrum for PC-1 and PC-2 SSCs.

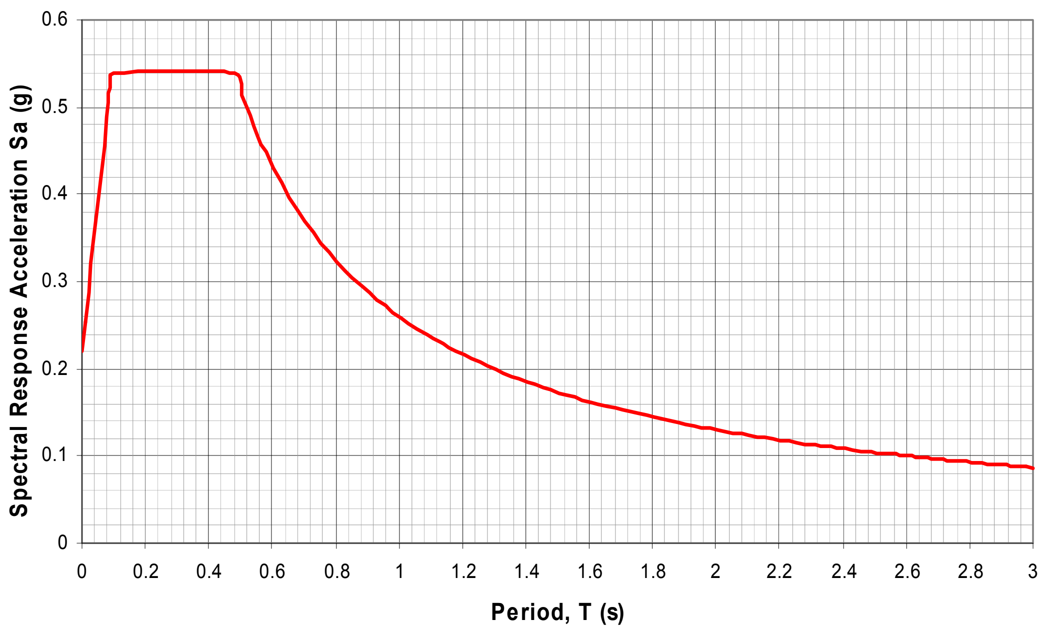


Figure 2-2. Horizontal Design Response Spectrum for PC-1 and PC-2 SSCs.

Other parameters required for the use of the IBC seismic provisions are shown in Table 2-5. The seismic importance factor, I_E, for PC-2, is used to reduce the response modification factor, R, as a means of controlling damage and producing "enhanced" performance.

Table 2-5. Classification of Buildings and Importance Factor

	PC-1	PC-2
Occupancy Category[1]	II	IV
Seismic Use Group[2]	I	III
Seismic Factor[1], I_E	1.0	1.5
Seismic Design Category[3]	D	D

[1]IBC Table 1604.5
[2]IBC Section 1616.2
[3]IBC Section 1616.3

IBC Section 1616.3 for the determination of the Seismic Design Category (SDC) has one exception that is not applicable to LANL. It states that the SDC is permitted to be determined from Table 1616.3(1) alone when three conditions apply. However, S_{DS} is larger than 0.50 g (S_{DS} = 0.54 g), and the SDC is also D regardless of the S_{D1} value.

2.1.5.3 PC-1 and PC-2 Analysis Procedures

Because the Seismic Design Category of all structures to be designed at LANL is D, the Index Force analysis procedure of ASCE 7 (Section 9.5.3) is not allowed. Figure 2-3 shows a flow chart indicating if the design of the structure requires the use of static or dynamic analysis. This determination is based on the structure irregularity type in accordance with ASCE 7, and the period of the building.

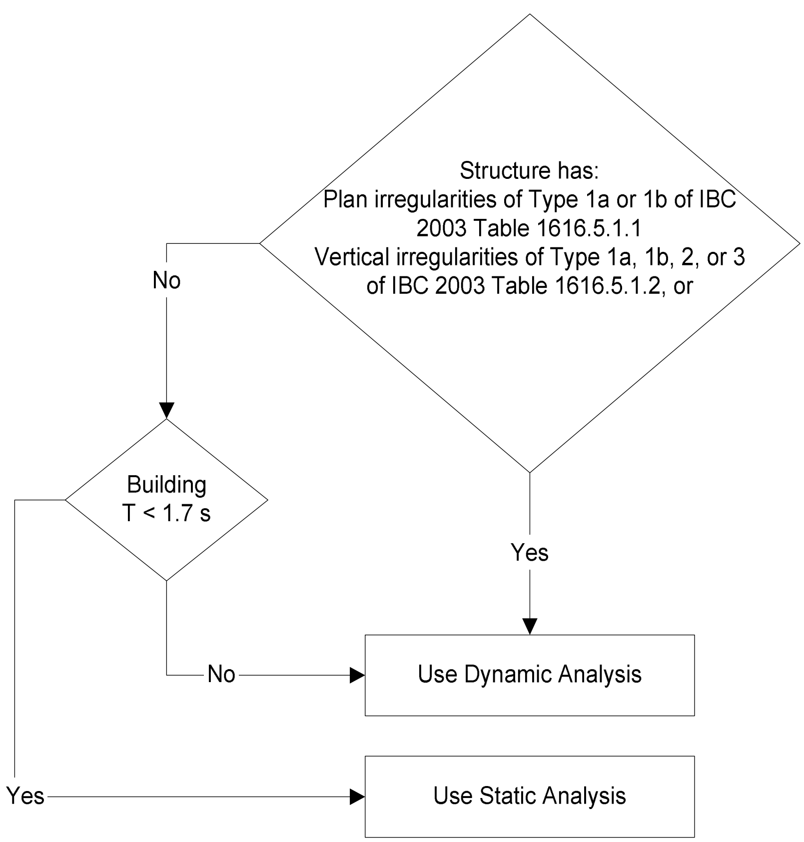


Figure 2-3. Flow chart for analysis procedure decision making for PC-1 and PC-2 SSCs.

Table 2-6. Permitted Analytical Procedures for PC-1 and PC-2 SSCs

Structural Characteristics	Static Analysis: Index Force (ASCE 7 Sect. 9.5.3)	Static Analysis: Simplified (IBC 2003 Sect. 1617.5)	Static Analysis: Equivalent Lateral Force (ASCE 7 Sect. 9.5.5)	Dynamic Analysis (ASCE 7): Modal Response Spectrum (Sect. 9.5.6)	Dynamic Analysis (ASCE 7): Linear Response History (Sect. 9.5.7)	Dynamic Analysis (ASCE 7): Nonlinear Response History (Sec. 9.5.8)
PC-1 buildings of light-framed construction not exceeding three stories in height	NP	P	P	P	P	
Other PC-1 buildings not exceeding two stories in height when constructed of any material with flexible diaphragms at every level.	NP	P	P	P	P	P
PC-1 and PC-2 regular structures with T < 1.7 s and all structures of light-frame construction not covered previously.	NP	NP	P	P	P	P
PC-1 and PC-2 irregular structures with T < 1.7 s and having only plan irregularities type 2, 3, 4, or 5 of Table 9.5.2.3.2 or Vertical irregularities, type 4 or 5 of Table 9.5.2.3.3	NP	NP	P	P	P	P
All other PC-1 and PC-2 structures	NP	NP	NP	P	P	P

Note: P – indicates permitted, NP – indicates not permitted

Once the decision has been made (whether to use static or dynamic analysis), there are several different procedures for each type of analysis. However, the use of one or another depends on the structural characteristics of the structure. Table 2-6 shows the permitted analytical procedures for the LANL PC-1 and PC-2 structures. This table has been modified from the ASCE 7 Table 9.5.2.5.1.

2.1.5.4 PC-1 and PC-2 Seismic-Force-Resisting Systems

Because the Seismic Design Category of all the structures at LANL is D, the following list shows the seismic-force-resisting systems that are not permitted to be used by the IBC code (Table 1617.6.2):

1. Bearing-Wall Systems
   - Reinforced concrete shear walls: Ordinary
   - Plain concrete shear walls: Ordinary and Detailed.
   - Reinforced masonry shear walls: Ordinary and Intermediate
   - Plain masonry shear walls: Ordinary and Detailed
   - Prestressed masonry shear walls: Ordinary and Intermediate.
2. Building-Frame Systems
   - Reinforced-concrete shear walls: Ordinary
   - Plain concrete shear walls: Ordinary and Detailed.
   - Composite braced frames: Ordinary
   - Composite reinforced-concrete shear walls with steel elements: Ordinary
   - Reinforced masonry shear walls: Ordinary and Intermediate
   - Plain masonry shear walls: Ordinary and Detailed
   - Prestressed masonry shear walls: Ordinary and Intermediate.

3. Moment-Resisting Frame Systems
   - Steel moment frames: Ordinary. Except (1) for buildings up to 35 ft, where the dead load of the walls, floors, and roof does not exceed 15 lb/ft^2, and (2) a one-story building up to 60 ft high, when the moment joints of field connections are constructed of bolted end plates and the dead load of the roof does not exceed 15 lb/ft^2. The dead weight of the portion of the walls more than 35 ft above the base must not exceed 15 lb/ft^2.
   - Reinforced concrete moment frames: Ordinary and Intermediate
   - Composite moment frames: Ordinary and Intermediate.
4. Dual Systems with Special Moment Frames
   - Reinforced concrete shear walls: Ordinary
   - Composite reinforced concrete shear walls with steel elements: Ordinary
   - Reinforced masonry shear walls: Intermediate
5. Dual Systems with Intermediate Moment Frames (steel intermediate moment-resisting frames are not permitted)
   - Reinforced concrete shear walls: Ordinary
   - Reinforced masonry shear walls: Ordinary and Intermediate
   - Braced frames: Ordinary composite
   - Composite reinforced concrete shear walls with steel elements: Ordinary
6. Shear Wall-frame Interactive System with Ordinary Reinforced Concrete Moment Frames and Ordinary Reinforced Concrete Shear Walls
7. Inverted Pendulum systems
   - Steel moment frames: Ordinary
8. Structural Steel Systems Not Specifically Detailed for Seismic Resistance.

The basic lateral and vertical seismic-force-resisting systems that are permitted to be used at LANL are indicated in Table 2-7. Each type is subdivided by the types of vertical element used to resist lateral seismic forces. Table 2-7 tabulates the response modification coefficient, R, the system overstrength factor, Ω_o, the deflection amplification factor, C_d, and the building height limitation for each of the specific seismic-force-resisting systems. These coefficients will be used in determining the base shear, element design forces, and design-story drift. Table 2-8 indicates the detailing reference section of IBC, ACI-318, ACI-530, etc., that is required for each of the systems listed in Table 2-7.

2.1.5.5 PC-3 and PC-4 SSCs Seismic Provisions

Facilities classified as PC-3 and PC-4 have missions that are critical to the National Nuclear Security Administration (NNSA), or contain operations with significant risk potential to public, worker, and environment safety. Following the graded approach philosophy outlined in DOE Order 420.1A, DOE G 420.1-2, and DOE-STD-1020, the design of PC-3 and PC-4 SSCs follows more stringent and conservative methods than used in model building codes, but more like methods used in practice for nuclear power plant design, where off-site release of hazardous materials must be prevented.

Earthquake induced loads are based on site-specific studies at specified annual probabilities of exceedance in accordance with DOE-STD-1020.

Table 2-7. Design Coefficients and Factors for PC-1 and PC-2 LANL Basic Seismic-Force-Resisting Systems

Basic Seismic-Force-Resisting System	Response-Modification Coefficient, R	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Building Height Limitation (ft)[a,b]
1. Bearing Wall Systems				
A. Ordinary steel-braced frames in light-frame construction	4	2	3 ½	65
B. Special reinforced concrete shear walls	5 ½	2 ½	5	160
C. Special reinforced masonry shear walls	5	2 ½	3 ½	160
D. Light-frame walls with shear panels-wood structural panels/sheet steel panels	6 ½	3	4	65
E. Light framed walls with shear panels-all other materials	2	2 ½	2	35
F. Special prestressed masonry shear walls	4 ½	2 ½	3 ½	35
2. Building Frame Systems				
A. Steel eccentrically braced frames, moment-resisting, connections at columns away from links	8	2	4	160
B. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	7	2	4	160
C. Special steel concentrically braced frames	6	2	5	160
D. Ordinary steel concentrically braced frames	5	2	4 ½	35[d]
E. Special reinforced-concrete shear walls	6	2 ½	5	160
F. Composite eccentrically braced frames	8	2	4	160
G. Composite concentrically braced frames	5	2	4 ½	160
H. Composite steel plate shear walls	6 ½	2 ½	5 ½	160
I. Special composite reinforced-concrete shear walls with steel elements	6	2 ½	5	160
J. Special reinforced-masonry shear walls	5 ½	2 ½	4	160
K. Light frame walls with shear panels-wood structural panels/ sheet steel panels	7	2 ½	4 ½	65
L. Light framed walls with shear panels-all other materials	2 ½	2 ½	2 ½	35
M. Special prestressed masonry shear walls	4 ½	2 ½	4	35
3. Moment-Resisting Frame Systems				
A. Special steel moment frames	8	3	5 ½	NL
B. Special steel truss moment frames	7	3	5 ½	160
C. Intermediate steel moment frames	4 ½	3	4	35[e]
D. Special reinforced-concrete moment frames	8	3	5 ½	NL
E. Special composite moment frames	8	3	5 ½	NL
F. Composite partially restrained moment frames	6	3	5 ½	100
G. Masonry wall frames	5 ½	3	5	160
4. Dual Systems with Special Moment Frames				
A. Steel eccentrically braced frames, moment resisting, connections at columns away from links	8	2 ½	4	NL
B. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	7	2 ½	4	NL
C. Special steel concentrically braced frames	8	2 ½	6 ½	NL
D. Special reinforced-concrete shear walls	8	2 ½	6 ½	NL
E. Composite eccentrically braced frames	8	2 ½	4	NL
F. Composite concentrically braced frames	6	2 ½	5	NL
G. Composite steel plate shear walls	8	2 ½	6 ½	NL
H. Special composite reinforced-concrete shear walls with steel elements	8	2 ½	6 ½	NL
I. Special reinforced-masonry shear walls	7	3	6 ½	NL
5. Dual Systems with Intermediate Moment Frames[c]				
A. Special steel concentrically braced frames	4 ½	2 ½	4	35[e]
B. Special reinforced-concrete shear walls	6	2 ½	5	160
C. Composite concentrically braced frames	5	2 ½	4 ½	160

Table 2-7. Design Coefficients and Factors for PC-1 and PC-2 LANL Basic Seismic-Force-Resisting Systems (Cont.)

Basic Seismic-Force-Resisting System	Response-Modification Coefficient, R	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Building Height Limitation (ft)[a,b]
6. Inverted Pendulum Systems				
A. Cantilevered column systems	2 ½	2	2 ½	35
B. Special steel moment frames	2 ½	2	2 ½	NL
C. Special reinforced-concrete moment frames	2 ½	2	1 ¼	NL

[a]NL – Not limited.
b See Section 1617.6.2.4.1 for buildings with 240 ft high or less for steel-braced frames and concrete cast-in-place shear walls.
c Steel intermediate moment resisting frames as part of a dual system are not permitted.
d Steel ordinary concentrically braced frames are permitted in penthouse structures and in single-story buildings up to a height of 60 ft when the dead load of the roof does not exceed 15 lb/ft^2.
e Steel ordinary moment frames and intermediate moment frames are permitted in a one-story building up to 60 ft high, when the moment joints of field connections are constructed of bolted end plates and the dead load of the roof does not exceed 15 lb/ft^2. The dead weight of the portion of the walls more than 35 ft above the base must not exceed 15 lb/ft^2.

Table 2-8. Seismic Detailing Reference Sections for Each of the Seismic-Force-Resisting Systems

	Seismic-Force-Resisting System	Detailing Reference Section IBC	ACI 530	AISC 341	ACI 318
1-A	Ordinary steel-braced frames in light-frame construction	2211			
1-D, 2-K	Light-framed walls with shear panels-wood structural panels/sheet steel panels	2306.4.1/2211			
1-E, 2-L	Light-framed walls with shear panels-all other materials	2306.4.5/2211			
3-G	Masonry wall frames	2106			
1-C, 2-J, 4-I	Special reinforced-masonry shear walls	2106.5	1.13.2.2.5/1.13.6		
1-F, 2-M	Special prestressed-masonry shear walls	2106.1.1.3/2106.5	1.13.2.2.5		
2-A, 4-A	Steel eccentrically braced frames, moment-resisting, connections at columns away form links			I.15	
2-B, 4-B	Steel eccentrically braced frames, non-moment-resisting, connections at columns away form links			I.15	
2-C, 4-C, 5-A	Special steel concentrically braced frames			I.13	
2-D	Ordinary steel concentrically braced frames			I.14	
2-F, 4-E	Composite eccentrically braced frames			II.14	
2-G, 4-F, 5-C	Composite concentrically braced frames			II.13	
2-H	Composite steel plate shear walls			II.17	
2-I, 4-H	Special composite reinforced-concrete shear walls with steel elements			II.16	
3-A, 6-B	Special steel moment frames			I.9	
3-B	Special steel truss moment frames			I.12	
3-C	Intermediate steel moment frames			I.10	
3-E	Special composite moment frames			II.9	
3-F	Composite partially restrained moment frames			II.8	
4-G	Composite steel plate shear walls			II.17	
1-B, 2-E, 4-D, 5-B	Special reinforced-concrete shear walls	1910.2.4/1910.5			21.1
3-D, 6-C	Special reinforced-concrete moment frames	1910.5			21.1

2.1.5.6 PC-3 and PC-4 Design Response Spectra

By DOE-STD-1020, the DBE response spectrum is developed from a probabilistic seismic hazard assessment of the site where mean annual exceedance probabilities for the DBE are specified for each performance category. The DBE is established through DOE-STD-1023.

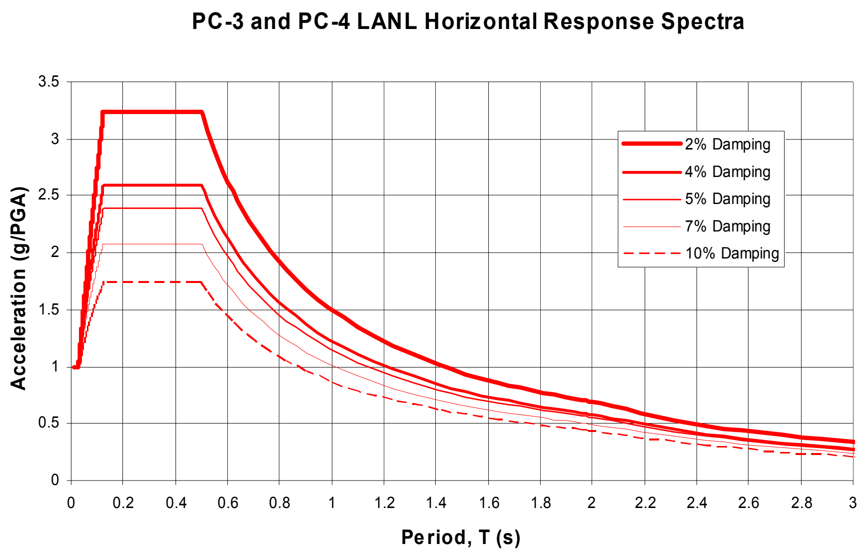
The latest probabilistic seismic hazard assessment performed for LANL was in 1995 (Wong et al., 1995). From this assessment, potential earthquake ground shaking was evaluated at all LANL Technical Areas. Because of the similarity of ground motion across LANL, a single DBE was developed for seismic design at all LANL sites. LANL site-specific DBE horizontal and vertical response spectra are shown in Figure 2-4 and Figure 2-5, respectively. These spectra must be scaled at all frequencies by the PGA at the mean annual exceedance probability for the performance category as shown in Table 2-9.

Table 2-9. LANL Site-Specific Probabilistic Accelerations*

Performance Category	Return Period (year)	Mean Annual Probability of Exceedance, P_H	Design Horizontal PGA (g)
PC-3	2,500	4×10^{-4}	0.34
PC-4	10,000	1×10^{-4}	0.58

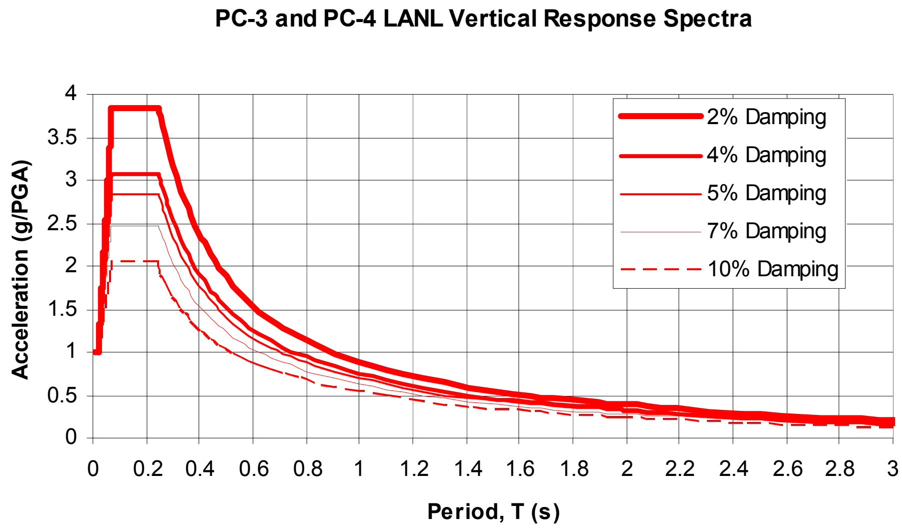
* After DOE-STD-10210-2002

Figure 2-4 and Figure 2-5 show the graphical representation of the horizontal- and vertical-design response spectra, respectively, for PC-3 and PC-4 SSCs.



Period (s)	2% Damping	4% Damping	5% Damping	7% Damping	10% Damping
0.010	1.00	1.00	1.00	1.00	1.00
0.028	1.00	1.00	1.00	1.00	1.00
0.125	3.23	2.59	2.39	2.08	1.75
0.500	3.23	2.59	2.39	2.08	1.75
2.000	0.69	0.58	0.55	0.49	0.44
3.000	0.33	0.28	0.27	0.25	0.22

Figure 2-4. DBE Horizontal Response Spectral Shape (scaled to PGA = 1 g).



Period (s)	2% Damping	4% Damping	5% Damping	7% Damping	10% Damping
0.010	1.00	1.00	1.00	1.00	1.00
0.020	1.00	1.00	1.00	1.00	1.00
0.071	3.83	3.07	2.83	2.46	2.08
0.250	3.83	3.07	2.83	2.46	2.08
0.400	2.38	1.91	1.76	1.53	1.29
1.000	0.89	0.75	0.70	0.63	0.56
2.000	0.39	0.33	0.31	0.28	0.25
3.000	0.21	0.17	0.16	0.14	0.13

Figure 2-5. DBE vertical response spectral shape (scaled to PGA = 1 g).

2.1.5.7 PC-3 and PC-4 Analysis Procedures

ASCE Standard XXX is intended to be used with ASCE 4 that provides criteria for the seismic analysis of safety-related nuclear SSCs. The seismic design and detailing of the components of the seismic-force-resisting system must comply with all the requirements for PC-1 and PC-2 SSCs. Design requirements must also comply with the following:

- For concrete structures: ACI-349, ACI-310, IBC 2003.
- For steel structures: AISC LRFD, AISC ASD, AISC LRFD/ASD, ANSI/AISC 341-02, ASCE 8-02, ASCE 19-96, AISC N690, and AISC N690L.
- For masonry structures: ACI-530-02.
- ASCE 7 for minimum nonseismic design loads for buildings and other structures. ASCE Standard XXX specifies the seismic load combinations.

PC-3 and PC-4 buildings must be evaluated by seismic dynamic analysis in accordance with the requirements of ASCE 4. Figure 2-6 shows the linear and nonlinear dynamic-analysis methods that can be used. The selection of the method will be determined in accordance with ASCE Standard XXX and ASCE 4.

DOE-STD-1020 allows performing linear-response spectrum dynamic analyses to evaluate the elastic seismic demand on SSCs. Inelastic energy absorption capability is allowed by permitting limited inelastic behavior. The inelastic energy absorption capacity of structures is accounted for by the parameter F_{μ}. Alternatively, nonlinear static analysis (pushover analysis) may be considered, and if necessary, a nonlinear dynamic analysis may also be used to get more accurate results.

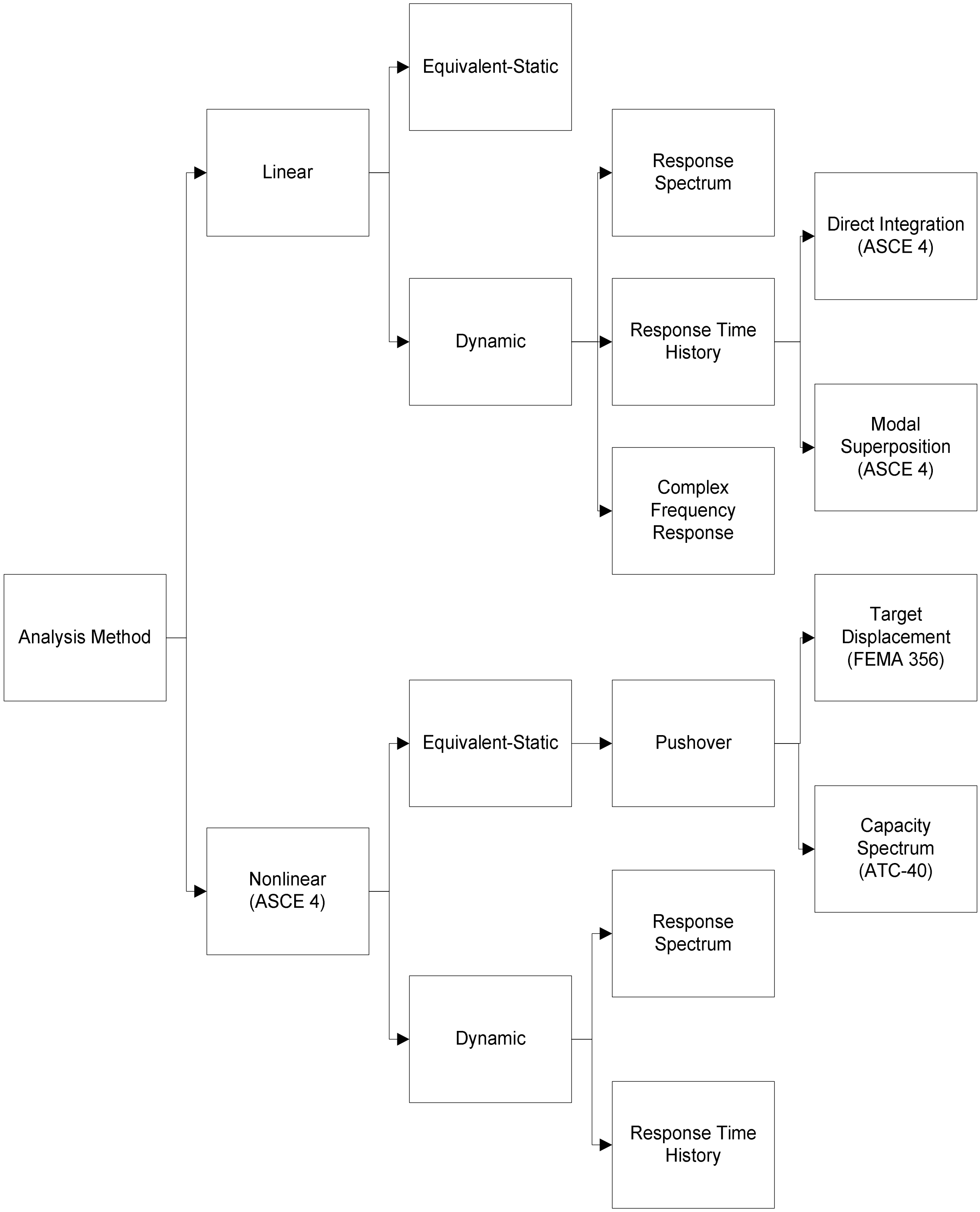


Figure 2-6. PC-3 and PC-4 Response Analysis Procedures.

2.1.5.8 PC-3 and PC-4 Seismic-Force-Resisting Systems

Some systems are not appropriate for use in nuclear facilities. These systems are unacceptable because of either large interstory drifts at high seismic demands or brittle failure mechanisms. Structural systems specifically prohibited include the following:

- Ordinary and Intermediate Moment-Resisting frame systems
- K-braced frames
- Plain concrete systems
- Precast concrete systems, which use gravity-only bearing connections
- Unreinforced masonry systems
- Wood Structures

All the systems listed in Section 2.1.5.4 that are not permitted for PC-1 and PC-2 SSCs are also not permitted for PC-3 and PC-4 SSCs. Acceptable structural systems are shown in Table 2-10.

Table 2-10. PC-3 and PC-4 LANL Permitted Basic Seismic-Force-Resisting Systems

Basic Seismic-Force-Resisting System
1. Bearing Wall Systems A. Ordinary steel-braced frames in light-frame construction B. Special reinforced-concrete shear walls C. Special reinforced-masonry shear walls
2. Building Frame Systems A. Steel eccentrically braced frames, moment-resisting, connections at columns away from links B. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links C. Special steel concentrically braced frames D. Ordinary steel concentrically braced frames E. Special reinforced-concrete shear walls F. Special reinforced-masonry shear walls
3. Moment-Resisting Frame Systems A. Special steel moment frames B. Special reinforced-concrete moment frames C. Masonry wall frames
4. Dual Systems with Special Moment Frames A. Steel eccentrically braced frames, moment resisting, connections at columns away from links B. Steel eccentrically braced frames, non-moment resisting, connections at columns away from links C. Special steel concentrically braced frames D. Special reinforced-concrete shear walls E. Special reinforced-masonry shear walls

2.1.6 Surface Fault Investigations in Los Alamos

Several previous studies have contributed to the understanding of the surface faulting hazard at LANL:

1. *Seismic hazard evaluations:*
   - the Dames and Moore (1972) seismic-hazard evaluation of the Plutonium Facility at TA-55
   - the Gardner and House (1987) preliminary 1984–1985 seismic hazard study of LANL
   - the 1991 to 1995 WCFS seismic-hazard evaluation of LANL

2. *Fault rupture hazard evaluations:*
    - the Woodward-Clyde Consultants (1991) evaluation of the potential for surface faulting at TA-55.
    - Olig et al., 1998, probabilistic surface rupture hazard at TA-3
    - Olig et al., 2001 probabilistic surface rupture hazard at TA-16
    - Kolbe et al. (1994) and Reneau et al. (1995) fault displacement hazard investigations at TA-67 for the proposed Mixed-Waste Disposal Facility.
    - Kolbe et al. (1995) fault displacement hazard investigation at TA-63 for the proposed Hazardous-Waste Treatment Facility and Radioactive Liquid-Waste Treatment Facility.

3. *Paleoseismic Investigations:*
    - Gardner et al. (1990) 1987 to 1988 paleoseismic investigations of the Guaje Mountain fault
    - Wong et al. (1995, 1996) paleoseismic investigation of the Pajarito, Rendija Canyon, and Guaje Mountain Faults.
    - Kelson et al. (1996) paleoseismic investigation of the Rendija Canyon fault.
    - Olig et al. (1996) paleoseismic investigation of the main Pajarito fault
    - McCalpin (1998, 1999) paleoseismic investigation of the main Pajarito fault
    - Reneau et al. (2002) paleoseismic investigation of the Pajarito fault zone.
    - Gardner et al. (2003) paleoseismic investigation of the Guaje Mountain fault zone.

4. *Mapping, drilling, trenching, structural, and geologic analysis for various technical areas:*

Reference	**Study Area**	**Geologic Mapping**	**Structural Analysis**	**Mineralogy**	**Geochemical Analysis**	**Trench Logging**	**Aerial Photograph Studies**	**Drilling**
Gardner et al., 1993	TA-55, TA-3, TA-16, TA-18							X
Carter and Gardner, 1993, 1995	RCD, GMF	X						
Reneau et al., 1995	TA-67	X	X	X	X	X		
McCalpin 1997	PF							
Reneau et al., 1998, Reneau and Vaniman 1998	TA-54	X	X					
Gardner et al., 1998	TA-48 and TA-55	X					X	
Gardner et al., 1999	Northwestern part of LANL, TA-3 to TA-55	X	X		X		X	X
Krier et al., 1998a	SCC and NISC facilities at TA-3	X			X			X
Krier et al., 1998b	CMR at TA-3	X			X			X
Gardner et al., 2001	TA-16	X	X		X			X
Lewis et al., 2002	Western part of LANL, TA-3 to TA-16	X	X		X	X		X
Lavine et al., 2003b	North-Central to Northeastern part of LANL	X	X	X	X			
Lavine et al., 2003a	LANL	X						

High-Precision Total Station Geologic Mapping

The total station is a theodolite coupled with a computer, which is used for a variety of geologic applications. Among others, the total station is used to generate geologic mapping of field camps (e.g., Wallace et al., 1996), log paleoseismic trenches, and topographic surveys. LANL has developed a new method of highly detailed geologic mapping with a total station to identify very small faults with the potential for surface rupture beneath sensitive facilities at LANL. The "high-precision geologic mapping" was started for the LANL seismic hazards program in 1996 (Lavine et al. 1997, 1998, 2003; Gardner et al., 1998, 1999, 2001), although some similar mapping was performed earlier at TA-54 (Reneau et al., 1995).

2.1.7 Surface Faulting

DOE-STD-1023 establishes that for sites containing only PC-1 and PC-2 SSCs, it is sufficient to use the information provided in the model building codes or national consensus standards. For sites containing facilities with PC-3 and PC-4 SSCs, site-specific characterization is required. It states that "*the potential for fault rupture and its associated tectonic surface deformation at the site must be evaluated. The amount and style of deformation and the likelihood of future displacement must also be characterized for any Quaternary (approximately last 2 million years) faults in close proximity to the site (within about 5 miles)*."

All Quaternary faults within a radius of 15 to 50 miles of a site should be assessed to determine if they are significant contributors to the seismic hazard of the site, and a detailed site characterization is necessary for active faults within a radius of 5 miles. The following factors should be addressed in the investigation: rate of fault movement, sense of slip (style of faulting), fault-dip and down-dip width, buried or blind faults, and fault segmentation. These factors will provide the basis for establishing the distance between the site and the earthquake source.

2.1.7.1 Setback Distance

DOE-STD-1022-94 specifies that sites with potential surface-fault rupture and associated deformation from active faults should be avoided. Sufficient data or detailed studies must be presented if surface deformation is not taken into account. DOE-STD-1022-94 refers to 40 CFR 264 for the minimum distance from active faults for hazardous waste treatment, storage, and disposal facilities. These facilities must not be located within 200 feet of a fault that has had displacement in Holocene time (the most recent time of the Quaternary period, 11,000 years) regardless of their performance category. DOE has not provided guidance on appropriate setback criteria for any other type of facility.

California (California Department of Conservation 2000) passed the Alquist-Priolo Earthquake Fault Zoning Act in 1972 to mitigate the hazard of surface faulting to structures for human occupancy. The Act prohibits the construction of buildings used for human occupancy on the surface trace of active faults and recommends that the buildings to be placed at a minimum setback distance from the fault of 50 ft. For the purposes of the Act, an active fault is one that has ruptured in the last 11,000 years (Holocene). This criterion was developed for near-vertical strike-slip faults. However, dipping normal faults have much broader zones of deformation that are often hundreds of feet wide and are asymmetric because of fault-scarp geometry, backtilting, and antithetic faulting on the downthrown side of the fault (McCalpin, 1987). Robinson (1993) specifies the following minimum setback distances for Utah and Juab Counties, Utah:

- 50 ft from the midpoint of a scarp that does not have a 30º slope
- 50 ft from the top and bottom slope break on a scarp that has 30º or more slope
- for scarps where a graben is present, 50 ft from the 30º slope break at the top and 50 ft from the farthest antithetic fault scarp.

To address these concerns related to normal faults, Utah (Salt Lake County Planning Division, 2002) provides recommendations for fault setbacks modified from McCalpin (1987) that depend on the slope of the fault scarps and whether backtilting or antithetic faults are present. Minimum setbacks are based on the type of proposed structure (Table 2-11). The setback is calculated as the greater of the setback given in Table 2-11 and that obtained using the following equations:

Downthrown Fault Block (Hanging Wall)

The fault setback, S, for the downthrown block will be calculated using the following formula:

$$S = U \cdot \left(2D + \frac{F}{\tan\theta} \right),$$

where:

S = Setback within which structures for human occupancy are not permitted,
U = Criticality Factor, based on the proposed occupancy of the structure (see Table 2-11)
D = Expected fault displacement per event (assumed to be equal to the net vertical displacement measured for each past event)
F = Maximum depth of footing or subgrade portion of the building
θ = Dip of the fault (degrees)
All units are in feet.

Upthrown Fault Block (Footwall)

The dip of the fault and depth of the subgrade of the portion of the structure are irrelevant; therefore, The setback is measured from the portion of the building closest to the fault, whether subgrade or above grade.

$$S = U \cdot 2D .$$

Table 2-11. Setback Recommendations and Critical Factors (U) for IBC Occupancy Classes (IBC 2003) (Salt Lake County Planning Division, 2002)

Class (IBC)	Occupancy Group	Criticality	U	Minimum Setback (feet)
A	Assembly	2	2.0	25
B	Business	2	2.0	20
E	Educational	1	3.0	50
F	Factory/Industrial	2	2.0	20
H	High hazard	1	3.0	50
I	Institutional	1	3.0	50
M	Mercantile	2	2.0	20
R	Residential (R-1, R-2, R-4)	2	2.0	20
R-3	Residential (R-3, includes Single Family Homes)	3	1.5	15
S	Storage	—	1.0	0
U	Utility and misc.	—	1.0	0

2.1.7.2 Surface-Fault Rupture and Associated Deformation

Because all the faults in Los Alamos were formed in the Quaternary period, all sites need to be evaluated to determine if there is a potential hazard because of surface faulting.
In accordance with DOE-STD-1022, the potential for fault rupture and associated tectonic surface deformation at the site must be evaluated. For any Quaternary fault within 5 miles from the site, the following information will be provided:

- The amount of deformation,
- The style of deformation, and
- The likelihood of future displacement.

Table 2-12 shows the surface-faulting events that were considered (Olig et al., 1996) in the site-specific probabilistic seismic assessment conducted by Wong et al. (1995). However, paleoseismic information was too limited with regard to the faulting events when compared to Table 2-13 that shows the most recent up-to-date data on surface faulting events. The information provided in 1995 was too limited to adequately characterize rupture behavior or earthquake recurrence along the Pajarito fault. Notice that the most recent event (MRE) for the Pajarito fault was dated "shortly before 50 to 60 ka," while in Table 2-13, the MRE occurred 2.2 to 1.4 cal ka (Trench 97-7). Based on the available data in 1995, recurrence intervals could not be reasonably constrained for the Pajarito fault. Data were inconclusive to determine whether the Rendija Canyon and Guaje Mountain faults had ruptured simultaneously with the Pajarito fault. Also, the possibility of both dependent and independent rupture behavior among the faults in the Pajarito fault system was addressed but not solved. Olig et al. (1996) reports that "additional paleoseismic and structural studies are needed to reduce uncertainties in rates of earthquake occurrence for the Pajarito fault system, and to simplify the modeling of expected rupture scenarios," (22 scenarios were considered). The following issues were said to be inconclusive:

1. "Do the Pajarito, Rendija Canyon and Guaje Mountain faults rupture dependently or independently of each other?
2. What is the relation of the Sawyer Canyon and Puye faults to the Pajarito Fault System?
3. How much have rupture patterns varied through time?
4. Have slip rates varied significantly through time, and in particular, have short-term rates been much higher than long-term rates, as observed elsewhere in the Rio Grande rift?"

Table 2-12. Most Recent and Penultimate Known Faulting Events on Faults within the Pajarito Fault System before 1995 (Olig et al., 1996)

Fault	MRE*	PE*
Pajarito Fault	Shortly before 50-60 ka	Shortly before MRE to ~63 ka > 57 ka
Rendija Canyon Fault	~ 9 ka or 19 to 27 ka	60 to 75 ka >140 ± 26 ka
Guaje Mountain Fault	4 to 6 ka	100 to 300 ka

*MRE = Most Recent Event, PE = Penultimate Event

The site-specific seismic hazard study was developed in 1995 and needs to be reviewed per DOE-STD-1023-95 about every 10 years. Recent paleoseismic studies have provided new information about the seismicity at LANL. The main issue of the Gardner et al. (2004) review is that at least three Holocene paleoseismic events have occurred in the Pajarito fault system. This number of Holocene seismic events suggests a higher rate of activity than the one incorporated in the probabilistic analysis of Wong et al. (1995), "where a minimum recurrence interval of 10,000 years for events in the Pajarito fault system was given a low weight and a recurrence interval of 20,000–40,000 years was given the highest weight."

Table 2-13. Up-to-date Most Recent and Penultimate Faulting Events on Faults within the Pajarito Fault System after 1995 (Gardner et al. 2004)

Locality/ Trench	Source	MRE*	PE*
97-7, -7A	1	1.4 cal ka or 1.8 to 1.4 cal ka or 2.2 to 1.4 cal ka	No evidence
97-3	1	(20 PDI ka or earlier) to 2.2 PDI ka	45 to 20 PDI ka
97-4	1	(19 PDI ka or earlier) to (2.1 PDI ka, 2.3 PDI ka, and 2.2 cal ka)	No evidence
98-4	2	21.3 cal ka to 10.0 PDI ka#	< 31 ^{14}C ka#
98-5	2	12.0 to 3.0 PDI ka#	44 to 11 PDI ka#
98-6	2	7.0 IRSL ka to 2.4 cal ka or 9.2 IRSL ka to 2.4 cal ka	24 PDI ka to (2.7 PDI ka or 7 IRSL ka)
EOC-2	3	8.6 to 5.5 cal ka or < 10.5 cal ka	(76 PDI ka or earlier) to 54 PDI ka
WETF-2C	4	7.3 to 1.3 cal ka	10.9 to 9.0 cal ka
GMF	5	6.5 to 4.2 cal ka (Cabra Canyon)	
	6	10 OSL ka to 3.4 cal ka (CHU-3)	~ 39 IRSL ka (CHU-1 and CHU-3)
	6	< 12.5 cal ka (CHU-2)	
RCF	7	> 8.1 cal ka	65 TL ka to 8.1 cal ka

*MRE = Most Recent faulting Events, PE = Penultimate faulting Events, as reviewed and reevaluated in Gardner et al. 2004.
#Most recent or penultimate mass wasting event, as reviewed and reevaluated in Gardner et al. 2004.
Acronyms: CHU = Chupaderos, EOC = Emergency Operations Center, GMF = Guaje Mountain Fault, ka = thousands of years before present, cal ka = thousands of calibrated radiocarbon years before present, IRSL = Infrared Stimulated Luminescence, OSL = Optically Stimulated Luminescence, PDI = Profile Development Index, RCF = Rendija Canyon Fault, TL = Thermoluminescence.
Sources: 1, McCalpin (1998); 2, McCalpin (1999); 3, Reneau et al. (2002); 4, Gardner et al. (2001); 5, Gardner et al. (1990); 6, Gardner et al. (in press); 7, Kelson et al. (1996)

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2.2 Volcanic Events

2.2.1 Valles Caldera Volcano

The one million-year-old Valles Caldera in Figure 2-7 is the centerpiece of the Jemez Volcanic Field in North Central New Mexico. The caldera was formed by collapse in response to eruption of over 300 km^3 of magmatic material. Subsequent resurgence of magma formed Redondo Peak, a structural dome, which is over 3,000 feet above the caldera floor, and a series of volcanic domes along the caldera's ring fracture system.

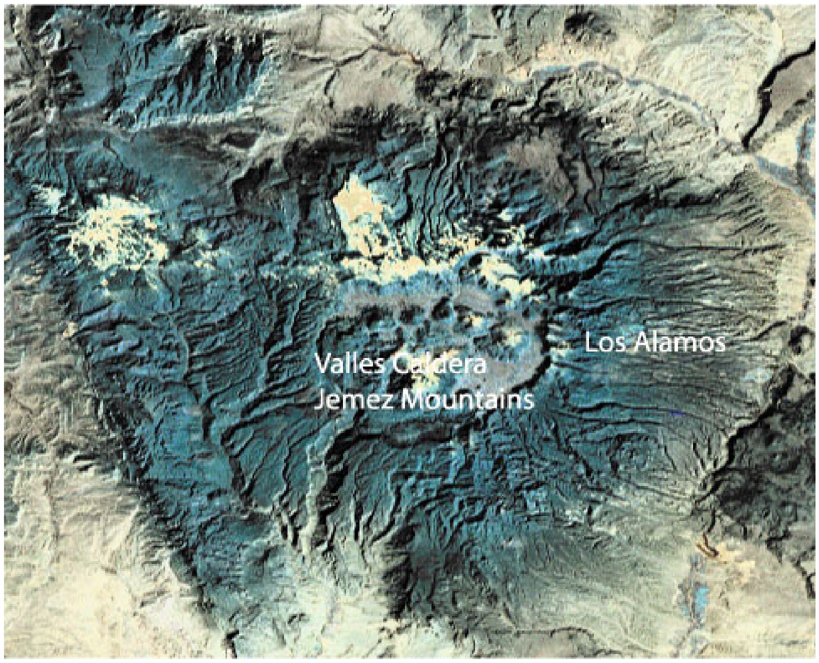

Figure 2-7. Valles Caldera. (Available from the USGS Geospatial Data Clearinghouse).

The Valles caldera of New Mexico is a resurgent caldera located in the midst of the Jemez volcanic field. With nearly 40 deep geothermal wells, which have resulted in extensive subsurface data, the Valles caldera is one of the best explored caldera complexes in the United States. It is the youngest of the two calderas in the region, having collapsed over and buried the Toledo caldera (which might have collapsed over older calderas).

2.2.2 Volcanic History of Valles Caldera

Eruption of the Otowi Member of the Bandelier Tuff at 1.6 Ma caused the formation of the Toledo caldera. Over the next roughly 400,000 years, smaller-volume eruptions continued in the Toledo caldera. Around 1.22 Ma, a major eruption of the Tshirege Member of the Bandelier Tuff occurred, resulting in the formation of the Valles caldera. Smaller eruptions between 1.2 and 0.52 Ma produced high-silica rhyolitic lavas and tephras. The youngest series of eruptions have been dated 60 Ka ± 15 Ka and produced the southwestern moat rhyolites. The SW moat rhyolites are made up of three members:

- the Battleship Rock Tuff (ash flow tuff), which mostly flowed down an ancestral San Diego Canyon,
- the El Cajete Pumice, which was dispersed over a wide region,
- and the Banco Bonito Rhyolite (lava flow), in the southwestern part of the Valles caldera.

The El Cajete Pumice was deposited in Los Alamos as ash and pumice, up to a few meters thick. It is much thicker in the Valles caldera.

Recent studies suggest that Valles caldera is entering a new cycle of activity, implying the potential volcanic hazard to the communities in and around the Jemez Mountains, including Los Alamos National Laboratory (Wolff and Gardner 1995, Reneau et al., 1996, and Steck et al., 1998).

2.2.3 Volcanic Hazards in Los Alamos

In accordance with DOE-STD-1022-94, the design of sites containing facilities with SSCs in Performance Category 1 and 2 can be investigated by following the procedures provided in model building codes or national consensus standards. For sites with SSCs in PC 3 and 4, site-specific NPH assessments have to be carried out. It states that "in regions where recent volcanic activity (Quaternary) has occurred, the likelihood of renewed volcanic activity and the associated potential hazards must be assessed."

The potential volcanic hazards include: lava flows, ballistic projections, tephra (ash) falls, pyroclastic flows and debris avalanches, lahars and flooding, seismic activity, ground deformation, tsunami, atmospheric effects, and acid rains and gases.

The Quaternary period goes back to about 1.8 Ma. Therefore, the Bandelier Tuff and the ensuing eruptions are also Quaternary. Because numerous eruptions have occurred in the Quaternary, a probabilistic volcanic hazard assessment has to be performed for Los Alamos for the design of PC-3 and -4 structures at LANL.

2.2.4 References

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2.3 *Tornadoes*

2.3.1 Fujita Scale Ranks Tornadoes by Damage

T. Theodore Fujita developed a wind damage scale to classify tornadoes. The F–for Fujita–scale uses numbers from 0 through 5. The ratings are based on the amount and type of wind damage. The ratings are

- **F-0. Light damage**. Wind up to 72 mph. Some damage to chimneys; breaks branches off trees; pushes over mustow-rooted trees; damages sign boards.
- **F-1. Moderate damage**. Wind 73 to 112 mph. The lower limit is the beginning of hurricane wind speed; peels surface off roofs; mobile homes pushed off foundations or overturned; moving autos pushed off the roads; attached garages may be destroyed.
- **F-2. Considerable damage**. Wind 113 to 157 mph. Roofs torn off frame houses; mobile homes demolished; boxcars pushed over; large trees snapped or uprooted; light object missiles generated.
- **F-3. Severe damage**. Wind 158 to 206 mph. Roof and some walls torn off well-constructed houses; trains overturned; most trees in forest uprooted.
- **F-4. Devastating damage**. Wind 207 to 260 mph. Well-constructed houses leveled; structures with weak foundations blown off some distance; cars thrown and large missiles generated.
- **F-5. Incredible damage**. Wind above 261 mph. Strong frame houses lifted off foundations and carried considerable distances to disintegrate; automobile sized missiles fly through the air in excess of 100 meters; trees debarked; steel-reinforced concrete structures badly damaged.

F-0 and F-1 tornadoes are considered “weak,” F-2 and F-3 are “strong,” and F-4 and F-5 are “violent.”

2.3.2 Tornadoes in Los Alamos

No tornadoes have ever been reported in Los Alamos County. However, funnel clouds have been observed in Los Alamos County. Fujita (1972) predicts a maximum wind speed of 200 mph should a tornado occur in the Los Alamos area. The design wind speed was obtained by adding a safety factor of 50 mph to 150 mph, the upper wind-speed range of an “F-2 tornado” that is possible, but unlikely, to occur in Los Alamos. The design tornado is estimated to have

- a maximum pressure drop of about 1.5 in. of mercury,
- a maximum pressure-change rate of 0.67 in./s, and
- a maximum rotational wind diameter of 100 ft.

Dust devils are swirls that go upward to fizzle out in clear air; they aren’t attached to clouds. Although they are most commonly found on deserts and form when air at the ground becomes much hotter than the air above. The lighter, hot air begins rising and takes on a whirling motion that carries dust and sand upward.

Dust devils are more likely to cause locally damaging winds in Los Alamos. Fujita (1972) states that dust devils could develop winds of up to 112 mph. Strong dust devils commonly produce 75-mph winds. On April 24, 1973, a strong dust devil knocked a trailer off its supports and rolled it one complete revolution at Los Alamos Meson Physics Facility, causing extensive damage (Bowen, 1990).

2.3.3 LANL Tornado Loads

DOE-STD-1020 has tabulated the recommended peak gust wind speeds for straight winds and tornadoes for several DOE facilities, including LANL. Table 2-14 summarizes the tornado loads for LANL. It can be observed that no tornado load design is necessary.

Table 2-14. LANL Basic Tornadoes Loads

Performance Category	Return Period (years)	Probability of Exceedance	Basic Wind Speed, V (mph)	Importance Factor
PC-1	N/A	N/A	N/A	N/A
PC-2	N/A	N/A	N/A	N/A
PC-3	50,000	20×10^{-6}	N/A	1.0
PC-4	500,000	2×10^{-6}	N/A	1.0

2.3.4 References

Bowen, B., "Los Alamos Climatology," Los Alamos National Laboratory report LA-11735-MS (May 1990).

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http://www.usatoday.com/weather/resources/basics/twist0.htm

2.4 Hurricanes

Hurricanes–called typhoons or tropical cyclones in some parts of the world–form over all of the world's tropical oceans except the South Atlantic and the Southeastern Pacific.

Los Alamos National Laboratory is located in New Mexico, which is a state with no coastal environment and 7,400 ft above sea level. Therefore, hurricanes do not occur here.

No specific hurricane load is required for the SSCs in LANL.

2.4.1 Reference

http://www.usatoday.com/weather/hurricane/when-where-hit.htm

2.5 *High Winds*

2.5.1 *Surface Winds*

The average surface winds at Los Alamos are 7 mph. The strongest winds occur in the storms and storms associated with cold fronts during the spring months. Sustained winds exceeding 25 mph and peak wind gusts exceeding 50 mph are common.

2.5.2 *Maximum wind gusts*

Wind gusts are common during the spring. According to Bowen, a maximum wind speed of 69 mph at TA-59 was recorded in March 6, 1986 (only 9-year period data were available and were measured at a 23-m height). A 77-mph wind gust was recorded from the south-southwest at East Gate on November 15, 1988. A maximum wind speed of 78 mph was recorded at the 92-m level at TA-50 on March 9, 1986.

2.5.3 *LANL Design Wind Loads*

DOE-STD-1020-02 recommends the use of peak gust wind speeds (3-second gust speed at 33 feet above the ground) for straight winds and Exposure Category C and importance factors for Los Alamos National Laboratory. Table 2-15 shows the return period, probability of exceedance, the basic wind speed, and importance factor for each of the Performance Categories.

Table 2-15. LANL Basic Wind Loads

Performance Category	Return Period (years)	Probability of Exceedance	Basic Wind Speed, V (mph)	Importance Factor
PC-1	50	200×10^{-4}	90	1.0
PC-2	100	100×10^{-4}	96	1.0
PC-3	1,000	10×10^{-4}	117	1.0
PC-4	10,000	1×10^{-4}	135	1.0

DOE-STD-1020-2002 establishes that all SSCs are to be designed following the most recent model building code, which is the IBC 2003. The IBC 2003 refers to ASCE 7-02, Chapter 6, for supplemental evaluation of the design wind loads. Nevertheless, this standard allows for PC-3 and PC-4 SSCs to reduce the load combinations given in ASCE 7-02 (LRFD or ASD) by 10 percent. Also, in combinations where the gravity load reduces the wind uplift, the 10% reduction is only applicable to the gravity load factor.

LANL Exposure Category

Using the conservative Exposure Category C according to ASCE 7-02, the potential sheltering from other adjacent structures and trees, and shelter from changes in the ground elevation can be neglected.

Topographic Effects

Topographic effects must be considered for SSCs located on mesas and close to the edge of canyons or escarpments.

2.5.4 *LANL Missile Load Criteria*

DOE-STD-1020-02 also takes into account the effect of objects or debris that could be carried by straight winds, hurricanes, or weak tornadoes. Table 2-16 shows the recommended missile specifications for PC-3 and PC-4 structures. PC-1 and PC-2 SSCs do not require considering missile criteria. The missile is a 15-lb 2×4 timber plank with 50-mph impact speed at a maximum height of 30 ft and 50 ft for PC-3 and PC-4, respectively.

Table 2-16. LANL Missile Criteria

Performance Category	Return Period (years)	Probability of Exceedance	Missile Criteria
PC-3	1,000	10×10^{-4}	2 × 4 timber plank 15 lb at 50 mph (horiz.); maximum height 30 ft
PC-4	10,000	1×10^{-4}	2 × 4 timber plank 15 lb at 50 mph (horiz.); maximum height 50 ft

This missile will

- Break annealed glass,
- Perforate sheet metal siding,
- Perforate wood siding up to ¾-in. thick, and
- Perforate form board.

When the missile passes through a window or a weak exterior wall, it can cause personnel injury and damage to interior contents of the building. DOE-STD-1020-02 also specifies the recommended straight wind missile barriers (Table 2-17) for PC-3 and PC-4 SSCs.

Table 2-17. Recommended Straight Wind Missile Barriers for PC-3 and PC-4 SSCs

Performance Category	Recommended Missile Barrier
PC-3	Concrete: 8-in. CMU wall with trussed horizontal joint reinforced at 16 in. on center Masonry: Single width brick veneer with stud wall.
PC-4	Concrete: 8-in. CMU wall with trussed horizontal joint reinforced at 16 in. on center Concrete: 4-in. concrete slab with #3 rebar at 6 in. on center each way in middle of slab.

2.5.5 Reference

Bowen, B., "Los Alamos Climatology," Los Alamos National Laboratory report LA-11735-MS (May 1990).

2.6 Floods

The flood design and evaluation criteria of SSCs must consider, according to DOE-STD-1020-02, two major events:

Regional flood hazards (i.e., river flooding)

New Mexico is characterized by not having large-scale floods. However, susceptible areas such as arroyos and canyons are prone to flash floods from heavy thunderstorms. Most of the facilities at LANL are located on top of the mesas. Hazards associated with river flooding can occur in facilities located in one of the following three main canyons at LANL:

- Pajarito Canyon,
- Los Alamos Canyon, and
- Water Canyon.

Local precipitation that affects roof design and site drainage.

All sites on top of the mesas and in the canyons must be designed for the effects of intense local precipitation that affects the roof design and the site drainage.

2.6.1 Post-Cerro Grande Fire

In May 2000, a prescription burn, started on Bandelier National Monument, blew out of control, and was designated as a wildfire. This wildfire burned around 7,650 acres within the boundaries of LANL, and severely burned the headwaters of many of the canyons that run through LANL. Because of the loss of vegetation and hydrophobic soils from steep canyon sides, surface runoff and soil erosion on hillsides above LANL were greatly increased over prefire levels. DOE/EA-1408 addressed the following emergency response actions to avoid the watershed conditions that resulted after this fire:

- A flood retention structure in Pajarito Canyon.
- A low-head weir and detention basin in Los Alamos Canyon.
- Reinforcements of the following four road crossings:
  - Embankment reinforcements in State Road 501 at Two-Mile Canyon,
  - Reinforcements in Pajarito Canyon and in Water Canyon, and
  - A land bridge Anchor Ranch Road in Two-Mile Canyon.
- A steel diversion wall upstream of TA-18 in Pajarito Canyon.

2.6.2 LANL Flood Loads

In accordance with DOE-STD-1020, buildings have to be designed for flood hazards according to their performance category (see Table 2-18).

Evaluation of the flood design basis for SSCs consists of the following:

- Determination of the Design-Basis Flood (DBFL) for each flood hazard (see Table 2-19) as defined by the hazard annual probability of exceedance and applicable combinations of flood hazards.
- Determination of the DBFL must be accomplished in accordance with DOE-STD-1023. The flood hazard assessment identifies the sources of flooding and the individual flood hazards. The DBFL for each flood hazard is defined in terms of:
- Peak-hazard level (e.g., flow rate, depth of water) corresponding to the mean hazard annual exceedance probability, including the combination of flood hazards.
- Corresponding loads associated with the DBFL peak-hazard level and applicable load combinations (e.g., hydrostatic and/or hydrodynamic forces, debris loads).

- Evaluation of the site stormwater management system.

The flood evaluation process is explained in detail in DOE-STD-1020-02. For new construction, the SSC should be constructed above the DBFL to eliminate the flood loads as part of the design, so flood hazards are not considered in the design basis (except for local precipitation). If this is not possible, DOE-STD-1020-02 lists alternate strategies to consider.

Design of civil engineering systems to the applicable DBFL and design requirements.

For PC-1 SSCs, the DBFL can be estimated from available flood hazard-assessment studies. For PC-2 through PC-4 SSCs, a comprehensive site-specific flood hazard assessment should be performed, unless the results of a screening analysis demonstrate that the performance goals are satisfied. The flood hazard assessment and flood screening analysis must be performed in accordance with DOE-STD-1023-95. The exterior walls of PC-3 and PC-4 of SSCs directly impacted by flood hazards should be constructed of reinforced concrete and designed according to the current ACI-349.

Guidelines for flood-resistant design and construction can be found in ASCE 24-98. ASCE 4-98 requires the design of structures within flood hazard areas to be governed by the loading provisions of ASCE 7-98.

Table 2-18. LANL Flood Criteria

Performance Category	**PC-1**	**PC-2**	**PC-3**	**PC-4**
Return Period (years)	500	2,000	10,000	100,000
Probability of Exceedance	200×10^{-5}	50×10^{-5}	10×10^{-5}	1×10^{-5}
Flood Hazard Input	Flood insurance studies or equivalent, including the combination in Table 2-19.	Site probabilistic hazard analysis, including the combinations in Table 2-19.		
Design Requirements	Governing local regulations and/or IBC 2003 for flood loads, roof design, and site drainage. Design of flood-mitigation systems (i.e., levees, dams, etc.) must comply with applicable standards.			

Table 2-19. LANL Design Basis Flood Events

Source of flooding[1]	**Case No.**	**Individual flood hazards***	**Sites**
River Flooding	1	Peak flood elevation.	N/A
	2	Wind-waves and Case 1.	
	3	Ice or debris forces (static and dynamic) and Case 1.	
	4	Peak and ground water level and Case 1.	
Levee/Dam Failure	1	Peak flood elevation as a result of all modes of failure (i.e., overtopping, seismically induced failure, random structural failures, upstream dam failure, debris or ice dam failure, etc.).	TA-2[2] TA-18[3] TA-41[2]
	2	Wind-waves and Case 1	
Local Precipitation	1	Flooding based on the site runoff analysis must be used to evaluate the site drainage system and flood loads on individual facilities.	All sites
	2	Ponding on roof to a maximum depth corresponding to the level of the secondary drainage system.	
	3	Rain and snow, as specified in applicable regulations.	
Snow	1	Snow and drift roof loads, as specified in applicable regulations.	All sites

*Events are added to the flood level produced by the primary hazard (source of flooding).
1Primary Hazard
2Los Alamos Reservoir is at this site in Los Alamos Canyon.
3The Flood Retention Structure is at this site in Pajarito Canyon.

2.6.3 References

American Nuclear Society, "Determining Design Basis Flooding at Power Reactor Sites," ANSI/ANS 2.8-1992, La Grange Park, Illinois, (1992).

Bowen, B., "Los Alamos Climatology," Los Alamos National Laboratory report LA-11735-MS (May 1990).

DOE/EA-1408, "Proposed Future Disposition of Certain Cerro Grande Fire Flood and Sediment Retention Structures at Los Alamos National Laboratory," Los Alamos, New Mexico (August 8, 2002).

2.7 Excessive Rains

In accordance with DOE-STD-1022, for sites containing PC-1 or PC-2 SSCs, it is sufficient to use the current building codes (IBC 2003 and ASCE 7). For PC-3 and PC-4 SSCs, an up-to-date site-specific probabilistic flood hazard analysis has to be performed following the guidance of DOE-STD-1023-95. Data must be collected, such as monthly and annual summaries (including averages and extremes) of precipitation at or near the site. Table 2-18 shows the return period years and probability of exceedance for each of the Performance Categories in accordance with DOE-STD-1022.

2.7.1 Rain in Los Alamos

The average annual precipitation (rainfall plus the water equivalent of frozen precipitation) is 18.7 in., with a standard deviation of 12.2 in. The lowest recorded annual precipitation is 6.8 in. and the highest is 30.3 in. The maximum precipitations recorded for a 24-hour and a 15-minute period are 3.5 in. and 0.9 in., respectively. The months with most precipitation are July and August (36% of the annual precipitation). This summertime precipitation is often known as the "monsoon" season. The extreme monthly average precipitation for the period of November 1910 through January 2004 is listed in Table 2-20.

Table 2-20. Extreme Monthly Average Precipitation Values for Los Alamos

Month	Precipitation (in.)	Year
January	6.75	1916
Feb	2.78	1987
Mar	4.11	1973
Apr	4.64	1915
May	4.47	1929
Jun	5.64	1986
Jul	7.93	1919
Aug	**11.18**	1952
Sep	5.79	1941
Oct	6.77	1957
Nov	6.60	1978
Dec	3.72	1918

The only available documented data showing precipitation frequency estimates for Los Alamos, New Mexico, is found in NOAA Atlas 14. Return periods range from 2 years to 1,000 years and for precipitation duration ranging from 5 minutes to 60 days (see Table 2-21). However, there is no information regarding larger return periods such as 2,000, 10,000, and 100,000 return-period years that correspond to PC-2, PC-3, and PC-4, respectively. A probabilistic precipitation hazard analysis should be performed to determine the precipitation frequency estimates for the return periods of 2,000, 10,000, and 100,000 years.

Table 2-21. Precipitation Frequency Estimates (in.) from NOAA Atlas 14, Volume 1, Version 3

Return Period (Year)	5 min	10 min	15 min	30 min	1 hr	2 hr	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	0.27	0.4	0.5	0.68	0.83	0.98	1.05	1.21	1.4	1.71	2.02	2.38	2.86	3.29	4.46	5.56	7.01	8.22
5	0.36	0.54	0.67	0.91	1.12	1.3	1.37	1.55	1.76	2.13	2.5	2.95	3.51	4.06	5.44	6.74	8.4	9.85
10	0.42	0.65	0.8	1.08	1.34	1.54	1.62	1.81	2.04	2.46	2.88	3.39	4.01	4.66	6.19	7.62	9.4	11.02
25	0.52	0.79	0.97	1.31	1.62	1.88	1.97	2.17	2.43	2.91	3.4	3.99	4.7	5.47	7.15	8.72	10.64	12.45
50	0.59	0.89	1.11	1.49	1.84	2.15	2.25	2.45	2.72	3.26	3.79	4.45	5.21	6.07	7.85	9.51	11.52	13.43
100	0.66	1	1.24	1.67	2.07	2.43	2.53	2.73	3.03	3.61	4.19	4.93	5.73	6.7	8.55	10.28	12.33	14.37
200	0.73	1.12	1.38	1.86	2.31	2.72	2.83	3.03	3.33	3.96	4.59	5.4	6.25	7.32	9.21	11.02	13.09	15.24
500	0.83	1.27	1.57	2.12	2.62	3.12	3.24	3.41	3.74	4.44	5.13	6.04	6.92	8.14	10.06	11.94	14.01	16.27
1,000	0.91	1.39	1.72	2.32	2.87	3.43	3.56	3.72	4.05	4.8	5.55	6.53	7.44	8.76	10.68	12.58	14.65	16.98

Figure 2-8 shows the Point Precipitation frequency estimates from NOAA Atlas 14 for Los Alamos, New Mexico.

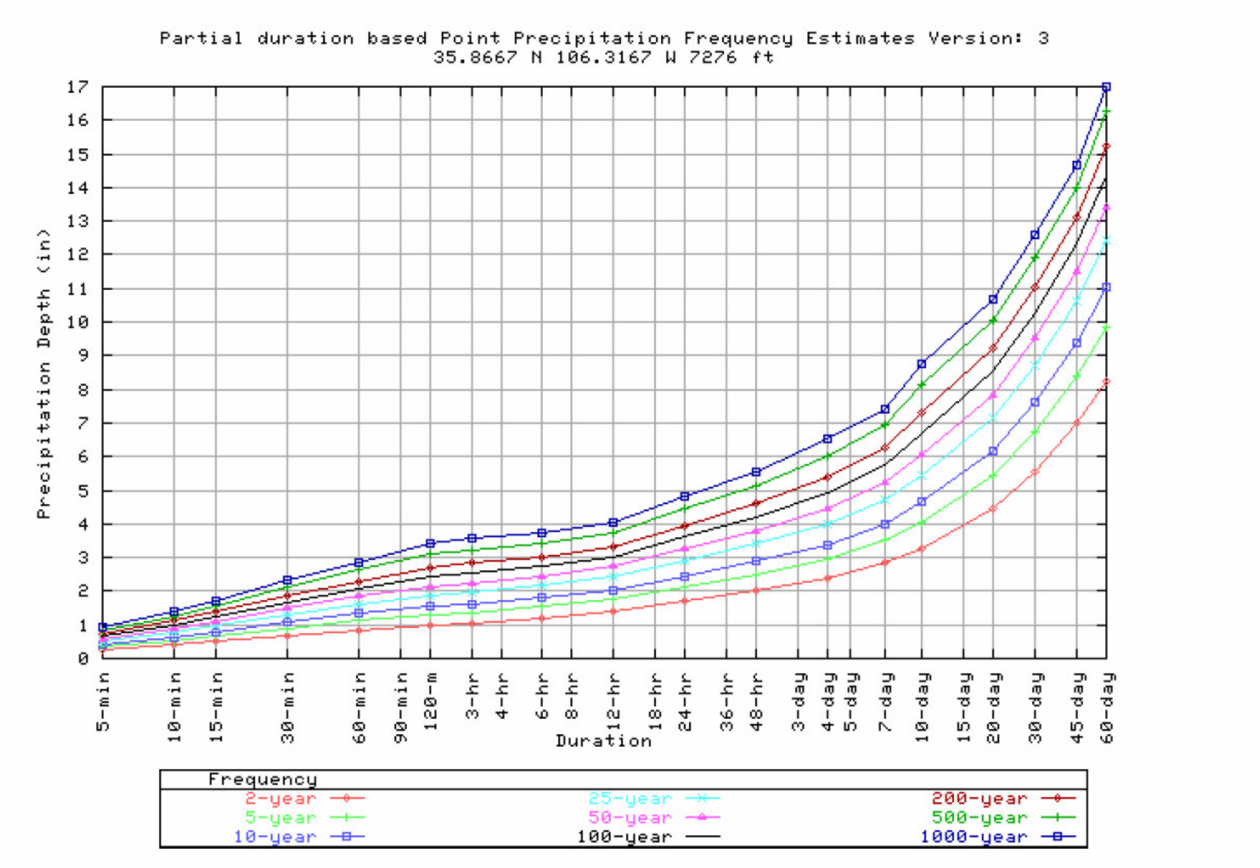

Figure 2-8. Precipitation Depth–Duration Estimates for Los Alamos, New Mexico from NOAA Atlas 14, Volume 1, Version 3.

2.7.2 LANL Rain Load (R)

Rain loading (R) used for roof load in structure design must be calculated using the procedure described in IBC 2003 (Section 1611) and ASCE 7 (Chapter 8) for PC-1 and PC-2 SSCs. Roof drainage systems are designed to handle all the flow associated with intense, short-duration rainfall events. The type and location of secondary drains and the hydraulic head above their inlets at the design flow must be known in order to determine rain loads.

The design rain load is the amount of water that could accumulate on a roof from blockage of the primary drainage system. The roof is designed to withstand the load created by that water plus the uniform load caused by water that rises above the inlet of the secondary drainage systems at its design flow. The rain load is given by

$$R = 5.2(d_s + d_h),$$

where:

R = rain load on the undeflected roof (lb/ft^2).

d_s = depth of water on the undeflected roof up to the inlet of the secondary drainage system when the primary drainage system is blocked (static head), (in.).

d_h = additional depth of water on the undeflected roof above the inlet of the secondary drainage system at its design flow (hydraulic head), (in.).

Rain loads for PC-3 and PC-4 SSCs have to be defined with a site-specific rain intensity hazard analysis. There is no documented up-to-date basis.

Note that the design rain loads will affect the design only if they are larger than the design snow loads and the roof live loads.

2.7.3 Ponding

Ponding instability must be investigated in accordance with ASCE 7 in roofs with a slope of less than ¼ in./ft (1.19º) and in roofs equipped with hardware to control the rate of drainage.

2.7.4 Site Drainage and Roof Design

DOE-STD-1020-02 specifies that applicable local regulations must be considered in the design of the site stormwater management system. The minimum design level for the stormwater system is a 25-year, 6-hour storm. From Table 2-21, the minimum design level for LANL is 2.17 in. Once the site and facility drainage design has been developed, it should be evaluated for the DBFL precipitation for each SSC if the SSC is built below the DBFL. Section 2.6 contains more information about the DBFL.

IBC 2003 refers to the International Plumbing Code for the design and installation of the roof drainage.

2.7.5 References

Bonnin, G. M., D. Todd, B. Lin, T. Parzybok, M. Yekta, and D. Riley, "Precipitation-Frequency Atlas of the United States," *NOAA Atlas 14*, Volume 1, Version 3, National Weather Service, Silver Spring, Maryland, http://hdsc.nws.noaa.gov/hdsc/pfds/sa/nm_pfds.html, (2003).

Bowen, B., "Los Alamos Climatology," Los Alamos National Laboratory report LA-11735-MS (May 1990).

International Code Council, *International Plumbing Code* (2000).

http://weather.lanl.gov/, The Weather Machine

2.8 Excessive Snow

In accordance with DOE-STD-1022, for sites containing PC-1 or PC-2 SSCs, it is sufficient to use the current building codes (IBC 2003). For PC 3 and 4 SSCs, an up-to-date site-specific probabilistic snow hazard analysis has to be performed following the guidance of DOE-STD-1023-95. Data must be collected, such as monthly and annual summaries (including averages and extremes) of snow fall at or near the site.

2.8.1 Snowfall in Los Alamos

According to Bowen, the record snowfall occurred in the winter of 1986-1987 with 153 in. It represents a return period of nearly 65 years. The maximum monthly snowfall is 65 in. (January 1987) and represents a return period of nearly 140 years. The highest recorded snowfall for a 24-hour period is 22 in.

Extreme snow depth value is 42 in. (January 1987) which represents a 120-year return period. This record snow depth resulted from the 48 in. of snow that fell during January 15–17, 1987 (the snow depth is less than the snow fall because of compaction and settling during and after a snowfall). The snow is generally dry; on average 20 units of snow are equivalent to 1 unit of water.

The extreme monthly average precipitation for the period of November 1910 through January 2004 is listed in Table 2-22.

Table 2-22. Extreme Monthly Average Snow for Los Alamos

Month	Snow (in.)	Year
January	**64.8**	1987
Feb	48.5	1987
Mar	37.0	1973
Apr	33.6	1958
May	17.0	1917
Jun	0.0	2003
Jul	0.2	1925
Aug	0.4	1957
Sep	4.0	1936
Oct	21.2	1996
Nov	34.5	1957
Dec	41.3	1967

2.8.2 LANL Ground Snow Load

Snow loading (S) used for roof load in structure design must be calculated using the procedure described in Chapter 7 of ASCE 7. The ground snow loads shown in Table 2-23 were determined from a statistical study of 77 years of LANL site-specific data. The site-specific ground snow load (p_g) study was developed in 1996–97 by “A” Division and needs to be recalculated for new data per DOE-STD-1023-95 on a 10-year cycle. Table 2-23 also shows the Importance Factor, I, to be used in accordance with DOE-STD-1020-02.

Table 2-23. LANL Ground Snow Loads

Performance Category	Return Period (yr)	Probability of Exceedance	Ground Snow Load, p_g (psf)	Importance Factor, I
PC-1	50	2×10^{-2}	16	1.0
PC-2	100	1×10^{-2}	19	1.0
PC-3	1,000	1×10^{-3}	29	1.2
PC-4	10,000	1×10^{-4}	41	1.2

Ground snow load data provided by ASCE 7 should not be used. The three service locations at which load measurements were made correspond to Albuquerque, Clayton, and Roswell. The three of them have lower snow accumulations than those observed in Los Alamos.
Unbalanced accumulation of snow at valleys, parapets, roof structures, and offset in roofs of uneven configuration (drifts) must also be considered in accordance with ASCE 7. The snow loading importance factor (I) is 1.0 for PC-1 and PC-2 SSCs and equal to 1.2 for PC-3 and PC-4 SSCs to be consistent with DOE-STD-1020 snow load provisions.

2.8.3 Rain-on-Snow Surcharge Load

The rain-on-snow surcharge loads will be considered only for PC-1 and PC-2 low-slope roof SSCs with a roof slope α less than ½ in./ft (2.38º). Table 2-24 shows the required additional rain-on-snow surcharge load required for PC-1 and PC-2 SSCs as a function of the exposure factor C_e, and the thermal factor C_t defined in ASCE 7-02.

Table 2-24. Rain-on-Snow Surcharge Loads for PC-1 and PC-2 SSCs

PC	$C_e = 0.9$			$C_e = 1.0$			$C_e = 1.1$		
	$C_t = 1.0$	$C_t = 1.1$	$C_t = 1.2$	$C_t = 1.0$	$C_t = 1.1$	$C_t = 1.2$	$C_t = 1.0$	$C_t = 1.1$	$C_t = 1.2$
1	0.00	0.09	1.10	0.20	1.32	2.44	1.32	2.55	3.78
2	0.00	0.00	0.36	0.00	0.63	1.96	0.63	2.09	3.56

Rain-on-snow surcharge loads will not be considered for PC-3 and PC-4 SSCs. In accordance with ASCE 7-02 Section C7.10, "where p_g is greater than 20 psf, it is assumed that the full rain-on-snow effect has been measured and a separate rain-on-snow surcharge is not needed."

2.8.4 Reference

Bowen, B., "Los Alamos Climatology," Los Alamos National Laboratory report LA-11735-MS, (May 1990).

http://weather.lanl.gov

2.9 Ice Cover

According to Bowen (1990), ice storms do not occur in Los Alamos. Melted snow and rain may freeze on roadways and sidewalks, but large accumulations on utility lines do not occur. Rime icing occurs when fog droplets come in contact with objects, such as roads and utility lines and occurs more frequently in low spots where cold air settles.

The following SSCs may be prone to this ice load:

- *Components and appurtenances* such as ladders, handrails, antennas, waveguides, radio frequency transmission lines, pipes, electrical conduits, and cable trays.
- *Ice-sensitive structures* such as lattice structures, guyed masts, overhead lines, light suspension and cable-stayed bridges, aerial cable systems, open catwalks and platforms, flagpoles, and signs.

2.9.1 Load Combinations

Minimum ice loads will comply with ASCE 7-02, Section 10.0. The ice thickness and the concurrent wind speed are determined based on a 50-year mean recurrence interval. A site-specific study for mountainous terrain and gorges must be performed if unusual icing conditions may exist. Site-specific studies must be used to determine the 50-year mean recurrence–interval ice thickness and concurrent wind speed.

When a structure is subjected to atmospheric ice and wind-on-ice loads, the following combinations must be considered for strength design:

1. $1.2(D+F+T) + 1.6(L-H) + 0.2D_i + 0.5S$
2. $1.2D + L + D_i + W_i + 0.5S$
3. $0.9D + D_i + W_i + 1.6H$,

where:
D = dead load
F = load due to fluids with well-defined pressures and maximum heights
T = shelf-straining force
L = live load
H = load resulting from lateral earth pressure, ground water pressure, or pressure of bulk materials
S = snow load
D_i = weight of ice
W_i = wind-on-ice

2.9.2 Design Ice Thickness for Freezing Rain (t_d)

The design ice thickness, t_d, is calculated as

$$t_d = 2.0tI_i f_z (K_{zt})^{0.35},$$

where:
t = nominal ice thickness (0.25 in. for Los Alamos)
K_{zt} = topographic factor obtained from Eq. 6-3 in Section 6.5.7.2 of ASCE 7.
I_i = importance factor. Table 2-25 shows the return period, probability of exceedance, and importance factor for each of the Performance Categories. The mean return periods have been chosen similar to DOE-STD-1020 wind-load return periods. The importance factors have been obtained from ASCE 7-02, Table C10-1. Data for the 10,000 year return period were not available in ASCE 7-02.

Table 2-25. Importance Factors for Ice Loads

Performance Category	Mean Return Period (years)	Probability of Exceedance	Importance Factor, I_i
PC-1	50	200×10^{-4}	1
PC-2	100	100×10^{-4}	1.25
PC-3	1,000	10×10^{-4}	2.3
PC-4	10,000	1×10^{-4}	Not known

f_z = factor that depends on the height above ground, z (in ft):

$$f_z = \begin{cases} \left(\dfrac{z}{33}\right)^{0.10} & for \quad 0\,ft < z \leq 900\,ft \\ 1.4 & for \quad z > 900\,ft \end{cases}$$

2.9.3 Weight of Ice (D_i)

The ice weight is a function of the design ice thickness, t_d, and must be determined in accordance with ASCE 7, Section 10.4, using the weight of glaze ice formed on all exposed surfaces of structural members, guys, components, appurtenances, and cable systems.

2.9.4 Wind on Ice-covered Structure (W_i)

Ice increases the projected area of the SSCs exposed to wind. The projected area is increased by adding t_d to all free edges of the projected area. Wind loads must be calculated in accordance with ASCE 7, Section 6. The concurrent wind speed V is 40 mph and the importance factor I = 1.0 for all Performance Categories.

2.9.5 Thunderstorms, Lightning, and Hail in Los Alamos

Thunderstorms are very common in Los Alamos. The average number of thunderstorms per year is 61, with most in July and August (the monsoon season) the months with most thunderstorm days (Bowen 1990).

Lightning in Los Alamos can be frequent and intense during thunderstorms. Because it can cause occasional brief power outages, lightning protection is an important design factor for most of the facilities at the Laboratory.

Hail is also very common at Los Alamos. The area around Los Alamos has the most frequent hailstorms in New Mexico (NOAA 1977). The diameter of the hailstones is about 0.25 in. Infrequently, hailstorms cause significant damage to property. A thunderstorm on August 11, 1982, dropped about 3 in. of hail near the Los Alamos airport, damaging windshields and vegetation in the area. On May 9, 1989, hailstones were up to 1 in. in diameter in White Rock, causing damage to cars, roofs, and vegetation and left 2 in. of hail in Los Alamos.

2.9.6 LANL Lightning Protection

DOE-G 420.1-1 specifies that "lightning protection systems must be considered for buildings and structures that contain, process, and store radioactive, explosive, and similarly hazardous materials. The lightning protection systems must by designed to comply with NFPA 780." Therefore, all explosive facilities, all facilities with a replacement value of $1 million or more (structure and equipment), and facilities of significant programmatic importance must be equipped with lightning protection.

The Engineering Standards Manual Chapter 7 Electrical, Section D5000 "General Electric Requirements" contains a subsection (5.6) that covers the lightning protection system in accordance with NFPA 780, IEEE Std 1100, IEEE C62 Surge Protection Standards Collection, and UL96A.

The lightning-protection system design also has to comply with LANL Construction Specification Section 16670 and DOE-M-440.1-1.

2.9.7 LANL Hail Protection

Modified bituminous membrane roofing will comply with the Impact Resistant test in accordance with ASTM D3746 (LANL Construction Specification Section 07550). Elastomeric membrane roofing will comply with the Puncture Resistance test in accordance with FTM 191B, Method 2031 (LANL Construction Specification Section 07531).

2.9.8 References

http://www.lanl.gov/f6stds/pubf6stds/engrman/7elec/htmls/elecnew2.htm, LANL ESM Chapter 7, Electrical Manual

http://weather.lanl.gov/html/climatology.html

http://www.lanl.gov/f6stds/pubf6stds/conspec/pdfs/pdf_history/16670-R2.pdf LANL Construction Specification Section 16670 on Lightning Protection.

Bowen, B., "Los Alamos Climatology," Los Alamos National Laboratory report LA-11735-MS, (May 1990).

IEEE C62, *Surge Protection Standards Collection* (2002).

IEEE Std 1100, *Recommended Practice for Powering and Grounding Electronic Equipment* (1999).

NFPA 780, *Standard for the Installation of Lightning Protection Systems*, 2000 Edition.

NOAA (National Oceanic and Atmospheric Administration, Climate of New Mexico, *Climatography of the United States No. 60*, (National Climatic Center, Ashville, North Carolina, March 1977).

UL 96A, *Installation Requirements for Lightning Protection Systems*, Eleventh Edition (July 2001).

2.10 Forest Fires

2.10.1 Background

About 5,200 historic fires have been mapped in the Jemez Mountains for the period 1909–1996 (USGS) from administrative records of local land-management agencies. Records show that lightning caused 75% of these recorded fires. Since 1954, there have been five major fires that burned in the Los Alamos National Laboratory Area:

- 1954: The Water Canyon fire.
- 1977: The La Mesa fire.

In June of 1977, the La Mesa fire burned 15,270 acres in and around Frijoles Canyon, Bandelier National Monument, and the adjacent Santa Fe National Forest.

- 1996: The Dome fire.

The Dome fire occurred in April of 1996 in Bandelier National Monument, and burned 16,516 acres in Capuling Canyon and the surrounding Dome Wilderness area.

- 1998: The Oso fire.

The Oso fire occurred in late June and early July of 1998 and burned more than 5,000 acres north of the town. Figure 2-9 shows the smoke of the fire while looking north from the Otowi Building.

Figure 2-9. Smoke From the Oso Complex Fire Can Be Seen as You Look North From the Otowi Building (Photo by Ed Vigil).

- 2000: The Cerro Grande fire.

Started as a prescribed burn on May 4, this fire escaped control and was declared a wildfire on May 5. It was not contained until June 6. It destroyed some 260 homes and did an estimated $300 million worth of damage to the Los Alamos National Laboratory. The Cerro Grande fire burned about 47,650 acres in Bandelier National Monument, the Santa Fe National Forest, Santa Clara Pueblo, San Ildefonso Pueblo, 7,500 acres in Laboratory property, and the City of Los Alamos.

One of the biggest concerns in the case of Los Alamos was the possible release of radioactive materials–a worry that proved mostly unnecessary. More of a problem was potential flooding from stormwater runoff on the now-bare slopes, or from landslides where soil-stabilizing plant cover had been destroyed. Other concerns include the following:

- Erosion and landslides (which might include mud, debris, or rolling rocks) in areas where vegetation that stabilized the land had been destroyed by fire.
- Flooding of streams receiving large amounts of runoff from the fire areas. Vegetation and litter that once slowed stormwater runoff are often destroyed by fire (see Figure 2-10).
- Water quality in streams receiving runoff from fire areas. The runoff may carry extra sediment and ash, which can kill fish by robbing streams of oxygen.
- Mobilization of other special hazards or materials. The biggest concern at Los Alamos was the possible release of radioactivity (it turned out to be minimal). But chemical wastes, and even natural asbestos fibers, were also concerns. Such hazardous substances might move in the air (as dust) or water after a fire.

Figure 2-10. TA-46 Building 2 After the Cerro Grande Fire, 2000.

2.10.2 LANL Fire Protection System

LANL Engineering Standards Manual, Chapter 2–Fire Protection specifies the requirements and guidance that apply to all existing and new LANL facilities, designs for new construction, and for modifications to existing buildings and structures. This manual refers to the National Fire Protection Association 101 and the IBC. Design criteria must follow DOE-STD-1066-99, Fire Protection Design Criteria, and DOE-M-440.1-1 for the special requirements for protection of explosive facilities from wild land fire exposure. The minimum fire resistance rating for LANL facilities must be IBC Type II-B or NFPA 220 Type 11(000).

Section 7.0 of LANL ESM Chapter 2 refers to NFPA 299, Standard for Protection of Life and Property from Wildfire for the evaluation of the degree of wild-land fire hazard for a particular facility, the Urban Wildland Interface Code (ICBO Item No. UWIS2K), and LANL Fire Protection Group for guidance.

2.10.3 References

http://www.lanl.gov/f6stds/pubf6stds/engrman/2fire/htmls/firepro2.htm, LANL Engineering Standards Manual, Chapter 2–Fire Protection (2002).

NFPA 101, *Life Safety Code Handbook* (2003).

NFPA 80A, *Recommended Practice for Protection of Buildings from Exterior Fire Exposures*.

NFPA 220, *Standard on Types of Building Construction* (1999).

NFPA 299, *Standard for Protection of Life and Property from Wildfire* (1997).

ICBO, *Urban Wildland Interface Code*, First printing, Item No. UWIS2K (2000).

3 Secondary Natural Phenomena Hazards

3.1 Drought

Extended periods of dryness are uncommon in Los Alamos, largely because of the reliable summer thundershowers. The worst drought occurred in 1956 when only 6.8 in. of precipitation fell during the whole year (Bowen, 1990).

Geotechnical reports must determine the ground characteristics to determine if foundation settlement may be an issue for the design of the SSCs.

Follow ASCE 7, Section 5.2, for foundation designs on expansive soils.

3.1.1 References

Bowen, B. M., "Los Alamos Climatology," Los Alamos National Laboratory report LA-11735-MS (May 1990).

Bowen, B. M., "Los Alamos Climatology Summary," Los Alamos National Laboratory report LA-12232-MS (1992).

3.2 Fog

3.2.1 Fog at Los Alamos

Generally fog seldom occurs in Los Alamos. The greatest number of fog days occurs during December when the nights are longest. The month of December has a monthly mean of 1.6 fog days over a 27-year period ending in 1988. Fog most often occurs on clear nights following snow or rain. The other cold months average less than 1 monthly fog day. August has no fog days reported (Bowen 1990).

Fog is not considered a structural hazard for any of the SSCs at LANL.

3.2.2 Reference

Bowen, B. M., "Los Alamos Climatology," Los Alamos National Laboratory report LA-11735-MS (May 1990).

3.3 *Frost*

Frost is ice crystals formed by water vapor deposition on a surface at temperatures of 32ºF or below.

3.3.1 Frost Protection

A) Footings and foundations of all SSCs must follow IBC 2003, Section 1805.2.1, for frost protection. One or more of the following methods can be used for frost protection:

- *Extending below the frost line of the locality.*

Extreme-value statistics from frost-penetration depths in the United States have been evaluated and mapped by the U.S. Department of Housing and Urban Development (July 2001). These frost depth (penetration) maps have been developed for various return periods (2, 5, 10, 25, 50, and 100 years) and were based on maximum annual frost depth under bare soil, sod using observed snow cover conditions, and snow-free bare soil conditions (see Table 3-1). Appendix B of the report mentioned above shows the maps of the 2-, 5-, 10-, 25-, 50-, and 100-year return periods for maximum annual frost depth under bare soil, sod using observed snow-cover conditions, and for snow-free bare soil conditions. The soils used to depict those maps have a clay content of 10%, a field capacity of 30%, and a porosity of 45%.

Table 3-1. Soil Freezing Depth (in.) for 2-, 5-, 10-, 50-, and 100-Year Return Periods

	2 years	**5 years**	**10 years**	**25 years**	**50 years**	**100 years**
Bare soil with snow	10	18	24	30	32	36
Bare soil with no snow	24	30	32	41	43	50

Los Alamos County requires that all foundations have a 36-in. minimum depth to maintain structural integrity. Therefore, the frost depth line is set to 36 in. for building foundations.

- *Constructing in accordance with ASCE-32*.

A frost protected mustow foundation (FPSF) is a practical alternative to deeper, more-costly foundations in regions with cold climates and seasonal ground freezing. The Standard ASCE-32 addresses the design and construction of frost-protected mustow foundations to prevent frost damage. An FPSF refers to a foundation that does not extend below the design frost depth but is protected against the effects of frost. The minimum footing depth for Los Alamos is 12 in., and no horizontal insulation is required if FPSF are used.

- *Erection on solid rock.*

B) Refer also to IBC 2003 Section 1905.12 for concrete materials and Section 2104.3 for masonry materials under frost environmental conditions during construction.

3.3.2 References

Bowen, B. M., “Los Alamos Climatology,” Los Alamos National Laboratory report LA-11735-MS (May 1990).

U.S. Department of Housing and Urban Development, *Development of Frost Depth Maps for the United States*, Maryland (July 2001).

3.4 High Temperatures

Extreme heat in Los Alamos is very rare. Los Alamos averages only 2 days per year with temperatures higher than 90ºF. The summer of 1980 was the hottest with 22 times higher than 90ºF. The highest temperature recorded in Los Alamos was 95ºF in 1935, 1981 and 1998 (Bowen 1990).

High temperatures in SSCs are considered to be a secondary natural hazard when caused by the following:

- *An exterior fire* (as a result of an earthquake, a volcanic explosion, lightning, or a forest fire), or
- *An internal fire* (explosion of gas lines, explosives, flammable and combustible materials, hazardous equipment, etc.)

3.4.1 LANL High-Temperature Protection

A) LANL Engineering Standards Manual, Chapter 2–Fire Protection specifies the requirements and guidance that apply to all existing and new LANL facilities, designs for new construction, and for modifications to existing buildings and structures. Section 7 of this Chapter refers to the exposure and NPH protection.

This manual refers to the National Fire Protection Association (NFPA) 101 and the IBC. Design criteria must also follow DOE-STD-1066-99, Fire Protection Design Criteria, DOE-M-440.1-1 for explosive facilities, and DOE-STD-1088-95 for Relocatable Structures. The minimum fire-resistance rating for LANL facilities must be IBC Type II-B or NFPA 220 Type 11(000).

Section 7.0 of LANL ESM, Chapter 2, refers to NFPA 80A for general external fire exposures, NFPA 299 and the Urban Wildland Interface Code (ICBO) for protection from wildfire exposure, and the LANL Fire Protection Group for guidance. This section also refers to NFPA 30, 70, and 37 as guidance for internal fire-exposure protection in the facilities. Section 14.0 lists special fire/explosion hazards, such as warehousing, gloveboxes and filter plenums, flammable and combustible materials, explosive materials, paint spraying/coating, and tank liquids storage and their corresponding DOE and NFPA guidelines.

A Fire Hazard Analysis must be performed as required to support the ESM.

B) Refer also to IBC 2003, Section 1905.13, for concrete materials and Section 2104.4 for masonry materials under high-temperature environmental conditions during construction.

3.4.2 References

Bowen, B. M., "Los Alamos Climatology," Los Alamos National Laboratory report LA-11735-MS (May 1990).

http://www.lanl.gov/f6stds/pubf6stds/engrman/2fire/htmls/firepro2.htm, LANL Engineering Standards Manual, Chapter 2–Fire Protection (2002).

ICBO, *Urban Wildland Interface Code, First printing*, Item No. UWIS2K (2000).

NFPA 101, *Life Safety Code Handbook* (2003).

NFPA 30, *Flammable and Combustible Liquid Code* (2003).

NFPA 37, *Standard for the Installation and Use of Stationary Combustion Engines and Gas Turbines*, (2002).

NFPA 70, *National Electric Code* (2002).

NFPA 80A, *Recommended Practice for Protection of Buildings from Exterior Fire Exposures*

NFPA 299, *Standard for Protection of Life and Property from Wildfire* (1997).

NFPA FPH, *Fire Protection Handbook*, Section 3-7, Fire Hazard Analysis, Nineteenth Edition (2003).

3.5 Low Temperatures

Temperature in Los Alamos drops to 0ºF or below only once or twice a year. The coldest day was -18ºF in 1963. The lowest monthly temperatures occur in January with a minimum, average, and maximum of 10.9ºF, 20.9ºF, and 29ºF, respectively (Bowen 1990).

3.5.1 Freeze-Thaw Data

Freeze-thaw days are defined as days in which the maximum temperature is above 32ºF and the lower temperature is 32ºF or below. The temperature change from above freezing to below freezing is stressful to materials such as concrete and asphalt. In Los Alamos, most freeze-thaw days occur from November (20.4 days) to March (21.5 days) with a maximum mean freeze-thaw day of 24.5 in December. Data provided by Bowen (1990) was measured at a 4- to 5-ft height above ground. Thus, the number of freeze-thaw days at ground level may be more than indicated because the diurnal temperature range is greatest at the ground. Also, the data do not include the multiple freeze-thaw cycles within a day.

Refer also to IBC 2003, Section 1905.12, for concrete materials and Section 2104.3 for masonry materials under low-temperature environmental conditions during construction.

3.5.2 Reference

Bowen, B. M., "Los Alamos Climatology," Los Alamos National Laboratory report LA-11735-MS (May 1990).

3.6 *Landslides*

The term landslide includes a wide range of ground movement, such as rock falls, deep failure of slopes, and mustow debris flows. Although gravity acting on an oversteepened slope is the primary reason for a landslide, other contributing factors for a landslide's occurrence are as follows:

- erosion by rivers, glaciers, or ocean waves created over steepened slopes,
- rock and soil slopes are weakened through saturation by snowmelt or heavy rains,
- earthquakes create stresses that make weak slopes fail,
- earthquakes of magnitude 4.0 and greater have been known to trigger landslides,
- volcanic eruptions produce loose ash deposits, heavy rain, and debris flows, and
- excess weight from an accumulation of rain or snow, stockpiling of rock or ore, from waste piles, or from man-made structures, may stress weak slopes to failure and other structures.

Slope material that becomes saturated with water may develop a debris flow or mud flow. The resulting slurry of rock and mud may pick up trees, houses, and cars, thus blocking bridges and tributaries causing flooding along its path. Although the physical cause of many landslides cannot be removed, geologic investigations, good engineering practices, and effective enforcement of land-use management regulations can reduce landslide hazards.

3.6.1 LANL Postfire Landslide Hazards

In June of 1977, the La Mesa fire in and around Frijoles Canyon, and in April of 1996, the Dome fire in Capulin Canyon burned 15,270 and 16,516 acres, respectively. Stream flow in both canyons was monitored and peak flow values were obtained for pre- and postfire conditions. Table 3-2 shows the dramatic increase of stormflows after the fire, especially in the first year after the fire. As vegetation is reestablished, the maximum peak flows were reduced in the following years (Veenhuis, 2000). Although postfire flood magnitudes in Frijoles and Capulin are much larger than the prefire magnitudes, they do not exceed the maximum floods per drainage area enveloping curves for two of the northern flood regions of New Mexico.

Table 3-2. Peak Flows at the Most Downstream Gage in the Frijoles and the Capulin Canyons

		Prefire maximum	Postfire maximum peak flow, (ft^3/s)		
Fire	**Canyon**	**peak flow, (ft^3/s)**	**First Year**	**Second Year**	**Third Year**
1977 the La Mesa fire	Frijoles	19	3,030	190	57
1996 the Dome fire	Capulin	25	3,630	375	125

Postfire landslide hazards include fast-moving, highly destructive debris flows that can occur in the years immediately after wildfires in response to high-intensity rainfall events, and those flows that are generated over longer time periods are accompanied by root decay and the loss of soil strength. Postfire debris flows are particularly hazardous because they can occur with little warning, can exert great impulsive loads on objects in their paths, can strip vegetation, block drainage ways, damage structures, and endanger human life. The Cerro Grande fire could also potentially result in the destabilization of preexisting deep-seated landslides over long time periods.

3.6.2 LANL Landslide Deposits

Figure 3-1 shows some of the landslide deposits (in red) in the Los Alamos Area, but the map is very incomplete. Although not shown on this map, extensive landslide complexes are known to exist on the Pajarito fault escarpment along NM 501 (West Jemez Road) and in Water and Los Alamos canyons. These landslide deposits (Holocene to middle Pleistocene) consist of a heterogeneous mixture of unconsolidated surficial materials and rock fragments in a wide range of sizes. The deposits include earth flows, rotational slides, translational slides, debris avalanches, and complex landslides (Varnes, 1978).
Extensive landslide deposits are found in the border between Technical Areas 33 and 70 and the White Rock Canyon. The landslide deposits in White Rock Canyon were mapped by Smith and others (1970) in

their overview of the Jemez Mountains and later studied in detail by Reneau and others (1995), Reneau and Dethier (1996a), and Dethier and Reneau (1996). The extensive landsliding is thought to have begun in the middle Pleistocene, after the Rio Grande had incised through Pleistocene volcanic rocks and into weakly indurated sediments of the Santa Fe Group (Reneau and Dethier, 1996a).

The following factors contributed to these landslides:

- Downcutting of canyons through volcanic rocks exposing underlying weakly indurated sedimentary rocks of low shear strength,
- Continued downcutting and removal of lateral support during times of high stream flow, resulting in the maintenance of steep slopes,
- Increases in pore water pressure in bedrock and surficial deposits, particularly during pluvial periods of the Pleistocene Epoch (Reneau and Dethier, 1996a), and
- Earthquakes caused by the Rio Grande rift seismic active area.

Landsliding in White Rock Canyon is known to have dammed the Rio Grande at least four times between 12,000 and 18,000 radiocarbon years ago and at about 40,000 radiocarbon years ago (Reneau and Dethier, 1996a).

Besides being incomplete in the western part of the Laboratory, limitations of the map in Figure 3-1 include the following:

- Small landslide deposits (< 0.004 square miles) were not recognized,
- Mustow landslide deposits covered by thick forest may not have been recognized,
- Older landslide deposits extensively modified by erosion may not have been identified, and
- Talus and debris flow deposits are not shown.

Because of the incomplete knowledge of landslides at LANL, field-site geotechnical investigations should be performed and address potential landslide deposits for the planning and design of new SSCs.

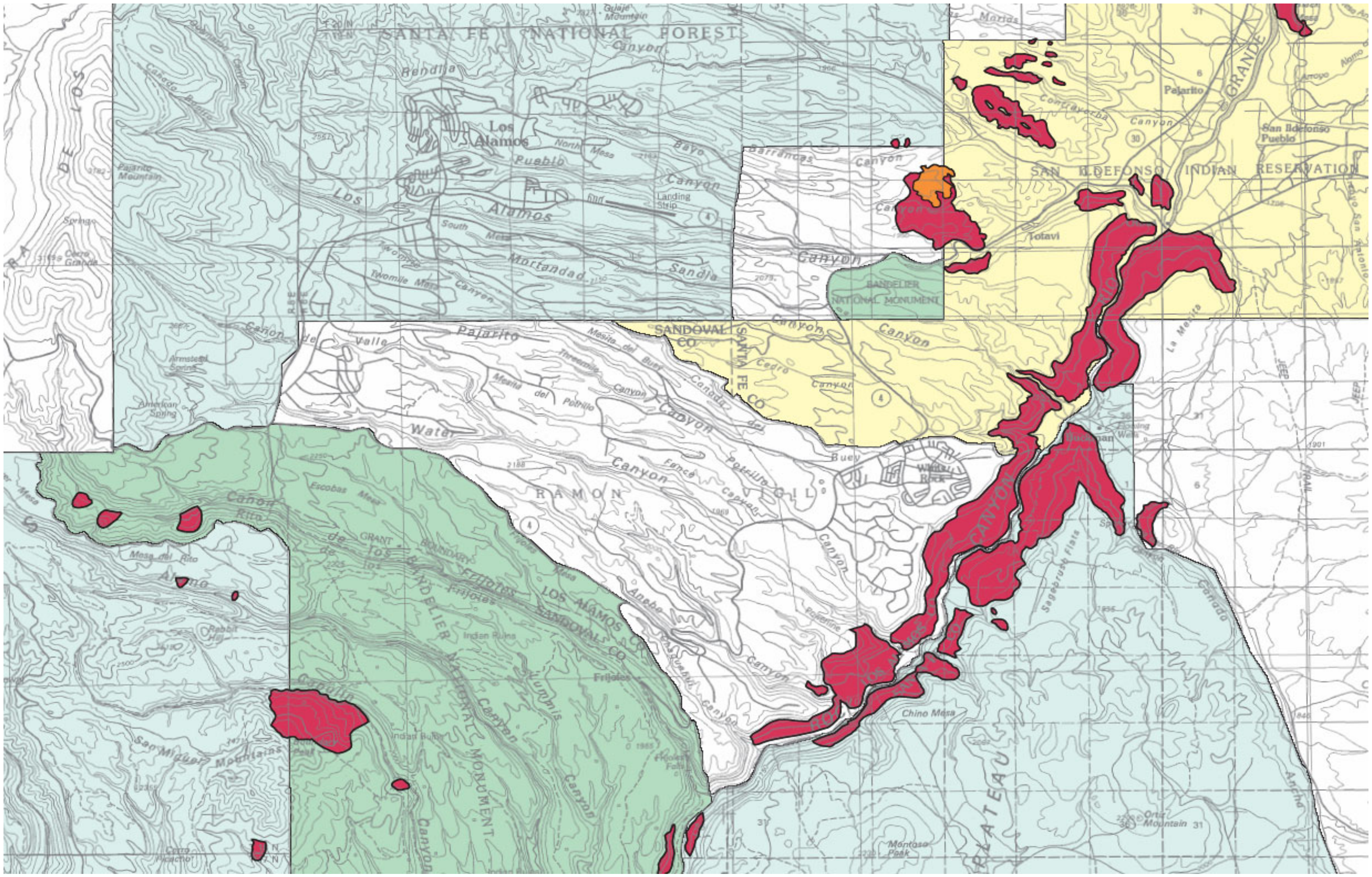

Figure 3-1. Section of the Preliminary Map of Landslide Deposits in the Los Alamos National Laboratory, New Mexico (by P. E. Carrara and D.P. Dethier, 1999).

3.6.3 References

http://landslides.usgs.gov/html_files/landslides/frdebris/cannon/cannon.html

http://landslides.usgs.gov/

http://pubs.usgs.gov/mf/1999/mf-2328/mf2328.pdf, Carrara, P. E. and Dethier, D. P., *Preliminary Map of Landslide Deposits in the Los Alamos 30'x60' Quadrangle*, New Mexico (1999).

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Reneau, S. L., D. P. Dethier, and J. S. Carney, "Landslides and other Mass Movements near Technical Area 33," Los Alamos National Laboratory report LA-12955-MS (1995).

Reneau, S. L. and D. P. Dethier, "Late Pleistocene Landslide-Dammed Lakes along the Rio Grande, White Rock Canyon, New Mexico," *Geological Society of America Bulleting* **108**, pp. 1492–1507 (1996a).

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Reneau, S. L., J. N. Gardner, and S. L. Forman, "New Evidence for the Age of the Youngest Eruptions in the Valles Caldera, New Mexico," *Geology* **24**, pp. 7–10 (1996).

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Varnes, D. J., "Slope Movement Types and Processes," in Schuster, R. L. and Krizek, R. J. Eds., "Landslides; Analysis and Control, Transportation Research Board Special Report 176," pp. 12–33 (1978).

Veenhuis, J. E., http://firescience.cr.usgs.gov/html/veenhuis_abs.html, 2nd USGS Windland Fire Workshop, Los Alamos, New Mexico, (2000).

3.7 Subsidence

Subsidence is the lowering of a portion of the earth's crust. Land subsidence can occur naturally or through human activity.

- *Natural subsidence* may occur when limestone, which is easily carved by underground water, collapses, leaving sink holes on the surface, such as in Florida. Dissolution of $CaCO_3$ results in caves and caverns that may collapse to depressions. Earthquakes can also cause subsidence of the land because of the movement of faults or causing liquefaction of soil. Volcanic activities cause subsidence by emptying the magma chambers.
- *Human induced subsidence* occurs by groundwater pumping or extraction of oil and gas. Land subsidence occurs when large amounts of ground water, oil, or gas have been withdrawn from certain types of rocks, such as fine-grained sediments. The rock compacts because the water is partly responsible for holding the ground up. When the water is withdrawn, the rocks fall in on themselves. Land subsidence may not be noticed too much because it can occur over large areas rather than in a small spot, like a sinkhole. That doesn't mean that subsidence is not a big event. States like California, Texas, and Florida have suffered damage to the tune of hundreds of millions of dollars over the years.

Land subsidence causes many problems including the following:

- Changes in elevation and slope of streams, canals, and drains,
- Damage to bridges, roads, railroads, storm drains, sanitary sewers, canals, and levees,
- Damage to private and public buildings, and
- Failure of well casings from forces generated by the compaction of fine-grained materials in aquifer systems.

In the Southwest, earth fissures are associated with land subsidence. These earth fissures are caused by the horizontal movement of sediments that occurs when groundwater is pumped. Such fissures are also often associated with extensional faulting.

DOE-STD-1022-94 states that ground settlement resulting from the ground shaking induced by NPH can be caused by two factors: (1) compaction of dry sands as a result of ground shaking, and (2) settlement caused by the dissipation of dynamically induced pore water in saturated sands. Differential settlement would cause more damage to facilities than would uniform settlement. Ground subsidence has been observed at the surface above relatively mustow cavities formed by mining activities and where large quantities of salt, oil, gas, or ground water have been extracted.

Los Alamos National Laboratory mainly located on top of the volcanic tuff mesas where the problem of subsidence is not an issue. If the possibility of surface subsidence were to occur at a particular site, consideration and investigation must be given to this hazard.

From the U.S. Department of Labor, the Code for Federal Regulations (CFR) for underground constructions states in 29 CFR 1926.800 Section (o)(2) the following: *"Subsidence areas*. The employer must ensure ground stability in hazardous subsidence areas by shoring, by filling in, or by erecting barricades and posting warning signs to prevent entry."

3.7.1 Sinkholes

Sinkholes are common where the rock below the land surface is limestone, carbonate rock, salt beds, or rocks that can naturally be dissolved by ground water circulating through them. As the rock dissolves, spaces and caverns develop underground. Sinkholes are dramatic because the land usually stays intact for a while until the underground spaces just get too big. If there is not enough support for the land above the spaces, then a sudden collapse of the land surface can occur. The most damage from sinkholes tends to occur in Florida, Texas, Alabama, Missouri, Kentucky, Tennessee, and Pennsylvania.

No sinkholes have ever been reported at LANL.

3.7.2 Long-Term Subsidence of the Rio Grande Valley

The Rio Grande Valley has a long-term subsidence as a result of the rift. See Section 2.1.2, for more information of the Rio Grande rift and its formation.

3.7.3 References

http://ga.water.usgs.gov/edu/earthgwlandsubside.html

http://geochange.er.usgs.gov/sw/changes/anthropogenic/subside

3.8 Surface Collapse

DOE-STD-1022-94 states that the existence of cavities in some geological materials (e.g., limestone, gypsum, anhydrite, etc.,) may lead to ground collapse. If collapse features are present, they must be considered and investigated with respect to their potential for causing deformation of the facility site and, if so, whether engineered stabilization measures are feasible.

3.9 Uplift

The ground uplift can be caused by various events such as the following:

- *A volcanic event*. Volcanoes will erupt magma through to the surface causing rising of the ground. Volcano forecasting has proved notoriously difficult. The reader is referred to Section 2.2**.**, Volcanic Events.
- *An earthquake event*. In an earthquake, serious damage can arise, not only from the ground shaking but from the fault displacement itself. Uplift of the ground can also be caused by an earthquake. Both normal and reverse faults produce vertical displacements. Both types of faults can be seen at the surface as fault scarps. For surface rupture phenomena, the reader is referred to Section 2.1, Earthquakes.
- *Excessive rain*. ASCE 7 Section 5.2 refers to the uplift on floors and foundations as a result of the upward pressure of water. Also, in the presence of expansive soils, foundations, slabs, and other components must be designed to tolerate the movement or resist the upward pressures caused by the expansive soils.

3.9.1 Reference

Bolt, B. A., *Earthquakes*, 3rd Edition, (W. H. Freeman and Company, New York, 1993).

3.10 Storm Surges

All hurricanes create storm surges. A storm surge is a rise in sea level along a coastline caused by the combination of a hurricane's surface winds and the physical geography of a coastline (see Figure 3-2). Surface winds above the ocean's surface push water toward the hurricane's eye, creating a mound of water. The mound of water is then influenced by the slope of the coastline as the hurricane approaches land. If the coastline is mustow, water cannot flow away from the mound and the mound grows. If the coastline is deep, water can disperse and the mound may grow slowly or disperse, depending on the hurricane strength. An example of a mustow-water coastline is the Gulf Coast, and an example of a deep-water coastline is found in New England.

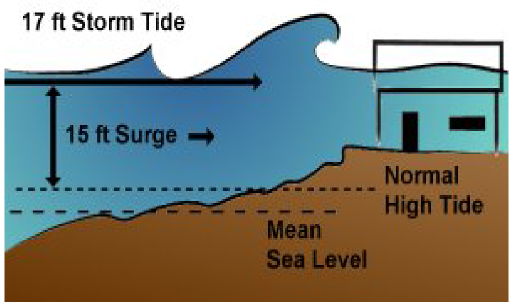

Figure 3-2. Storm surge.

3.10.1 Storm Surges in Los Alamos

No waterspouts have ever been reported in Los Alamos. Because Los Alamos is located inland, miles away from the sea, there is no risk of storm surges. Therefore, there is no load criteria related to this natural event.

3.10.2 Reference

http://www.nhc.noaa.gov/HAW2/english/storm_surge.shtml

3.11 Waterspouts

Waterspouts are simply tornadoes over water. Although they have a similar structure to some tornadoes, they form very differently. Waterspouts are common in tropical areas where thundershowers occur frequently, such as around the Florida Keys. Places around the Gulf of Mexico along with the Atlantic Coast northward to Chesapeake Bay are also likely to see waterspouts. Waterspouts have been reported on the West Coast from Tatoosh Island, Washington, south to San Diego, but they tend to be weak and short lived. Waterspouts also skip across the Great Lakes and Utah's Great Salt Lake from time to time.

At the ocean surface, winds are rushing faster and faster as they swirl into the vortex and then upward. Often with the waterspouts, the vortex is seen coming down from the cloud, but not obviously touching the ocean. Such vortices that don't seem to touch the ocean are called "funnels" or "funnel clouds."

3.11.1 Waterspouts in Los Alamos

No waterspouts have ever been reported in Los Alamos. Therefore, there is no load criteria related to this natural event.

3.11.2 Reference

http://www.usatoday.com/weather/tornado/wtspouts.htm

4 Loads Not Related to Natural-Phenomena Hazards

4.1 Live Load (L) and Roof Live Load (L_r)

Live loads are those loads produced by the use and occupancy of the building or other structure and do not include construction or natural phenomena loads such as wind load, snow load, rain load, earthquake load, flood load or dead load. IBC 2003 refers to ASCE 7-02, Chapter 4. Live loads must include all loads resulting from the occupancy and use of the structure, whether acting vertically down, vertically up, or laterally.

The live load to be used in design must be determined following the provisions of IBC 2003 or ASCE 7. Live load provisions include minimum uniformly distributed and concentrated live loads as shown in Table 4-1. Floors and other similar surfaces must be designed to support the uniformly distributed or concentrated live loads whichever produces the greater load effects. The concentrated load must be located so as to produce the maximum load effects in the structural members.

Table 4-1. References for Live Loads in IBC 2003 and ASCE 7-02

Live Loads	IBC	ASCE 7
Minimum uniformly distributed and concentrated live loads, L_o	Table 1607.1	Table 4-1
Partition loads	1607.5	4.2.2
Truck and bus garages	1607.6	
Loads on handrails, guards, grab bars, vehicle barriers	1607.7	4.4
Loads on fixed ladders		4.4
Impact loads	1607.8	4.7
Crane loads	1607.12	4.10
Interior walls and partitions	1607.13	
Roof loads, L_r	1607.11	4.9

4.1.1 Roof Live Loads (L_r)

Roof live loads are those loads produced

- During maintenance by workers, including their equipment and materials, and
- During the life of the structure by movable objects such as planters and by people.

All roofs at LANL must be designed for a minimum roof live load of 30 psf to account for unforeseen live loads (construction, man shoveling snow, etc).

4.1.1.1 Awnings and Canopies

Awnings and canopies must be designed for a uniform live load of 20 psf as well as for snow loads and wind loads.

4.1.2 Reduction in Live Loads

The minimum uniformly distributed live loads, L_o, are permitted to be reduced according to the general and alternate live load reduction provided in IBC 2003, Section 1607.9.

4.1.2.1 Reduction of Parking Garage Loads

IBC, Section 1607.9.1.2 states that "the live loads must not be reduced in passenger vehicles garages, except the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20%, but must not be less than calculated in Section 1607.9.1." However, ASCE 7, Section C4.8.3, states that "in view of the possible impact of very heavy vehicles in the future such as sport-utility vehicles, however, a design load of 40 psf is recommended with no allowance for reduction according to bay area." Therefore, no reduction in live load is permitted for LANL parking garages. Floors must be designed to support the uniformly distributed live load of 40 psf or the concentrated load of 3,000 lb acting on an area of 4.5 in. by 4.5 in., whichever produces the greater load effects.

4.2 Dead Load (D)

Dead loads are loads that remain permanently in place. They must include the weight of materials of construction incorporated into the building, including but not limited to walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding, and other similarly incorporated architectural and structural items, and fixed service equipment, including the weight of cranes.

4.2.1 Service Equipment

The weight of permanent service equipment, such as plumbing stacks, piping, heating and air-conditioning equipment, electrical equipment, flues, fire sprinkler systems, permanent distribution systems, and similar fixed furnishings must be included for purposes of design. The weight of service equipment that may be removed with change of occupancy of a given area must be considered as live load per DOE-O-6430.1A.

A dead load of 10 psf must be added to the best-estimate dead load for all floors for use in design to accommodate future dead load. This future dead load does not need to be included in the renovations or modifications to existing structures, if an accurate compilation of existing dead load is conducted based on documented site-specific verification of the load.

4.3 Self-Straining Forces (T)

The structural design must consider self-straining forces arising from the following:

- Restrained dimensional changes (contraction or expansion) resulting from:
    - Temperature change,
    - Shrinkage,
    - Moisture change,
    - Creep in component material, and
    - Similar effects
- Movement resulting from differential settlements of foundations.

Unless specifically addressed through analysis, the effects of self-straining forces must be accommodated by the placement of relief joints, suitable framing systems, or other details to minimize the effects of self-straining forces.

Examples include moments in rigid frames that undergo differential foundation settlements and shears in bearing walls that support concrete slabs that shrink. Unless provisions are made for self-straining forces, stresses in structural elements, either alone or in combination with stresses from external loads, can be high enough to cause structural distress.

According to DOE-O-6430.1A, the design of structures must include the effects of stresses and movements resulting from variations in temperature. The rise and fall in the temperature must be determined for the localities in which the structures are to be built. Structures must be designed for movements resulting from the maximum seasonal temperature change. The design must provide for the lags between air temperatures and the interior temperatures of massive concrete members or structures. In cable-supported structures, changes in cable sag and tension must be considered.

Concrete and masonry structures must be investigated for stresses and deformations induced by creep and shrinkage. For concrete and masonry structures, the minimum linear coefficient of shrinkage must be assumed to be 0.0002 inch/inch, unless a detailed analysis is undertaken. The theoretical shrinkage displacement must be computed as the product of the linear coefficient and the length of the member. Contraction joints, formed, sawed, or tooled groove in a concrete structure will be defined to create a weakened plane and regulate the location of cracking resulting from the dimensional change of different parts of the structure in accordance with ACI 318 and ACI 349.

4.4 Fluid Loads (F)

Fluid load, *F*, defines structural actions in structural supports, framework, or foundations of a storage tank, vessel, or similar container as a result of stored liquid products.

The product in a storage tank shares characteristics of both dead and live load. It is similar to a dead load in that its weight has a maximum calculated value, and the magnitude of the actual load may have a relatively small variation. However, it is not permanent; emptying and filling causes fluctuating forces in the structure, the maximum load may be exceeded by overfilling; and densities of stored products in a specific tank may vary.

The design of components of buildings and other structures must include the effects of fluid and gas pressures, both internal and external.

4.5 Lateral Soil Pressure Loads (H)

In the design of basement walls, foundation, retaining walls, and similar approximately vertical structures below grade, provisions must be made for the lateral pressure of adjacent soil. Soil and hydrostatic pressure loads must follow IBC 2003, Section 1610, or ASCE 7, Section 5.0.

4.5.1 Minimum Lateral Soil Loads

Soil loads specified in ASCE 7 Table 5-1 or IBC Table 1610.1 as a function of backfill soil material type must be used as the minimum design lateral soil load unless specified otherwise in a soil investigation report approved by LANL. The lateral pressure must be increased if soils with expansion potential are present at the site as determined by a geotechnical investigation.

Basement walls and other walls in which horizontal movement is restricted at the top must be designed for at-rest pressure. Retaining walls free to move and rotate at the top are permitted to be designed for active pressure. Retaining walls must be designed to ensure stability against overturning, sliding, excessive foundation pressure, and water uplift. Retaining walls must be designed for a safety factor of 1.5 against lateral sliding and overturning.

4.5.2 Surcharge Loads

Due allowance must be made for possible surcharge from fixed or moving loads.

4.5.3 Hydrostatic Loads

When a portion or the whole of the adjacent soil is below a free water surface, computations must be based on the weight of the soil diminished by buoyancy, plus full hydrostatic pressure. The hydrostatic pressures must correspond to the maximum probable groundwater level.

4.5.4 Seismic Loads

IBC Section 1802.2.7 states that, for Seismic Design Category D structures, a soil investigation must be conducted and must include the determination of the lateral pressures on basement and retaining walls resulting from earthquake motions.

ASCE 4, Section 3.5.3 provides acceptable means of accounting for seismic-induced lateral soil pressures on subterranean structural walls. Per ASCE 4, the summation of the calculated dynamic seismic soil pressures and the static earth pressure must not exceed the soil static passive earth pressure. Seismic-induced soil pressure loads are included in earthquake loads (E).

The seismic design and evaluation for underground high-level waste storage tanks will be performed following the guidelines of the Brookhaven National Laboratory report BNL-52361.

4.6 Blast Hazards

DOE-O-6430.1A states that building structures (excluding explosive facilities) that house operations that may release energy from the rupture of equipment or explosions, either inadvertently or purposely (such as testing), must be designed to control the resulting internal shock pressure loads per applicable criteria. The probable consequence of design basis accidents (DBAs) involving internally generated missiles or blast effects must be considered. Such DBAs typically involve the failure of high-speed rotating machinery, cranes, experimental facilities, high-energy fluid system components, or explosives. Structures required to function following such accidents must be designed to withstand these DBAs.

4.6.1 Designed Experiment Blast Load (L_{EB})

LANL conducts experiments involving explosions. Where such experiments take place within a building, the experimental explosion effects must be contained within an internal structure so that loadings on the building are minimized. The containment structure may impose reaction forces on the building during the experimental explosion. Such reaction forces are one form of designed experiment blast loads, L_{EB},

In addition, experimental explosions may take place exterior to the building under consideration. The forcing function and duration of the blast loading will be based on the TNT equivalency of the maximum quantity and closest possible distance from the structural component in question of explosives and propellants in the designed experiment in accordance with TM-5-1300.

The dynamic characteristics of these short-duration blast loads must be considered in building evaluation and design. For external explosions, potential fragments and ground shock must be considered in addition to blast overpressure. Analysis for such loads may be conservatively performed by linear static analysis at the peak loading. Alternately, the dynamic nature of the load may be accounted for to obtain more realistic results.

Designed Experiment Blast Loads, L_{EB}, will most likely affect only limited portions of the building where experiments are conducted. Because these loads can be repeated many times during the life of the structure, the structure should be designed to remain elastic to avoid the progressive damage to the building. Experimental blast loads are short-duration, pulse loads.

4.6.2 Accidental Blast Load (A_B)

When evaluating for accidental blast load, the loading, A_B, will replace E (earthquake) loads in the load combination equations. The resulting loads on building structures can be of very large amplitude, but these loads will be of very short duration in a single pulse, on the order of a fraction of a second. All potential blast effects must be considered, including blast overpressure, gas pressure, fragments, and ground shock.

Accidental blast load combinations must be based on deformation-based acceptance criteria and must consider inelastic energy absorption by limiting deformations to ductility or plastic hinge rotation limits. Because the amplitude of blast overpressure acting on building surfaces can be very large compared to earthquake or wind forces acting on the surfaces, it is necessary to account for yielding of building structural members in order to obtain an economical design. Therefore, structural analysis for accidental blast loads is accomplished by nonlinear response history analyses. Such analyses may be accomplished by nonlinear, dynamic finite-element computer programs. Alternately, there are more simple approximate methods (i.e., TM-5-1300) that fully account for both the dynamic character of the structure and the blast load as well as the nonlinear behavior of the structure withstanding the blast loads.

4.6.2.1 Explosive Facilities

Explosive facilities are those facilities or locations used for storage or operations with explosives or ammunition. Accidental explosions may occur during the handling of high-explosive materials resulting in a detonation. Per DOE-O-420.1A, the safety design of all new DOE explosives facilities and all modifications to existing explosives facilities must conform to the DOE explosives safety requirements established in the DOE Explosives Safety Manual, DOE-M-440.1-1. Facility structural design and construction must comply with the requirements of TM-5-1300, Structures to Resist the Effects of

Accidental Explosions, and DOE/TIC-11268, A Manual for the Prediction of Blast and Fragment Loading of Structures. Blast-resistant design for personnel and facility protection must be based on the TNT equivalency of the maximum quantity of explosives and propellants permitted. In accordance with TM-5-1300, the TNT equivalency must be increased by 20 percent for design purposes.

DOE-G-420.1-1 states that the technical basis for establishing explosives quantity-distance separation for facility location, design, and operation (under normal and potential DBA conditions) must follow the stricter of the criteria provided in Department of Defense Standard, DoD 6055.9-STD, Ammunition and Explosives Safety Standards. DoD 6055.9 specifies the minimum distance for protection from hazardous fragments to facility boundaries, critical facility, and inhabited structures, unless it can be shown that there will be no hazardous fragments or debris at lesser distances.

4.6.2.2 Facilities for Storage and Handling of Flammable Materials

Accidental explosions may result from the storage and handling of flammable materials, such as hydrocarbons, because of a release of hydrocarbons followed by the ignition resulting in a vapor cloud explosion or deflagration. A release of flammable vapor in a region of adequate confinement and obstacle density is a potential source of a vapor cloud explosion. The blast load resulting from a potential vapor cloud explosion, in terms of incident side-on overpressure and the associated impulse or duration may be estimated using the guidelines provided by the CCPS book, Guidelines for Evaluating the Characteristics of Vapor Cloud Explosions, Flash Fires, and Bleves.

4.6.3 Abnormal Loads Associated with Nuclear Facilities

Accidents during the operation of high-energy systems could lead to loads on SSCs. Design-basis accident conditions for the building under consideration will be provided by LANL, if applicable.

Nuclear-safety-related concrete structures must be designed for impulsive and impactive loads using the ACI 349, Appendix C. Impactive loads are time-dependent loads as a result of the collision of masses that are associated with finite amounts of kinetic energy.

Impactive loading may be defined in terms of time-dependent force or pressure. Examples of impactive loads to be considered are loadings as a result of whipping pipes and of fuel cask drop.

Impulsive loads are time-dependent loads which are not associated with the collision of solid masses. Impulsive loads are, for example, loadings as a result of jet impingement, compartment pressurization, and pipe-whip restraint reactions.

Examples of abnormal loads generated by a postulated high-energy pipe break accident are listed below:

- Differential Pressure Load (P_a)
  Differential pressure load, or related internal moments and forces, generated by a postulated pipe break.
- Differential Temperature Load (T_a)
  Internal moments and forces caused by temperature distributions within the concrete structure occurring as a result of accident conditions generated by a postulated pipe break.
- Piping and Equipment Reactions (R_a)
  Piping and equipment reactions, or related internal moments and forces, under thermal conditions generated by a postulated pipe break.
- Missile impact load (Y_m)
  Missile impact load, or related internal moments and forces, on the structure generated by a postulated pipe break.
- Jet Impingement Load (Y_j)
  Jet impingement load, or related internal moments and forces, on the structure generated by a postulated pipe break.
- Line break reactions (Y_r)

Loads, or related internal moments and forces, on the structure generated by the reaction of the broken pipe during a postulated break.

4.6.4 External Man-Induced Blast Hazards

LANL must specify whether external man-made hazards, such as airplane crash impact, or terrorist attacks need to be considered. The following guidelines are provided to assist the project manager in making this determination:

- Buildings with high occupancy, greater than 300 occupants (i.e., NISC, Admin Bldg)
- Buildings with high consequence of failure (high risk, essential mission, etc.)
- ML-1 or ML-2 nuclear facilities
- Important buildings for which potential terrorist threats are not mitigated by other security measures

4.6.4.1 Airplane Crashes

Unless the safety analysis can demonstrate that the risk from an aircraft crashing into the facility is acceptable, potential aircraft crashes must be considered among the spectrum of man-made missiles that confinement structures must be designed to withstand or against which they must be protected. LANL must specify whether an airplane crash hazard evaluation is required based on a probabilistic hazard analysis of the building.

The methodology for evaluating an aircraft crash impact will be performed using the DOE Standard 3014-96. This standard provides sufficient information to evaluate and assess the significance of aircraft crash risk on facility safety without expending excessive effort where it is not required.

4.6.4.2 Nearby Explosions and Terrorist Attacks

The potential effects of a major explosion at a nearby facility or a transportation route, must be considered among the spectrum of external blast effects and missiles that confinement structures must be designed to withstand or against which they must be protected. Major explosions can also occur inside the facility being designed.

Department of Defense (DoD) UFC 4-010-01 documents the minimum antiterrorism standards for buildings. This document seeks ways to minimize the likelihood of mass casualties from terrorist attacks against DoD personnel in the buildings in which they work and live. The document provides mandatory and recommended minimum antiterrorist standards for new and existing inhabited buildings and mandatory standards for expeditionary and temporary structures.

The philosophy of these standards is to build greater resistance to terrorist attack and to provide the easiest and more economical methods to minimize injuries and fatalities in the event of a terrorist attack. The primary methods to achieve this outcome are as follows:

- To maximize the standoff distance
- To construct superstructures to avoid progressive collapse
- To minimize flying debris hazards
- To provide effective building layout
- To limit airborne contamination
- To provide mass notification
- To facilitate future upgrades

The location, size, and nature of terrorist threats are unpredictable. The following are the terrorist tactics upon which UFC 4-010-01 standards are based:

- Explosives. Their means of delivery are as follows:
  - Vehicle bombs,
  - Waterborne vessel bombs,
  - Place bombs,
  - Mail bombs,

- Indirect fire weapons,
- Direct Fire weapons,
- Fire, and
- Chemical, biological and radiological weapons.

4.6.4.2.1 Mandatory Minimum Standards for Structural Design

If the minimum standoff distances are achieved, conventional construction should minimize the risk of mass casualties from a terrorist attack. Even if those standoff distances can be achieved, however, the following additional structural issues must be incorporated into building designs to ensure that buildings do not experience progressive collapse:

- *Progressive Collapse Avoidance*

  For all new inhabited buildings of *three stories or more* above ground, the superstructure must be designed to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage. This is achieved through an arrangement of the structural elements that provides stability to the entire structural system by transferring loads for any locally damaged region to adjacent regions capable of resisting those loads without collapse. This will be accomplished by providing sufficient continuity, redundancy, or energy dissipating capacity (ductility, damping, hardness, etc.), or a combination thereof, in the members and connections of the structure.

  All exterior vertical load-carrying columns and walls must be designed to sustain a loss of lateral support at any of the floor levels by adding one story height to the nominal unsupported length. In addition, the structures must be analyzed to ensure that they can withstand the removal of one primary exterior vertical or horizontal load-carrying element (i.e., a column or a beam) without progressive collapse. All floors must be designed to improve their capacity to withstand load reversals as a result of the explosive effects. The floors will be designed to withstand a net uplift equal to the dead load plus one-half the live load.

- *Structural Isolation*

  All additions to existing buildings must be designed to be structurally independent from the adjacent existing building.

  Where there are areas of buildings that do not meet the criteria for inhabited buildings, the superstructure design of these areas must be independent from the inhabited area.

- *Building Overhangs*

  Avoid building overhangs with inhabited spaces above them where people could gain access to the area underneath the overhang. If such overhangs must be used, incorporate mitigating measures in accordance with UFC 4-010-01.

- *Exterior Masonry Walls*

  Unreinforced masonry walls are prohibited for the exterior walls of new buildings.

4.6.4.2.2 Mandatory Minimum Standards for Architectural Design

Even where the minimum standoff distances are achieved, many architectural design issues must be incorporated to improve overall protection of personnel inside the buildings, such as windows, skylights, glazed doors, building entrance layout, exterior doors, mailrooms, roof access, and overhead mounted architectural features (UFC 4-010-01).

4.6.5 Vibratory Loads

Equipment supports must be designed to avoid resonance resulting from the harmony between the natural frequency of the structure and that of the operating frequency of reciprocating or rotating equipment supported on the structure. Resonance must be prevented by designing equipment isolation supports to reduce the dynamic transmission of the applied load to as low a level as can be economically achieved in the design.

4.7 References

CCPS, *Guidelines for Evaluating the Characteristics of Vapor Cloud Explosions, Flash Fires, and Bleves*, (Center for Chemical Process Safety, American Institute of Chemical Engineers, New York, 1994).

Brookhaven National Laboratory, “Seismic Design and Evaluation Guidelines for the DOE High Level Waste Storage Tanks and Appurtenances,” Brookhaven National Laboratory report BNL-52361 (1995).

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Unified Facilities Criteria (UFC), *DoD Design and Analysis of Hardened Structures to Conventional Weapons Effects*, UFC 3-340-01 (June 2002).

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Department of Energy, *A manual for the Prediction of Blast and Fragment Loading of Structures*, DOE/TIC-11268, U. S., Albuquerque Operations, Amarillo Area Office, Facilities and Maintenance Branch, P.O. Box 30030, Amarillo, TX 79210 (1992).

Department of the Army Technical Manual, *Structures to Resist the Effects of Accidental Explosions*, Technical Manual TM-5-1300, 6 Volumes, Special Publication ARLCD-SP-84001 (November 19, 1990).

5 Design Loads on Structures during Construction

Minimum design-load requirements during construction for buildings and other structures will follow the guidelines of ASCE 37-02. This standard provides minimum design-load requirements during construction for buildings and other structures. The standard addresses partially completed structures and temporary structures used during construction. The loads specified are suitable for use either with strength design or with allowable stress-design criteria.

5.1 Specified loads

The following loads must be considered during construction:

Loads		
Final Loads	D	– Dead load
	L	– Live load
Construction loads		
1) Dead load of temporary structures	C_D	– Construction dead load
2) Material dead loads	C_{FML}	– Fixed material load
	C_{VML}	– Variable material load
3) Construction procedure loads	C_P	– Personnel and equipment loads
	C_H	– Horizontal construction loads
	C_F	– Erection and fitting forces
	C_R	– Equipment reactions
	C_C	– Lateral pressure of concrete
Lateral earth pressures	H	– Lateral soil pressure load
Environmental loads	W	– Wind loads
	T	– Thermal loads
	S	– Snow loads
	E	– Earthquake loads
	R	– Rain
	I	– Ice

5.2 Load Combinations

Load combinations for strength and allowable stress design, minimum load factors for use with strength design, and reduction of construction loads are found in ASCE 37-02.

5.3 Final Loads

5.3.1 Dead Loads, D

The dead load, D, is the weight of the permanent construction in place at the particular time in the construction sequence that is under consideration. It includes all construction in place that is temporarily shored or braced. It also includes construction for which the primary structural system is complete, but which is being used to support construction materials and construction equipment. See also Section 4.2.

5.3.2 Live Loads, L

The live load, L, is the load produced by the use or occupancy of a structure that is under construction and may vary at different stages of construction. These loads may be imposed on

1. Construction in place,
2. Partially demolished structures, or
3. Temporary structures.

Refer also to Section 4.1.

5.4 Construction Loads

Construction loads are those loads imposed on a partially completed or temporary structure during and as a result of the construction process. Construction loads include, but are not limited to:

1. Materials,
2. Personnel, and
3. Equipment.

5.4.1 Construction Dead Load, C_D

The construction dead load, C_D, is the dead load of temporary structures that are in place at the stage of construction being considered.

5.4.2 Material Dead Loads

The material dead loads consist of two categories:

1. C_{FML} – fixed material load (FML)
   The FML is the load from materials that is fixed in magnitude.
2. C_{VML} – variable material load (VML)
   The VML is the load from materials that varies in magnitude during the construction process.

Material loads may be either distributed or concentrated loads. The distinction between an FML and a VML is not location or position on the structure; rather, it is the variability of the loading magnitude.

The stockpiling of any material is considered a VML (scaffold, forms, rebar, metal deck, barrels, drywall, tile, roofing materials, and so on). Some materials, such as scaffold or forms, are considered VMLs when they are stockpiled but may be considered FMLs when they are placed in their final end-use position.

5.4.2.1 Concrete Load

The weight of concrete placed in a form for the permanent structure is a material load. When the concrete gains sufficient strength so that the formwork, shoring, and reshoring are not required for its support, the concrete becomes a dead load.

5.4.2.2 Materials Contained in Equipment

Materials being lifted by, or contained in, equipment are part of the equipment load, not a material load. Once such materials have been discharged from the equipment, they become a material load.

5.4.3 Construction Procedure Loads

5.4.3.1 Personnel and Equipment Loads, C_P

Personnel and equipment loads must be considered in the analysis or design of a partially completed or temporary structure. The design or analysis of the structure must be governed by either a uniformly distributed or a concentrated personnel and equipment load, whichever creates the most severe strength and/or serviceability condition. The governing load must be assumed to be placed in the pattern or location that creates the most severe strength and/or serviceability condition.

Uniformly Distributed Loads
Uniform loads must be selected to result in forces and moments that envelope the forces and moments that would result from the application of concentrated loads that could occur and are not separately considered.

Concentrated Loads
ASCE 37-02 recommends the minimum concentrated personnel and equipment loads as shown in Table 5-1.

Table 5-1. Minimum Concentrated Personnel and Equipment Loads

Action	Minimum Load[a] (lb)	Area of Load Application (in. x in.)
Each person	250	12 x 12[b]
Wheel of manually powered vehicle	500	Load divided by tire pressure[c]
Wheel of powered equipment	2,000	Load divided by tire pressure[c]

[a]Use actual loads when they are larger than tabulated here.
[b]Need not be less than 18 in. center to center.
[c]For hard rubber tires, distribute load over an area 1 in. by the width of the tire.

The concentrated load must be located to produce the maximum strength and/or serviceability conditions in the structural members.

Impact Loads
The concentrated loads specified in Table 5-1 include adequate allowance for ordinary impact conditions. Provisions must be made in the structural design for loads that involve predictable unusual vibration and impact forces.

5.4.3.2 Horizontal Construction Loads, C_H

Horizontal load criteria are provided in ASCE 37-02, if appropriate:

1. For wheeled vehicles transporting materials:
   - 20% of the fully loaded vehicle weight f or a single vehicle
   - 10% of the fully loaded vehicle weight for two or more vehicles.

   This force must be applied in any direction of possible travel, at the running surface.
2. For equipment reaction (see Section 5.4.3.4)
3. 50 lb per person applied at the level of the platform in any direction.
4. 2% of the total vertical load in any direction and must be spatially distributed in proportion to the mass. This load need not be applied concurrently with wind or seismic load.

5.4.3.3 Erection and Fitting Forces, C_F

Forces caused by erection (alignment, fitting, bolting, bracing, guying, and so on) must be considered.

5.4.3.4 Equipment Reactions, C_R

The reaction from equipment, with due consideration of all loading conditions, must be used in the design of the temporary or partially completed structure. The equipment reactions must include the full weight of the equipment operating at its maximum rated load in conjunction with any applicable environmental loads, unless the use is restricted and revised reactions are developed.

The rated equipment is that from which reactions are given by the equipment manufacturer or supplier. For nonrated equipment, the designer is to determine the reactions by analysis.

The reaction of equipment must be increased by 30% to allow for impact, unless other values (either larger or smaller) are recommended by the manufacturer, are required by the LANL POC, or are justified by analysis.

5.4.3.5 Lateral Pressure of Concrete, C_C

Form Pressure
Formwork must be designed for the lateral pressure of the newly placed concrete given by:

$$C_C = w \times h$$

where
C_C = lateral pressure (psf)
w = unit weight of fresh concrete (pcf)
h = depth of fluid or plastic concrete (ft)

If concrete is made of Type–I cement, w = 150 pcf, containing no pozzolans or admixtures, having a slump of 4 in. or less, and normal internal vibration to a depth of 4 ft or less, then,

1)

$$C_C = 150 + 9{,}000 \cdot \frac{R}{T},$$

for columns with $600\ psf \le C_C \le 3{,}000\ psf$ and $C_C < 150 \cdot h$,

for walls with $600\ psf \le C_C \le 2{,}000\ psf$, $C_C < 150 \cdot h$, and a rate of placement of less than 7 ft per hour.

2)

$$C_C = 150 + \frac{43{,}000}{T} + 2{,}800 \cdot \frac{R}{T},$$

for walls with $600\ psf \le C_C \le 2{,}000\ psf$, $C_C < 150 \cdot h$, and a rate of placement of 7 to 10 ft per hour.

where
R = rate of placement (ft/h)
T = temperature of concrete in the form (ºF)

If concrete is pumped from the base of the form, the form must be designed for a full hydrostatic head of concrete, C_C = w x h, plus a minimum allowance of 25% for pump surge pressure.

Slipform Pressure
For a slipform concreting operation, the lateral pressure of fresh concrete to be used in designing the forms, bracing, and wales must be calculated as

$$C_C = c + 6{,}000 \cdot \frac{R}{T},$$

where c is 100 psf for concrete placed in 6- to 10-in. lifts with slight or no vibration, and 150 psf for concrete that requires additional vibration, such as gastight or containment structures.

Shoring Loads
When shores are required to support the load of newly placed concrete, these shores must be maintained until the concrete has gained enough strength to be self-supporting.

5.5 Lateral Soil Pressure Load

Design values of lateral soil pressures must be determined in accordance with Section 4.5.

5.6 Environmental Loads

During construction, the importance factor, I, must be 1.0 for all environmental loads, regardless of what the importance factor is for the completed structure.

5.6.1 Wind Load

Wind loads must be calculated in accordance with Section 2.5. The design wind pressure based on ASCE 7-02 does not need to meet the minimum design wind load requirements.

Design velocity
The design wind speed must be taken as the following factor (Table 5-2) multiplied by the basic wind speed in Table 2-15:

Table 5-2. Wind Load Factor

Construction Period	Factor
Less than 6 weeks	0.75
6 weeks to 1 year	0.80
1 to 2 years	0.85
2 to 5 years	0.9

Follow ASCE 37-02 for wind loads of frameworks without cladding and structures placed in regions of accelerated wind speed (near building edges and corners).

5.6.2 Thermal Loads

Provisions must be made for thermal distortions of the structure and architectural components when structures are erected during the conditions shown in ASCE 37-02.

Buildings that are most susceptible are those that have relatively unrestrained frames supporting rigid elements, such as precast panels or masonry infilling walls, that are not a part of the primary structural system. Long buildings, in which the cumulative dimensional changes can be large, and buildings erected during the extremes of the construction season, when ambient temperatures can be very different from end-use temperature can be particularly susceptible. Also, structures with braced bays or shear walls in line but spaced far apart can generate substantial forces as the intermediate framing attempts to move with temperature changes.

5.6.3 Snow Loads

When snowfall is expected during the construction period, snow loads must be determined for surfaces on which snow could accumulate in accordance with Section 2.8.

Ground Snow Loads
The ground snow loads, p_g, given in Table 2-23, must be modified by the following factors in Table 5-3:

Table 5-3. Snow Load Factors

Construction Period	Factor
5 years or less	0.8
More than 5 years	1.0

Thermal, Exposure, and Slope Factors
The thermal factor, C_t, and the exposure factor, C_e, will be for the conditions that exist during construction. The slope factor, C_s, will be determined based on the construction-phase values of C_t and C_e.

Drainage
Where drainage provisions may become blocked during construction (e.g., by freezing), the extra loads created by such blockages must be included.

Loads in Excess of the Design Value
Surfaces on which snow and ice accumulate must be monitored, and any loads in excess of construction-phase design loads must be removed before construction proceeds.

5.6.4 Earthquake Loads

Earthquake loads must be calculated in accordance with Section 2.1.

All temporary structures and supports must be designed and treated as Performance Category (PC) – 1, regardless of the group classification of the completed structure. The restrictions on types of structural systems in seismic Performance Category D do not apply, as long as the height of the temporary bracing system above the final seismic resisting system does not exceed 100 ft.

The R factor used for temporary bracing systems must not exceed 2.5, consistent with a low level of ductility and redundancy, unless the system is detailed in accordance with the provisions of ASCE 7-02. Therefore, the seismic base shear is $V = 0.26W$ if the simplified analysis procedure (IBC 2003, Section 1617.5) is used, or $V = 0.22W \leq 0.1W/T$ for the equivalent lateral force procedure (ASCE 7-02, Section 9.5.5). Only the requirements dealing with the strength of the seismic-resisting structural system need be satisfied.

The drift limitations and the nonstructural provisions are not required for temporary structures and for structures during their construction phases.

5.6.5 Rain Loads

Rain loads must be calculated in accordance with Section 2.7.

For temporary conditions that exist for one month or less, rain loads need not be considered for construction during months with historical rainfall averages of less than 1 inch per month. Monthly precipitation means for Los Alamos and White Rock is less than 1 inch between November and June (Bowen, 1990).

5.6.6 Ice Loads

Ice loads will be calculated in accordance with Section 2.9.

For construction during seasons when structures are not susceptible to an accumulation of ice, ice loads need not be considered.

Structures that will be enclosed when construction is complete and that are designed for live loads of 20 psf or more need not be considered as ice-sensitive structures while open during construction.

5.7 Reference

Bowen, B. M., "Los Alamos Climatology," Los Alamos National Laboratory report LA-11735-MS (May 1990).

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