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To the Graduate Council:

I am submitting herewith a thesis written by Wayne S. Moore entitled "Wind design loads for temporary structures." I have examined the final electronic copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

Richar M. Bennett, Major Professor

We have read this thesis and recommend its acceptance:

Edwin G. Burdette, James H. Deatherage, Terry L. Miller

Accepted for the Council:
Carolyn R. Hodges

Vice Provost and Dean of the Graduate School

(Original signatures are on file with official student records.)

To the Graduate Council:

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We have read this thesis and
recommend its acceptance:

Edwin G. Burdette

Terry L. Miller

Accepted for the Council:

Associate Vice Chancellor and
Dean of the Graduate School

WIND DESIGN LOADS FOR TEMPORARY STRUCTURES

A Thesis
Presented for the
Master of Science
Degree

The University of Tennessee, Knoxville

Wayne S. Moore
May 1996

DEDICATION

This thesis is dedicated to my loving wife

Deborah H. Moore

and my darling daughter

Kristen E. Moore

without whose infinite love, patience, and understanding,

this could not have been accomplished.

ACKNOWLEDGMENTS

It has been both an honor and a pleasure to be able to return to the University of Tennessee-Knoxville and complete my Masters of Science degree. I would like to sincerely thank the faculty, staff, and graduate students of the Department of Civil and Environmental Engineering for their encouragement and support. I am particularly grateful to Dr. Richard M. Bennett, who served as my major professor and who provided much needed guidance in the completion of my studies and in the preparation of this thesis. I would also like to thank Dr. Edwin G. Burdette, Dr. James H. Deatherage, and Dr. Terry L. Miller for serving on my committee and in participating in the review of this thesis.

I would like to thank my former coworkers at PARSONS- Fairfield, Ohio and present coworkers at Lockwood Greene Technologies- Oak Ridge, Tennessee for their well-wishes and support. In particular, I am grateful to Dr. C.V. Char for his guidance in the preparation of this thesis.

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Any errors and/or omissions in this report should be solely attributed to the author and should not reflect negatively on any of the reviewers of this thesis.

ABSTRACT

Determining appropriate wind speeds and loads for temporary structures for construction projects has perplexed owners, engineers, and contractors alike. In addressing wind loads for temporary structures such as shoring, personnel, and work space structures, two avenues appear to have been taken. For commercial construction projects, less conservative methods have been practiced, including the lack of design altogether. An entirely different approach would be that taken for government construction projects, in particular the United States Department of Energy (DOE). The DOE does not differentiate between temporary, short-lived structures and permanent structures whose normal design lives approach 50 years. Thus, the same wind design load criteria are applied to both classes of structures.

It is estimated that, on average, the cost of temporary structures to resist wind loads for commercial construction is low, relative to the overall cost of construction. Given the disproportionate number of structural failures that occur during construction, as compared to failures during service life, this low cost at times becomes apparent. A large number of these failures occur due to lateral loading, primarily wind. On the other hand, the DOE seems to advocate a "hell for stout" approach to the design of temporary structures. This conservative approach is resulting in increased construction costs and schedule slippages. This is especially evident for DOE decontamination and decommissioning (D&D) construction projects where construction change orders are resulting from a difference in design philosophy between contractors and the DOE pertaining to temporary structures. Typically, when a contractor prepares a bid for a D&D construction project, a unit cost for the design and construction of

temporary structures will be included. This cost will normally be based on the contractor's historical design and construction practice for temporary structures and will tend to be less than that required for the structure to resist the wind design loads for permanent structures. With the anticipated total cost of all DOE D&D construction projects approaching $50 billion (Decontamination and Decommissioning Integrated Demonstration Strategy, 1991), a significant cost savings may be realized if a more refined method for determining wind loads for temporary structures is determined.

In this study, the rationale for applying the same criteria for wind design for permanent structures and temporary structures for DOE D&D construction projects is explored. Related codes, guides, and publications used for determining wind design loads for temporary structures in other industries and countries are also explored and evaluated. For the purpose of this study, temporary structures are defined as those structures with design lives less than five years. Permanent structures are defined as those structures with design lives greater than this. In addition, the scope of this study is limited to those structures classified as PC-1 or PC-2 (DOE 5480.28, 1993), or General Use or Important/Low Hazard per criteria as defined in the now superseded document, UCRL 15910 (1988). These classifications are consistent with the type of operations primarily conducted within temporary facilities for DOE construction projects and can be related to commercial structures by comparing the appropriate importance factors to governing basic building codes.

Structural reliability methods are used to determine design wind speeds for DOE PC-1 and PC-2 type SSCs with design lives from 2 to 5 years. These are obtained by setting the reliability

indices and probabilities of failure for temporary structures equal to those obtained for permanent structures. Wind speeds are obtained with regard to the nature of the wind loading as well as the structural resistance of the SSC.

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ACRONYMS

ALARA	As Low As Reasonably Achievable
ASCE	American Society of Civil Engineers
ASD	Allowable Stress Design
ATC	Applied Technology Council
BCLDP	Battelle Columbus Laboratories Decommissioning Project
BOCA	Building Officials and Code Administrators International
BSSC	Building Seismic Safety Council
CDF	Cumulative Distribution Function
COV	Coefficient of Variation
D&D	Decontamination and Decommissioning
DOD	Department of Defense
DOE	Department of Energy
EPRI	Electric Power Research Institute
ES&H	Environment, Safety, and Health
FEMA	Federal Emergency Management Agency
FOSM	First-Order Second-Moment
FS	Factor of Safety
FT	Feet
GDC	General Design Criteria
HC	Hazard Category
LBS	Pounds
LLNL	Lawrence Livermore National Laboratory
LRFD	Load Reduction Factor Design
MPH	Miles per Hour
NA	Not Applicable
NPH	Natural Phenomena Hazard
NRC	Nuclear Regulatory Commission
PC	Performance Category
PDF	Probability Density Function
PSF	Pounds per Square Foot
SAR	Safety Analysis Report
SBC	Standard Building Code
SEAOC	Structural Engineers Association of California
SEN	Secretary of Energy Notice
SSCs	Structures, Systems, and Components
UBC	Uniform Building Code
USD	Ultimate Strength Design
YRS	Years

I. INTRODUCTION

Statement of the Problem

The current trend of downsizing within the federal government has resulted in the closure or partial closure of numerous United States Department of Energy (DOE) facilities. Once these facilities have ceased production, they typically enter into an environmental restoration phase due to the presence of, primarily, low level radioactive and/or chemical contamination. There are numerous methods that may be employed in the Decontamination and Decommissioning (D&D) of facilities contaminated with low level radioactive and/or chemical contamination (Kohli and Bessinger, 1992). Although these methods may differ radically for the given contamination, they all typically require the erection of temporary structures for operations such as contaminant containment, decontamination and size reduction operations (including ventilation), shower and change room facilities, and construction management operations (Decontamination Manual, 1992). Additional examples of temporary structures may include structures, systems, or components (SSCs) used during the construction of permanent facilities such as shoring/bracing, scaffolding, and work trailers.

There is at present a shortage of information pertaining to wind loads for temporary structures. The approaches taken to resist these loads by contractors for commercial and government construction projects vary greatly. On one end of the spectrum, contractors for commercial construction projects employ less conservative methods to resist wind loads. This approach has included, on occasion, the lack of design altogether. On the other end of the spectrum would be government, in particular the DOE, construction projects. For these projects, one might argue

that methods which are too conservative are employed. These methods include the design of temporary structures using the same wind design load criteria as used for permanent facilities whose design lives normally approach 50 years. In an attempt to address the inadequacy of information on wind design loads for short-lived temporary structures, the author examines, in this thesis, the approach to wind design of temporary structures for DOE D&D construction projects. The lives of these structures, as well as the hazard (facility usage) classifications, are comparable to the majority of commercial and government construction projects.

It is the policy of the DOE to design, construct, and operate facilities so as to protect workers, the general public and the environment from impacts of natural phenomena hazards on DOE facilities (DOE 5480.28, 1993). This document provides for a graded approach to natural phenomena hazard (NPH) mitigation. The design of DOE facilities (including NPH design) is to be done in accordance with the General Design Criteria (GDC) contained within DOE 6430.1A (1989). Implemented within the GDC (DOE 6430.1A, 1989) are the policies of DOE Orders on Environment, Safety, and Health (DOE 5480.1B, 1986) and, Safety Analysis and Review System (DOE 5481.1B, 1986). For facilities classified as nuclear, DOE additionally requires that these facilities be designed in accordance with nuclear safety policies contained within Nuclear Safety Analysis Reports (DOE 5480.23, 1992) and DOE Secretary of Energy Notice (SEN) on Nuclear Safety Policy (SEN-35-91, 1991). These documents require that DOE nuclear facilities be designed, evaluated, and constructed to meet NPH mitigation criteria and to do so in a cost effective manner. The goals of this document are similar to those of DOE 5480.28 (1993) in that the facility shall provide:

(1) A safe work environment;

(2) Protection against property loss or damage;

(3) For continued operation of essential facilities; and

(4) Protection of public health, property, and the environment against exposure to hazardous waste.

Figure 1-1 illustrates the relationships among the various DOE documents dealing with the mitigation of NPH. The Technical Standards, Codes, and Criteria are discussed in further detail in Section II of this document.

The current method of determining wind loads for DOE permanent facilities assumes a typical structure design life of approximately 50 years. This is consistent with national building codes including the Uniform Building Code (UBC, 1994), Standard Building Code (SBC, 1994), and National Building Code (BOCA, 1992). However, typical design lives for temporary structures erected for DOE D&D construction projects average less than five years. Since the GDC (6430.1A, 1989) does not address the structural design requirements for temporary structures, the wind loads used are the same as those for permanent structures. This methodology may be resulting in excessively conservative designs for these structures. For example, the probability of equaling or exceeding the design wind speed during a referenced period of time:

$$P = 1 - (1 - P_a)^n \qquad 1.1$$

where: P_a = the annual probability of exceedance

n = the reference period in years

Thus, if a design wind speed is based on a 50-year mean recurrence interval, $P_a = 0.02$, then the probability of equaling or exceeding the design wind speed in five years is approximately 10

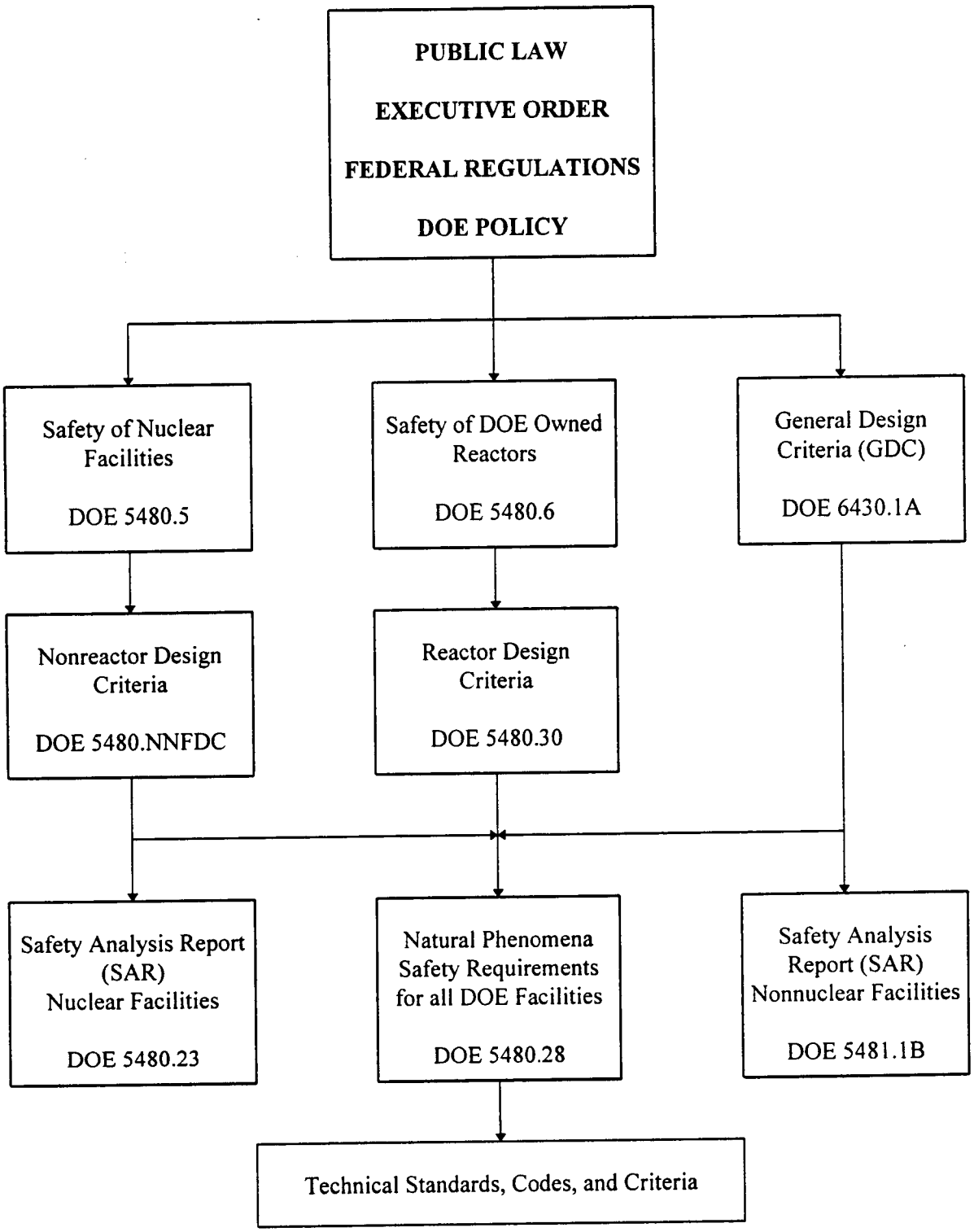

Figure 1-1. Relationships Among DOE Documents for NPH Mitigation

percent. In comparison, the probability of a permanent structure, one with a design life of 50 years, to experience a design wind speed based on a 50-year mean recurrence interval is approximately 64 percent.

Scope and Objective

The objective of this thesis is to explore and discuss the feasibility of using established wind design load criteria currently being used for permanent structures to design temporary structures for DOE D&D construction projects. In addition to exploring DOE documents, an extensive publications search was made of building codes, design guides, and publications. Also, guidelines and methods developed by other government agencies as well as other industries will be investigated for applicability. It is anticipated that the findings of this study will be applicable to temporary structures for other government and commercial construction projects with similar hazard (facility usage) classifications.

Temporary facilities erected for DOE D&D construction projects typically house operations which pose low environmental hazards to personnel and the environment. Because of this, the majority of these facilities are classified as PC-1 or PC-2 (DOE 5480.28, 1993), or General Use or Important/Low Hazard per criteria as defined in the now superseded document, UCRL-15910 (1988). Thus, only these classes of structures are investigated in this report. Structures with these classifications require that only a static analysis need be performed for the given wind loads.

It has been the author's experience that temporary facilities erected for construction projects have been dismantled within one to five years. Thus, this thesis is limited in scope to the analysis of structures with design lives of five years or less. This is consistent with current DOE D&D construction bid document processes which utilize a time-restricted performance-based approach. For these types of contracts, the adage "time is money" is especially taken to heart by construction contractors.

Given that the average design life for temporary structures erected for construction projects has ranged from one to five years, the author has opted not to address the use of lesser values for ground and roof snow loads. Also, in an attempt to limit the scope of this thesis, other natural phenomena events such as earthquake, flooding, and tornadoes have been neglected.

II. WIND LOADS ON PERMANENT STRUCTURES

Method for Determining Loads

ASCE-7 (1993) is a recognized method for determining wind loads on permanent structures including DOE PC-1 and PC-2 type structures (DOE 6430.1A, 1989), as shown in Figure 2-1. The velocity pressure formula used in determining design wind pressures and forces is:

$$q_z = 0.00256K_z(IV)^2 \qquad 2.1$$

where: q_z = the velocity pressure at height z (psf)

K_z = the exposure coefficient

I = the importance factor

V = the design wind speed (mph)

K_z is obtained from ASCE-7 (1993), Table 6 and accounts for the exposure category and structure height. Tables 2-1 and 2-2 provide the minimum wind design criteria and basic wind speeds for DOE sites, respectively. Once the velocity pressure is calculated, the design wind pressures and forces are calculated using ASCE-7 (1993) with appropriate gust factors.

Basis for Load Determination Method

A graded approach is used in performing safety analyses and evaluating DOE facilities for normal operating and accident conditions, including accidents caused by NPH events (DOE 5480.23, 1992). When using this graded approach for the purpose of NPH design and evaluation, DOE 5480.23 (1992) places the SSCs comprising a facility into one of five PCs. As noted in Section I, the majority of temporary structures erected for DOE D&D construction projects falls into the PC-1 or PC-2 categories. This is consistent with the NPH performance

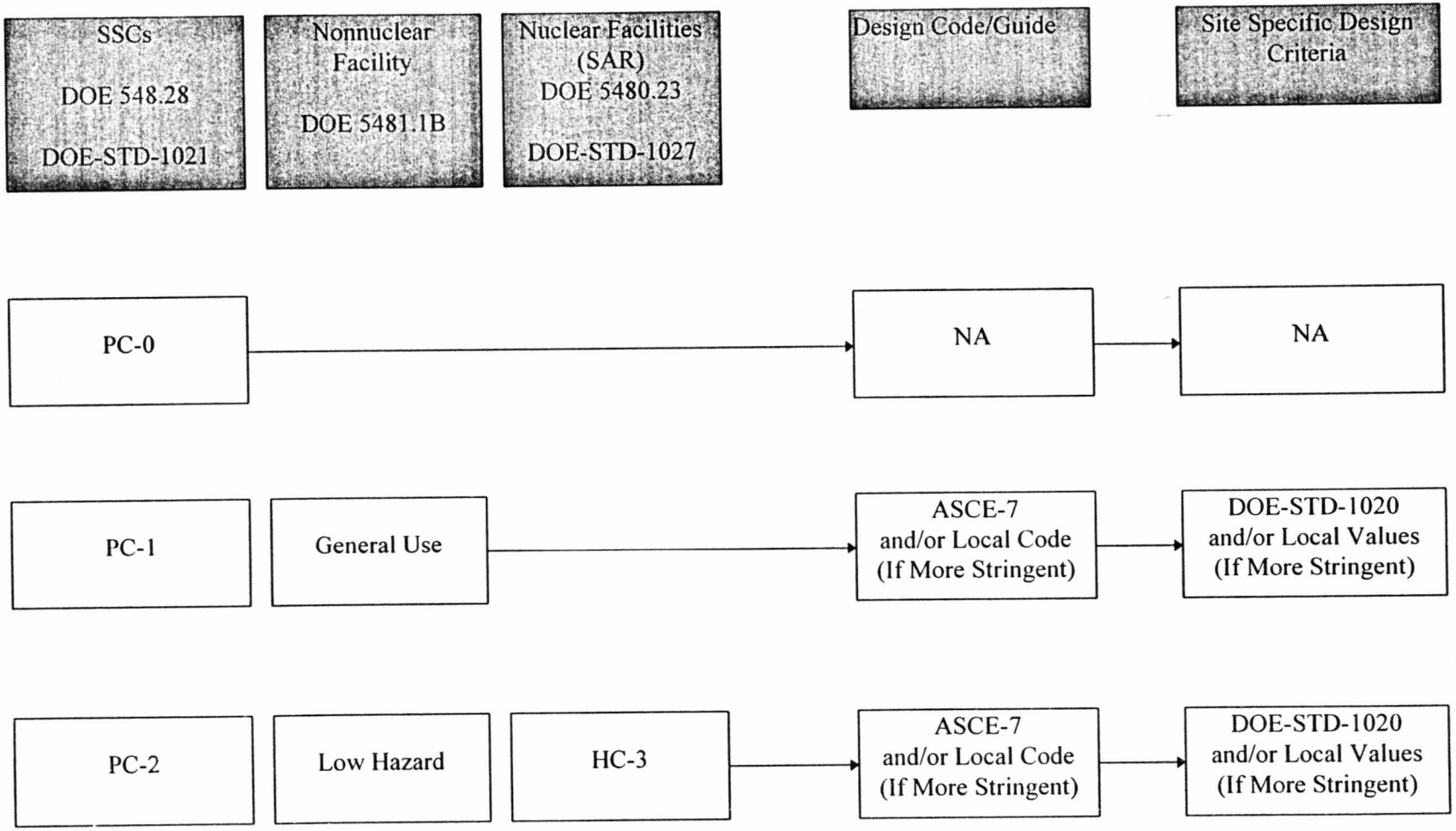

Figure 2-1. Wind Load Determination Method

Table 2-1. Summary of Minimum Wind Design Criteria

Performance Category	**PC-1**	**PC-2**
Hazard Annual Probability of Exceedance	$2x10^{-2}$	$2x10^{-2}$
Return Period	50 years	50 years
Importance Factor(1)	1.0	1.07
Missile Criteria	NA	NA

Source: DOE-STD-1020 (1994)

(1) Reference ASCE-7 (1993), Table 5

Table 2-2. Recommended Design Wind Speeds for DOE Sites (mph)

DOE Project Sites	Fastest-Mile Wind Speeds at 10m Height	
	PC-1	PC-2
Kansas City Plant, MO	72	72
Los Alamos National Laboratory, NM	77	77
Mound Laboratory, OH	73	73
Pantex Plant, TX	78	78
Rocky Flats Plant, CO	109	109
Sandia National Laboratories, NM	78	78
Sandia National Laboratories, CA	72	72
Pinellas Plant, FL	93	93
Argonne National Laboratory (East), IL	70(1)	70(1)
Argonne National Laboratory (West), ID	70(1)	70(1)
Brookhaven National Laboratory, NY	70(1)	70(1)
Princeton Plasma Physics Laboratory, NJ	70(1)	70(1)
Idaho National Engineering Laboratory, ID	70(1)	70(1)
Feed Materials Production Center, OH	70(1)	70(1)
Oak Ridge (X-10, K-25, Y-12), TN	70(1)	70(1)
Paducah Gaseous Diffusion Plant, KY	70(1)	70(1)
Portsmouth Gaseous Diffusion Plant, OH	70(1)	70(1)
Nevada Test Site, NV	72	72
Hanford Project Site, WA	70(1)	70(1)
Lawrence Berkeley Laboratory, CA	72	72
LLNL, CA	72	72
LLNL (Site 300), CA	80	80
Energy Technology & Engineering Ctr., CA	70(1)	70(1)
Stanford Linear Accelerator Center, CA	72	72
Savannah River Site, SC	78	78

Source: DOE-STD-1020 (1994)

(1) Minimum straight wind speed per ASCE-7 (1993).

categorization guidelines for SSCs defined in DOE-STD-1021 (1993). To ensure that the appropriate level of conservatism introduced in the NPH design process is appropriate for facility occupancy and other characteristics such as importance, cost, and hazards to humans and the environment. DOE-STD-1020 (1994), and formerly UCRL-15910 (1990), uses the performance goal as the target design parameter. For SSCs, this performance goal is defined as the annual frequency of probable failure to perform or annual probability of exceedance of acceptable behavior limits.

Qualitative performance goals for behavior (i.e., maintain structural integrity, maintain ability to function, and maintain confinement of hazardous materials) and quantitative target probabilistic performance goals for PC-0 through PC-4 structures are established in DOE 5480.28 (1993). PC-0 structures need not consider NPH design loads and include structures such as lightweight nonmission dependent equipment, furniture, etc.. NPH criteria for PC-3 and PC-4 SSCs are not discussed since the majority of temporary structures do not fall into these higher hazard categories. Table 2-3 relates the DOE PC-1 and PC-2 classifications to the UBC (1994) as well as provides general facility descriptions and performance goals. The performance annual probability of exceedance for PC-1 SSCs is consistent with model building codes such as the UBC (1994), SBC (1994), and BOCA (1992) for wind loadings. The primary concern is not the repair and/or replacement of the structure, but rather the prevention of major structural failure and the maintaining of life safety under severe wind loads. For PC-2 structures, the performance goal is established such that the capacity to function is provided in addition to occupant safety.

Table 2-3. PC Relationships and Performance Goal Criteria

Source	**SSC Category**	
DOE-STD-1020	PC-1	PC-2
Uniform Building Code	General Facilities	Essential Facilities
Performance Goal Annual Probability of Exceedance	$1x10^{-3}$ (to the onset of significant damage to SSCs)	$5x10^{-4}$ (to the onset of significant damage to SSCs)
General Facility Description		
DOE-STD-1020	Administrative Buildings Storage Facilities Cafeterias Maintenance/Repair	Laboratories Production Facilities Computer Centers Emergency Handling Hazard Recovery
Uniform Building Code	Standard Occupancy (not listed for PC-2)	Hospitals Fire/Police Stations Emergency Centers

Relatively minor structural damage is allowed which results in minimal interruption to operations and that is easily and quickly repaired following an NPH event (DOE-STD-1020, 1994).

The performance goals noted in Table 2-3 relate to the probability of onset of damage to the SSC subjected to an NPH event, but do not include the consequences beyond SSC damage. The performance goal annual probability of exceedance is a combined function of the annual probability of exceedance of the event, design factors of safety, as well as any other conservative assumptions. DOE-STD-1020 (1994) defines the risk reduction factor, R_R, as the ratio of the hazard annual probability of exceedance to the performance goal annual probability of exceedance. R_R establishes the level of conservatism used in the design process. Values of R_R greater than 1.0 would indicate a conservative approach to design while values less than 1.0 indicate a less than conservative approach. If the value of $R_R = 1.0$, this indicates that the performance goal and hazard annual probabilities are the same, thus theoretically introducing no conservatism in the design approach. However, conservatism is still present given that the allowable stresses are some fraction (factor of safety) of the ultimate stresses.

Figure 2-2 illustrates the parameters defining both the probabilistic and deterministic phases for meeting the prescribed performance goals. As illustrated in the figure, reasonable NPH loads are selected in the probabilistic phase. The term "reasonable" in this context is synonymous with slightly conservative. Additional conservatism is then added in the deterministic phase of design. For example, additional conservatism is obtained by specifying factors of safety for Allowable Stress Design (ASD) and load factors for Load Reduction Factor Design (LRFD) for

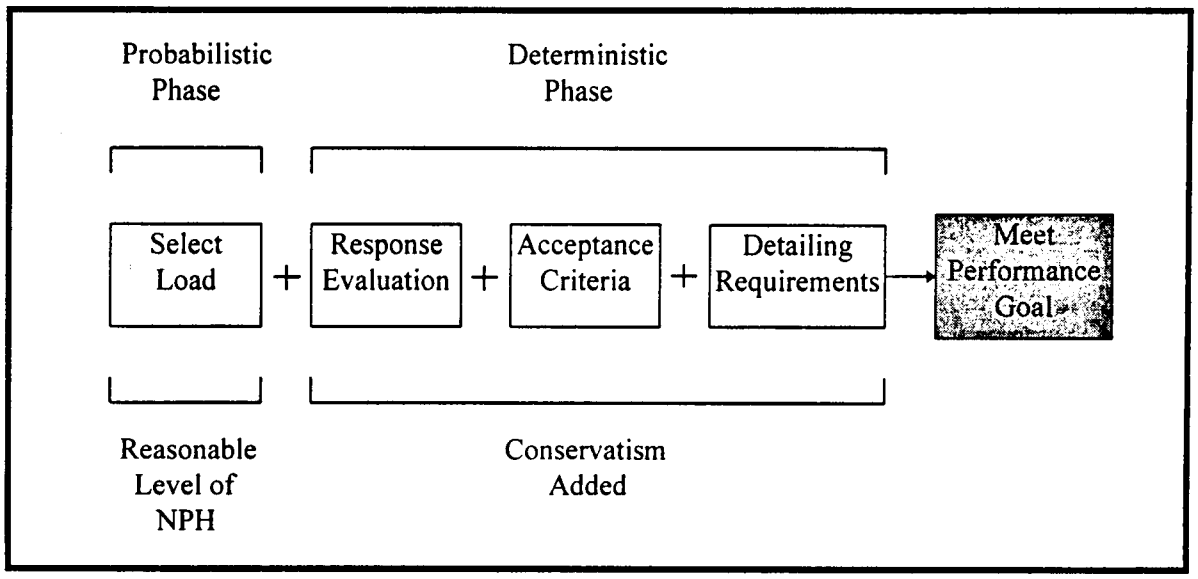

Figure 2-2. Probabilistic and Deterministic Phases to Meet Performance Goal

steel design.

DOE-STD-1020 (1994) mandates a uniform approach to wind design based on the provisions of ASCE-7 (1993). Designers are referenced to Figure 1 in ASCE-7 (1993) in order to determine design wind speeds. These fastest-mile design wind speeds are based on a return period of 50 years, or hazard probability of exceedance of $2x10^{-2}$. Table 2-4 provides the target ratios of hazard probabilities to performance goal probabilities for PC-1 and PC-2 SSCs.

The fastest-mile design wind speeds used to develop Figure 1 in ASCE-7 (1993) were obtained from data collected from 129 weather stations in the contiguous United States and summarized by Simiu et al. (1979). These fastest-mile wind speeds were obtained at a height of 33 feet (10 meters) above ground level with terrain conditions similar to ASCE-7 (1993) exposure category C. Simiu et al. (1979) estimated the fastest-mile wind speeds assuming an Extreme Value Type I distribution for return periods from 2 to 1,000,000 years. For example, Table 2-5 contains fastest-mile wind speed information developed for Knoxville, Tennessee and is based on data obtained from 1942 through 1974. This information was based on 33 sample observations, with a sample mean of 48.84 mph and sample standard deviation of 6.88 mph. The sample minimum and maximum values were 37.22 mph and 65.91 mph, respectively.

Table 2-4. Target Ratios of Wind Hazard Probabilities to Performance Goal Probabilities

Performance Category	Performance Goals (P_F)	Wind Hazard Probability (P_H)	Ratio of Hazard to Performance Probability (R_R)
PC-1	$1x10^{-3}$	$2x10^{-2}$	20
PC-2	$5x10^{-4}$	$1x10^{-2}$ (1)	20

Source: DOE-STD-1020 (1994)

(1) 2x10-2 with I=1.07

Table 2-5. Fastest-Mile Wind Speed Information for Knoxville, TN

Return Period (in years)	Predicted Extreme Wind Based on Extreme Value Type I Distribution	Estimated Std. Deviation Sampling Error Cramer-Rao	Estimated Std. Deviation Sampling Error Method of Moments
2.0	47.78	1.10	1.10
3.0	50.79	1.34	1.40
4.0	52.71	1.53	1.65
5.0	54.13	1.68	1.85
6.0	55.26	1.81	2.02
7.0	56.20	1.91	2.16
8.0	57.00	2.00	2.29
9.0	57.71	2.09	2.40
10.0	58.33	2.16	2.50
20.0	62.36	2.64	3.16
30.0	64.68	2.92	3.55
33.0	65.22	2.99	3.64
40.0	66.31	3.13	3.82
50.0	67.57	3.28	4.03
60.0	68.60	3.41	4.21
70.0	69.47	3.52	4.36
80.0	70.23	3.62	4.49
90.0	70.89	3.70	4.60
100.0	71.48	3.77	4.70
200.0	75.38	4.27	5.37
300.0	77.65	4.56	5.76
400.0	79.26	4.76	6.04
500.0	80.52	4.92	6.25
600.0	81.54	5.05	6.43
700.0	82.40	5.16	6.58
800.0	83.15	5.26	6.71
900.0	83.81	5.34	6.82
1000.0	84.40	5.42	6.92
2000.0	88.28	5.92	7.60
3000.0	90.55	6.21	7.99
4000.0	92.16	6.41	8.27
5000.0	93.41	6.58	8.49
6000.0	94.43	6.71	8.66
7000.0	95.29	6.82	8.81
8000.0	96.04	6.91	8.94
9000.0	96.70	7.00	9.06
10000.0	97.29	7.08	9.16
50000.0	106.30	8.24	10.73
100000.0	110.18	8.74	11.41
500000.0	119.20	9.91	12.98
1000000.0	123.08	10.41	13.65

Source: Simiu et al. (1979)

III. WIND LOADS FOR TEMPORARY STRUCTURES

This chapter focuses on the applicability of probabilities of failure for permanent DOE structures to temporary short-lived structures under maximum wind load conditions. Probabilities of failure for permanent structures exposed to maximum wind speeds with annual mean return periods of 50 years (0.02 probability of exceedance) are determined. These are then set as target probabilities of failure for temporary short-lived structures. Consideration is given to maximum wind speed hazard exceedance probabilities, maximum wind loads, and structural resistance for given types of construction.

In determining target probabilities of failure for permanent SSCs, the following equation is used:

$$P_f = 1 - \Phi(\beta) \qquad 3.1$$

where: P_f = the probability of failure

Φ = the standard normal cumulative distribution function

β = the reliability index

The reliability index, β, used in Equation 3.1 is a function of the type of loading (i.e. demand) on the SSC, and the structural resistance (i.e. supply). As noted, maximum wind loads will be considered based on maximum (fastest-mile) wind speeds. Structural resistance varies for given types of construction. Most temporary construction consists of lightweight SSCs such as work-trailers, pole-barns, scaffolding, and shoring/bracing. The primary materials used in these SSCs are steel, aluminum, and wood. Thus, only the structural resistance provided by these materials is considered.

Maximum Wind Speed Hazard Exceedance Probabilities

The design wind speed map (Figure 1) in ASCE-7 (1993) was developed using fastest-mile wind speeds obtained at 33 feet (10 meters) above adjacent ground for exposure category C terrain conditions. These values are based on an annual mean return period of 50 years (0.02 probability of exceedance). These values are also tabulated in Table C7 of the Commentary (Section 6) in ASCE-7 (1993) for various cities throughout the United States. Additionally, the maximum fastest-mile wind speeds are computed for annual mean return periods of 25 and 100 years, with annual probabilities of exceedance of 0.04 and 0.01, respectively. These values correspond to the maximum 50-year mean recurrence wind speed multiplied by an importance factor, I, of 0.95 and 1.07, respectively. Importance factors correspond to structure categories and are intended to account for different desired reliability levels (Bennett, 1986). Table 3-1 lists the structure categories and corresponding importance factors. Note that only values of I for locations greater than 100 miles from hurricane oceanline have been provided.

DOE has tabulated values for recommended basic wind speeds for DOE sites in DOE-STD-1020 (1994). Maximum fastest-mile design wind speeds obtained from site-specific studies were used in this table and were based on data obtained by Coats and Murray (UCRL-53526, 1985) for annual probabilities of exceedance of 0.02, which is consistent with the basic wind speed map in ASCE-7 (1993). Where site-specific values obtained were less than the ASCE-7 (1993) recommended minimum of 70 mph, 70 mph was used in the table.

Table 3-1. Structure Categories and Importance Factors

Category	Nature of Occupancy	Importance Factor
I	All buildings and structures except those listed below	1.00
II	Buildings and structures where the primary occupancy is one in which more than 300 people congregate in one area	1.07
III	Buildings and structures designated as essential facilities, including, but not limited to: Hospitals and other medical facilities having surgery or emergency treatment areas Fire or rescue and police stations Structures and equipment in government Communications centers and other facilities required for emergency response Power stations and other utilities required in an emergency Structures having critical national defense capabilities Designated shelters for hurricanes	1.07
IV	Buildings and structures that represent a low hazard to human life in the event of failure, such as agricultural buildings, certain temporary facilities, and minor storage facilities	0.95

Source: ASCE-7 (1993)

The extreme fastest-mile wind speeds contained in the ASCE-7 (1993) basic wind speed map (Figure 1) and Table C7 are based on data collected by Simiu et al. (1979) for 129 weather stations in the contiguous United States. These data were analyzed using an Extreme Value Type I distribution. Fastest-mile wind speeds listed in Table C7 (ASCE-7, 1993) are based on data obtained over a minimum of 10 continuous years to enhance reliability. These criteria are consistent with the criteria used by DOE (DOE-STD-1022, 1994) for site characterization of NPH loads.

In determining the probability that a structure will experience a maximum wind speed at least once in its design life, the following equations are applicable. The probability that the maximum wind speed will not be exceeded in any year is:

$$P\text{ (not exceeded in a year)} = 1 - P_E = 1 - 1/T_R \qquad 3.3$$

where: P_E = the probability of exceedance

T_R = the return period

The probability that the maximum wind speed will not be exceeded N years in a row is:

$$P\text{ (not exceeded in N successive years)} = (1 - P_E)^N \qquad 3.4$$

where: N = the design life of the structure

The probability that the maximum wind speed will be exceeded at least once in the design life of the structure is:

$$P\text{ (exceeded within the N years)} = 1 - (1 - P_E)^N \qquad 3.5$$

Using Equation 3.5, the probability that the maximum wind speed will be exceeded in the design life of a structure for varying annual exceedance probabilities can be calculated. Some of these

probabilities are shown in Table 3-2. As can be seen in this table, the probability that a structure with a design life of 50 years (permanent structure) will experience a maximum wind speed with an annual exceedance probability of 0.02 (50-year return period) is 0.636, or approximately 64 percent. However, the probability that a temporary structure with a design life of 5 years will experience a 50-year maximum wind speed is 0.096, or 9.6 percent.

If the probability that a permanent structure will experience a maximum wind speed at an annual exceedance probability of 0.02 is set as the target allowable hazard exceedance probability, then maximum wind speed return periods for various short-lived temporary structures may be selected which yield probabilities less than or equal to the target value. Maximum wind speeds at an annual exceedance probability of 0.02 correspond to normal (PC-1) structures. For a permanent (50-year design life) structure, this target probability is 64 percent with an annual exceedance probabilities of 0.02.

Yearly exceedance probabilities and maximum wind speed return periods are provided in Table 3-3 for temporary structures with design lives from 2 to 5 years. These yearly exceedance probabilities and return periods yield probabilities of wind speed exceedance over design life near the target value of 64 percent.

Structural Reliability

Wind Loads

Table 3-2. Probabilities of Maximum Wind Speed Exceedance Over Structure Design Life

Exceedance Probability	0.50	0.33	0.25	0.20	0.10	0.04	0.02	0.01
Return Period (yrs)	2	3	4	5	10	25	50	100
Design Life (yrs)								
2	0.750	0.556	0.438	0.360	0.190	0.078	0.040	0.020
3	0.875	0.704	0.578	0.488	0.271	0.115	0.059	0.030
4	0.938	0.802	0.684	0.590	0.344	0.151	0.078	0.039
5	0.969	0.868	0.763	0.672	0.410	0.185	0.096	0.049
6	0.984	0.912	0.822	0.738	0.469	0.217	0.114	0.059
7	0.992	0.942	0.867	0.790	0.522	0.249	0.132	0.068
8	0.996	0.961	0.900	0.832	0.570	0.279	0.149	0.077
9	0.998	0.974	0.925	0.866	0.613	0.308	0.166	0.087
10	0.999	0.983	0.944	0.893	0.651	0.335	0.183	0.096
20	0.999	0.999	0.997	0.989	0.878	0.558	0.332	0.182
30	0.999	0.999	0.999	0.999	0.956	0.706	0.455	0.260
40	0.999	0.999	0.999	0.999	0.985	0.805	0.554	0.331
50	0.999	0.999	0.999	0.999	0.995	0.870	0.636	0.395
75	0.999	0.999	0.999	0.999	0.999	0.953	0.780	0.529
100	0.999	0.999	0.999	0.999	0.999	0.983	0.867	0.634

Table 3-3. Target Wind Speed Hazard Exceedance Probabilities for Temporary Structures

Design Life (yrs)	**Exceedance Probability**	**Return Period (yrs)**	**Probability of Wind Speed Exceedance Over Design Life**
2	0.33	3	0.556
3	0.25	4	0.578
4	0.20	5	0.590
5	0.10	10	0.410(1)

(1) Conservatively chose 10-year return period (0.10 exceedance probability), however a 5-year return period (0.20 exceedance probability) yields a 0.672 probability of maximum wind speed exceedance over the structure design life and is closer to the target value 0.636.

It has been determined that the Poisson modeling process and selection of the Extreme Value Type I cumulative distribution function (CDF) most accurately predicts annual maximum (fastest-mile) wind speeds (Belk, 1988). An Extreme Value Type I CDF has the form:

$$F_x(x) = \exp\left[-e^{-\alpha(x-u)}\right] \qquad 3.6$$

where: $F_x(x)$ = the CDF for maximum wind speed

$$\alpha = \frac{\pi}{\sigma_x\sqrt{6}} \qquad 3.7$$

$$u = m_x - \frac{0.577215}{\alpha} \qquad 3.8$$

σ_x = the standard deviation

m_x = the mean

The variables, α and u, described in Equations 3.7 and 3.8, are the scale and location parameters, respectively. They may additionally be described as follows:

α = the inverse measure of dispersion of X

u = the characteristic largest value of the initial variate X

The corresponding probability density function (PDF) is:

$$f_x(x) = \alpha\, e^{-\alpha(x-u)} \exp\left[-e^{-\alpha(x-u)}\right] \qquad 3.9$$

Statistical annual maximum (fastest-mile) wind speed data have been developed by Simiu et al. (1979) for 129 cities in the US. These statistics were obtained using an Extreme Value Type I CDF and are based on 10 to 40 years of climatological records. Analyses of these data show that the mean and coefficient of variation (COV) are dependent on geographical locations (Ellingwood et al., 1980).

In order to determine σ_x and m_x values for 2-, 3-, 4-, and 5-year return periods, the Method of Moments was used to analyze the corrected fastest-mile wind speeds obtained by Simiu, et al. (1979). Once σ_x and m_x are obtained, α and u values may be calculated using Equations 3.7 and 3.8. Values for σ_x, m_x, α and u were computed for those cities located at or near DOE project sites. These sites and corresponding σ_x, m_x, α and u values are presented in Table 3-4.

Annual data have been developed for those cities listed in Table 3-4 using the corrected fastest-mile wind speeds obtained by Simiu, et al. (1979). Using the predicted 50-year maximum wind speed, the 50-year mean wind speed, m_{50}, can be solved as follows (Ellingwood, et al., 1980):

$$m_{50} = m_{ann}(1+\sqrt{6}/\pi)\,COV_{ann}\ln 50 \qquad 3.10$$

where: m_{50} = the mean 50-year wind speed

m_{ann} = the mean annual wind speed

COV_{ann} = the COV of the annual wind speed data

$$= \sigma_x/m_x \qquad 3.11$$

The parameters, u and α, for 50-year return periods may be determined using annual data and by setting the CDF, $F_x(x)$, equal to 1 - 1/50, or 0.98, as follows:

$$F_x(x) = 0.98 = \exp[-e^{-\alpha(u_{50} - u)}] \qquad 3.12$$

where: α = the annual α value

u = the annual u value

u_{50} = the 50-year u value

Table 3-4. Maximum Wind Speed Data for Various Return Periods

Cities	2-Year Return Period				3-Year Return Period				4-Year Return Period				5-Year Return Period			
	m	σ	α	u	m	σ	α	u	m	σ	α	u	m	σ	α	u
Knoxville, TN	52.250	6.403	0.2003	49.368	54.455	6.170	0.2079	51.678	56.125	5.384	0.2382	53.702	57.167	5.636	0.2276	54.630
Kansas City, MO	53.947	6.113	0.2098	51.196	56.750	5.302	0.2419	54.364	57.444	5.354	0.2395	55.035	57.286	6.055	0.2118	54.561
Albuquerque, NM	60.727	8.142	0.1575	57.063	63.867	8.210	0.1562	60.172	65.546	8.836	0.1452	61.569	65.889	8.069	0.1590	62.257
Dayton, OH	57.177	7.126	0.1800	53.969	59.909	5.787	0.2216	57.305	61.000	5.292	0.2424	58.619	60.571	6.705	0.1913	57.554
Denver, CO	52.000	4.019	0.3191	50.191	52.778	4.466	0.2872	50.768	53.333	4.179	0.3069	51.452	54.000	4.301	0.2982	52.064
Sacramento, CA	49.857	10.748	0.1193	45.020	51.889	9.675	0.1326	47.535	52.286	11.161	0.1149	47.263	52.400	10.900	0.1177	47.495
Jacksonville, FL	53.643	11.291	0.1136	48.562	57.444	11.780	0.1089	52.143	59.571	12.581	0.1019	53.909	61.200	12.637	0.1015	55.513
Chicago, IL	49.000	4.679	0.2742	46.895	51.000	4.191	0.3060	49.296	52.000	4.598	0.2789	49.931	51.000	4.860	0.2639	49.241
Rochester, NY	56.333	5.145	0.2493	54.018	58.583	4.582	0.2799	56.521	59.556	3.609	0.3553	57.931	60.000	3.742	0.3428	58.316
Pocatello, ID	57.684	6.228	0.2059	54.881	59.750	5.277	0.2431	57.375	61.333	4.472	0.2868	59.321	61.286	5.648	0.2271	58.744
Las Vegas, NV	59.500	6.686	0.1918	56.491	62.250	6.602	0.1943	59.279	64.667	5.508	0.2329	62.188	64.500	7.778	0.1649	60.999
Spokane, WA	50.222	5.976	0.2146	47.533	53.417	4.699	0.2729	51.302	54.333	4.637	0.2766	52.247	55.143	4.947	0.2592	52.916
Savannah, GA	50.938	10.786	0.1189	46.083	51.900	12.360	0.1038	46.337	55.125	12.699	0.1010	49.410	56.500	13.635	0.0941	50.364

Alpha for the 50-year wind, α_{50}, may then be solved by manipulating Equation 3.8 such that:

$$\alpha_{50} = \frac{0.577215}{m_{50} - u_{50}} \qquad 3.13$$

The standard deviation for the 50-year wind, σ_{50}, is then determined by:

$$\sigma_{50} = \frac{\pi}{\alpha_{50}\sqrt{6}} \qquad 3.14$$

Statistical data for 50-year predicted maximum wind speeds has been calculated for those cities listed in Table 3-4. This, along with the annual data, is presented in Table 3-5.

As noted, the reliability index, β, is a function of the type of loading on the SSC. Wind loads are derived using statistical data on maximum wind speeds, importance parameters, pressure coefficients, exposure data, and gust factors. It is assumed that the values for importance factors will be identical for both permanent and temporary structures. However, the exposure, gust, and pressure factors contribute to the overall variability of wind loads and need to be analyzed.

ASCE-7 (1993) defines wind load in lbs as:

$$W = qGC_P \qquad 3.15$$

where: q = the velocity pressure (psf)

G = the gust factor

C_P = the pressure coefficient

Table 3-5. Maximum Wind Speed Data for Annual and 50-Year Return Periods

Cities	Predicted Extreme Wind (1)	Annual Data				50-Year Return Period			
		m	σ	α	u	m	σ	α	u
Knoxville, TN	67.57	48.818	6.7844	0.1890	45.765	69.512	6.8414	0.1875	66.433
Kansas City, MO	71.95	50.546	7.8458	0.1635	47.015	74.477	7.9746	0.1608	70.888
Albuquerque, NM	78.17	57.222	7.7544	0.1654	53.732	80.875	7.8524	0.1633	77.341
Dayton, OH	74.33	53.657	7.6001	0.1688	50.237	76.839	7.7035	0.1665	73.372
Denver, CO	62.28	49.111	4.6105	0.2782	47.036	63.174	4.6750	0.2743	61.070
Sacramento, CA	73.99	46.104	10.2063	0.1257	41.510	77.235	10.3721	0.1237	72.567
Jacksonville, FL	76.46	48.714	10.0880	0.1271	44.174	79.485	10.2299	0.1254	74.881
Chicago, IL	60.08	47.029	4.8294	0.2656	44.855	61.759	4.8461	0.2647	59.578
Rochester, NY	67.71	53.324	5.2231	0.2456	50.974	69.256	5.2905	0.2424	66.875
Pocatello, ID	71.56	53.553	6.9153	0.1855	50.440	74.646	6.9970	0.1833	71.497
Las Vegas, NV	75.03	54.769	7.0375	0.1823	51.602	76.235	7.1747	0.1788	73.006
Spokane, WA	65.23	47.703	6.5271	0.1965	44.765	67.612	6.5926	0.1945	64.645
Savannah, GA	73.70	47.656	9.5634	0.1341	43.352	76.827	9.6900	0.1324	72.466

(1) Determined by Simiu et al. (1979) using Extreme Type I CDF

The velocity pressure, q, is determined using the formula:

$$q = 0.00256K(IV)^2 \qquad 3.16$$

where: K = the exposure coefficient

I = the importance factor

V = the maximum (fastest-mile) design wind speed (mph)

As can be seen, wind load is proportional to the wind speed squared. Although the maximum wind speed may be efficiently modeled using an Extreme Type I distribution, it is not readily apparent which probability distribution best models wind load. The square of an Extreme Type I distribution may not follow an Extreme Type I distribution.

Ellingwood et al. (1980) assumed that G, C_P, and K may be described by a normal distribution function. The CDF of wind load, F_W, was then determined, using Monte Carlo simulation and numerical integration, to closely follow an Extreme Type I distribution.

Statistical data for G, C_P, and K may be obtained from ASCE-7 (1993). The gust factor, G, for a given height, z, is defined as:

$$G_z = 0.65 = 3.65T_z \qquad 3.17$$

where: $T_z = \dfrac{2.35(D_o)^{1/2}}{(z/30)^{1/a}}$ 3.18

D_o = the surface drag coefficient

z = the mean height of the structure

a = the power law coefficient

The exposure coefficient, K, for a given height, z, may be obtained using the following:

For $z < 15$ feet,

$$K_z = 2.58(15/z_g)^{2/a} \qquad 3.19$$

where: z_g = the gradient height

For $z \geq 15$ feet,

$$K_z = 2.58(z/z_g)^{2/a} \qquad 3.20$$

Pressure coefficients, C_P, have been determined using the latest boundary-layer wind-tunnel and full-scale tests. The mean values for G, C_P, and K are assumed to be identical to those values listed in ASCE-7 (1993). Thus, the ratio of the mean values to the nominal values is 1.0. Additional statistical data for G, C_P, and K factors are presented in Table 3.6.

In lieu of assuming that the CDF of wind load, F_W, follows an Extreme Type I distribution, a classical First-Order Second-Moment (FOSM) analysis was performed. This analysis takes into account the nonlinear failure surface generated by an Extreme Type I distribution of the wind speed, Normal distribution of G, C_P, and K, and distribution function of the structural resistance, R. This analysis is discussed in further detail later in this section.

Structural Resistance

The primary materials used for temporary structures include structural (hot-rolled) steel, cold-formed steel, aluminum, and timber. Structural resistance, R, varies depending on material strength, geometric cross-section, load type, and failure mode.

Table 3-6. Statistical Data for G, K, and C_P Factors

Factor/ Coefficient	Ratio of Mean to Nominal	COV	σ	Distribution Function
G	1.0	0.11	0.11	Normal
K	1.0	0.16	0.16	Normal
C_P	1.0	0.12	0.12	Normal

Source: Ellingwood et al. (1980)

For metal structures (hot-rolled steel, cold-formed steel, and aluminum), the nominal resistance, R_n, to wind load is defined as:

$$R_n = FS\ F_W \qquad 3.21$$

where: FS = the factor of safety

F_W = the wind load

The mean structural resistance, R_m, to wind load is determined by:

$$R_m = R_n\ (R_m/R_n) \qquad 3.22$$

where: R_m/R_n = the ratio of the mean yield resistance to nominal resistance

Ellingwood et al. (1980) determined values for R_m/R_n and COV for metal structures for various failure modes. From R_m and COV values, standard deviations for resistance, σ_r, may be calculated using the following:

$$\sigma_r = R_m\ COV \qquad 3.23$$

Factors of safety used in this study are based on ASD for yielding of tension members on the gross cross-section. Yielding of tension members was chosen as the representative failure mode given the typical lateral nature of wind loading. This failure mode is directly applicable to the design of tension members used for bracing and guying. Statistical values obtained for this failure mode should be comparable to those obtained for flexural and compressive failure modes. Factors of safety for metal structures used in this study are listed below:

Structural (Hot-Rolled) Steel	FS = 1.67
Cold-Formed Steel	FS = 1.67
Aluminum	FS = 1.65

For this thesis, the design of metal structures is based on the following standards. For structural (hot-rolled) steel, the provisions of the American Institute of Steel Construction (AISC, 1989) is

used. The design of cold-formed steel structures is based on the American Iron and Steel Institute (AISI, 1979). The design of aluminum structures is governed by the Aluminum Association (AA, 1980).

It has been determined that the resistance of metal structures is most accurately described using a Lognormal CDF (Ellingwood et al., 1980 and Galambos et al., 1982). A Lognormal CDF has the form:

$$F_x(x) = F_U\,[1/\zeta\,(\ln x - \lambda)] \qquad 3.24$$

where: $\zeta = \sqrt{\sigma^2_{\ln x}}$ 3.25

$\lambda = \ln m_x - \zeta^2/2$ 3.26

Ratios of R_m/R_n and COV values determined by Ellingwood et al. (1980) and used in this study are:

Structural (Hot-Rolled) Steel	$R_m/R_n = 1.05$	COV = 0.11
Cold-Formed Steel	$R_m/R_n = 1.10$	COV = 0.11
Aluminum	$R_m/R_n = 1.10$	COV = 0.08

Mean resistance, R_m, and standard deviations, σ_r, for metal structures are calculated using Equations 3.21, 3.22, and 3.23. These statistics are provided in Table 3-7 and are based on wind loads using the following wind speed criteria:

(1) For cities whose design wind speed illustrated in Figure 1 (ASCE-7, 1993) approximately concurred with the 50-year extreme fastest-mile wind speed predicted by Simiu et al. (1979), the predicted value is used.

(2) For cities whose design wind speed illustrated in Figure 1 (ASCE-7, 1993) was greater than the 50-year extreme fastest-mile wind speed predicted by Simiu et al. (1979), the value from Figure 1 is used.

Statistical data on the resistance of timber structures is more difficult to obtain than for metal structures. This is in part due to the material properties of wood and in the current ASD approach to wood design. Unlike steel and aluminum, there is no yield plateau on the stress-strain diagram for wood. Similar to concrete, the stress increases nonlinearly to the ultimate stress, F_u, and then drops off rapidly. However, unlike the ultimate strength design (USD) method employed in concrete design, load factors are not utilized. Thus, an engineer does not utilize some fraction of F_u (i.e., similar to $0.85f_c'$ used in the USD method for concrete design) as the allowable stress, F_a, but rather may use an F_a which typically ranges from 1/5 to 1/8 of F_u (Breyer, 1988, Nunnally, 1987, and USDA Handbook 72, 1987). This F_a, in turn, is multiplied by factors to account for items such as load duration, moisture content, temperature, size, form, etc. Most of these factors do not vary greatly from 1.0; however, other factors may allow the engineer to design to up to 1.33 times F_a. For this thesis, the author has chosen to limit the actual stress, f_a, to F_a, thus limiting the FS to 1.0. This does not imply lack of conservatism. Conversely, given that basic F_a values range from 1/5 to 1/8 F_u values, one could argue that there is excessive conservatism in current wood design practice.

Table 3-7. Resistance Statistics for Permanent Metal Structures

City	Material	Mean Resistance R_m	Standard Deviation σ_r
Knoxville, TN	Structural Steel	8575.00	943.25
	Cold-Formed Steel	8983.33	988.17
	Aluminum	8893.50	711.48
Kansas City, MO	Structural Steel	10108.00	1111.88
	Cold-Formed Steel	10589.33	1164.83
	Aluminum	10483.44	838.68
Albuquerque, NM	Structural Steel	10693.46	1176.28
	Cold-Formed Steel	11202.67	1232.29
	Aluminum	11090.65	887.25
Dayton, OH	Structural Steel	9668.66	1063.55
	Cold-Formed Steel	10129.07	1114.20
	Aluminum	10027.78	802.22
Denver, CO	Structural Steel	8575.00	943.25
	Cold-Formed Steel	8983.33	988.17
	Aluminum	8893.50	711.48
Sacramento, CA	Structural Steel	9580.41	1053.85
	Cold-Formed Steel	10036.62	1104.03
	Aluminum	9936.25	794.90
Jacksonville, FL	Structural Steel	14812.00	1629.32
	Cold-Formed Steel	15517.33	1706.91
	Aluminum	15362.16	1228.97
Chicago, IL	Structural Steel	8575.00	943.25
	Cold-Formed Steel	8983.33	988.17
	Aluminum	8893.50	711.48
Rochester, NY	Structural Steel	8575.00	943.25
	Cold-Formed Steel	8983.33	988.17
	Aluminum	8893.50	711.48
Pocatello, ID	Structural Steel	8961.46	985.76
	Cold-Formed Steel	9388.20	1032.70
	Aluminum	9294.31	743.55
Las Vegas, NV	Structural Steel	9851.63	1083.68
	Cold-Formed Steel	10320.75	1135.28
	Aluminum	10217.54	817.40
Spokane, WA	Structural Steel	8575.00	943.25
	Cold-Formed Steel	8983.33	988.17
	Aluminum	8893.50	711.48
Savannah, GA	Structural Steel	14175.00	1559.25
	Cold-Formed Steel	14850.00	1633.50
	Aluminum	14701.50	1176.12

For timber structures, it has been determined that a Weibull CDF most accurately predicts the structural resistance (Ellingwood et al., 1980 and Galambos et al., 1982). The Weibull CDF may be described by (Ang and Tang, 1984):

$$F_x(x) = 1 - \exp\{-[(x - \varepsilon)/(w-\varepsilon)^k]\} \qquad 3.27$$

where: w = the upperbound value

ε = the lowerbound value

In determining the structural resistance of a member in tension, similar to the methodology used for metal structures, where $\varepsilon = 0$, the CDF becomes:

$$F_x(x) = 1 - e^{-(x/w)k} \qquad 3.28$$

Mean resistance, R_m, and standard deviations for resistance, σ_r, for timber structures are calculated using Equations 3.21, 3.22, and 3.23. The values of R_m/R_n and COV determined by Ellingwood et al. (1980) and used to determine R_m and σ_r are 2.69 and 0.16, respectively. These statistics are provided in Table 3-8 and are based on the same wind speed/load criteria used for metal structures.

Safety (Reliability) Levels for Permanent Structures

The nonlinear performance function, g(X), generated by the Extreme Type I wind speed, Normal G, C_P, and K factors, and the structural resistance distribution function (Lognormal for metal structures and Weibull for timber structures) is evaluated using classical FOSM analyses. The solution of the reliability index, β, requires multiple integrations of the joint PDF over the nonlinear region $g(X) > 0$. For practical purposes however, only an approximation of β is required. Shinozuka (1983) notes that the point $(x_1'^*, x_2'^*,...,x_n'^*)$ on the failure surface with the

Table 3-8. Resistance Statistics for Permanent Timber Structures

City	Mean Resistance R_m	Standard Deviation σ_r
Knoxville, TN	13181.00	2108.96
Kansas City, MO	15537.44	2485.99
Albuquerque, NM	16437.38	2629.98
Dayton, OH	14862.11	2377.94
Denver, CO	13181.00	2108.96
Sacramento, CA	14726.46	2356.23
Jacksonville, FL	22768.16	3642.91
Chicago, IL	13181.00	2108.96
Rochester, NY	13181.00	2108.96
Pocatello, ID	13775.04	2204.01
Las Vegas, NV	15143.36	2422.94
Spokane, WA	13181.00	2108.96
Savannah, GA	21789.00	3486.24

smallest distance from the origin of the reduced variates is the most probable failure point. The tangent plane to the failure surface at $(x_1'^*, x_2'^*,...,x_n'^*)$ is then used to approximate the actual failure surface. From this, β and the probability of failure, P_f, may then be determined. Figure 3-1 illustrates a two-variable case where, depending on whether the nonlinear failure surface is convex or concave toward the origin, the approximation will be on the safe or unsafe side.

The pertinent tangent plane at $x'^* = (x_1'^*, x_2'^*,...,x_n'^*)$ is defined by Ang and Tang (1984) as:

$$\sum_{i=1}^{n} (X_i' - x_i'^*)\,(\partial g/\partial X_i')_* = 0 \qquad 3.29$$

where: $(\partial g/\partial X_i')_*$ is evaluated at $(x_1'^*, x_2'^*,...,x_n'^*)$

The point of most probable failure is defined as:

$$x_i'^* = -\alpha_i^*\beta \qquad 3.30$$

where: α_i = the direction cosine

$$\alpha_i^* = \frac{(\partial g/\partial X_i')_*}{\sqrt{\sum_i (\partial g/\partial X_i')_*^2}} \qquad 3.31$$

Upon evaluating the derivatives at $(x_1'^*, x_2'^*,...,x_n'^*)$, x_i^* is determined using:

$$x_i^* = \sigma_{xi}x_i'^* + m_{xi} = m_{xi} - \alpha_i^*\sigma_{xi}\beta \qquad 3.32$$

Solving the limit state equation:

$$g(x_1'^*, x_2'^*,...,x_n'^*) = 0 \qquad 3.33$$

yields the reliability index, β. The probability of failure may then be calculated using Equation 3.1:

$$P_f = 1 - \Phi(\beta)$$

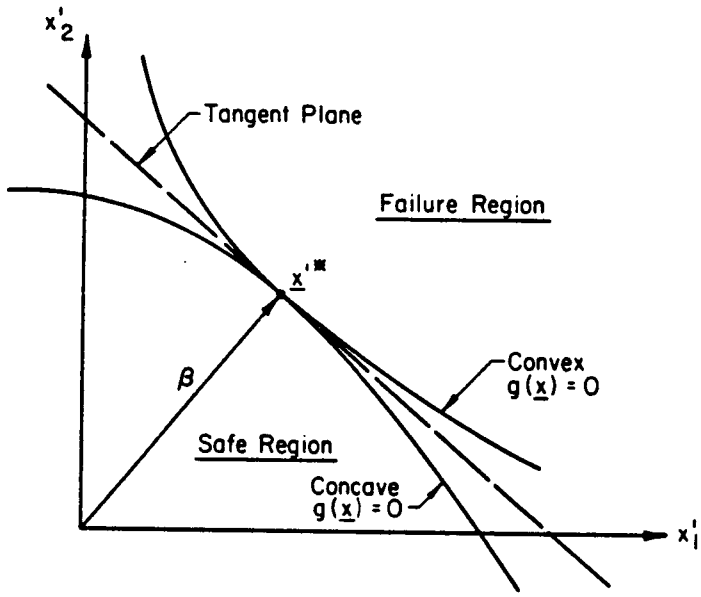

Figure 3-1. Tangent Plane to g(X) = 0 at x'*

In order to verify β and P_f values, a second-order approximation of the limit-state surface was performed. The second-order method assumes that the failure plane may be approximated by a quadratic surface at the design point, x'^*, as shown in Figure 3-2. Madsen (1985) notes that the approximation is such that the two surfaces have the same tangent hyperplane and second-order derivatives at x'^*. A three-term approximation to the probability content outside the approximating quadratic surface is as follows:

$$P_f \cong \Phi(-\beta_i) \prod_{j=1}^{n-1} (1 - \beta_i \kappa_{ij})^{-1/2} + [\beta_i \Phi(-\beta_i) - \varphi(\beta_i)] \{ \prod_{j=1}^{n-1} (1 - \beta_i \kappa_{ij})^{-1/2} \qquad 3.34$$

$$- \prod_{j=1}^{n-1} (1 - (\beta_i + 1)\, \kappa_{ij})^{-1/2} \} + (\beta_i + 1)[\, \beta_i \Phi(-\beta_i) - \varphi(\beta_i)] \, \{ \prod_{j=1}^{n-1} (1 - \beta_i \kappa_{ij})^{-1/2}$$

$$+ \mathrm{Re} \, \{ \prod_{j=1}^{n-1} (1 - (\beta_i + i)\, \kappa_{ij})^{-1/2} \} \}$$

where: κ_{ij} = the main curvatures

Re = the real part

i = the imaginary unit

= $\sqrt{-1}$

Table 3-9 illustrates β and P_f values for permanent metal and timber structures subjected to 50-year maximum wind loads.

To illustrate the extreme conservatism in designing temporary SSCs for 50-year maximum wind loads, β and P_f values were determined for a structural (hot-rolled) steel structure located in Knoxville, TN. As can be seen in Table 3-10, the probabilities of failure are much less than those for permanent structural steel structures, thus further justifying this study.

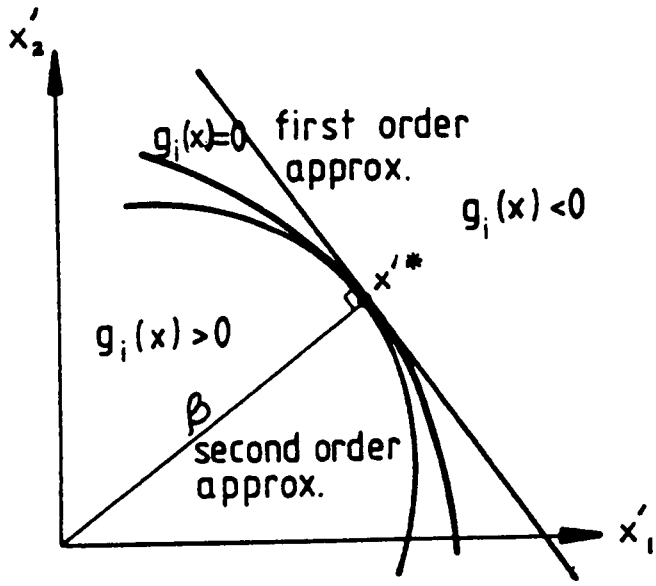

Figure 3-2. Second-Order vs First-Order Approximation

Table 3-9. β and P_f for Permanent Structures

City	Material	1st-Order Approximation		2nd-Order Approximation	
		β	$P_f (x10^{-2})$	β	$P_f (x10^{-2})$
Knoxville, TN	Structural (Hot-Rolled) Steel	1.849	3.221	1.859	3.152
	Cold-Formed Steel	1.982	2.375	1.993	2.316
	Aluminum	2.007	2.238	2.021	2.163
	Timber	2.764	0.286	2.661	0.390
Kansas City, MO	Structural (Hot-Rolled) Steel	1.862	3.133	1.875	3.041
	Cold-Formed Steel	1.988	2.343	2.002	2.265
	Aluminum	2.009	2.225	2.027	2.133
	Timber	2.759	0.290	2.658	0.393
Albuquerque, NM	Structural (Hot-Rolled) Steel	1.619	5.271	1.627	5.190
	Cold-Formed Steel	1.756	3.956	1.764	3.886
	Aluminum	1.778	3.772	1.790	3.677
	Timber	2.592	0.478	2.488	0.643
Dayton, OH	Structural (Hot-Rolled) Steel	1.605	5.423	1.614	5.328
	Cold-Formed Steel	1.740	4.097	1.749	4.014
	Aluminum	1.760	3.920	1.773	3.812
	Timber	2.574	0.502	2.471	0.674
Denver, CO	Structural (Hot-Rolled) Steel	2.648	0.405	2.653	0.399
	Cold-Formed Steel	2.791	0.263	2.798	0.257
	Aluminum	2.844	0.223	2.857	0.214
	Timber	3.310	0.047	3.241	0.060
Sacramento, CA	Structural (Hot-Rolled) Steel	1.374	8.467	1.393	8.189
	Cold-Formed Steel	1.490	6.806	1.509	2.564
	Aluminum	1.499	6.698	1.520	6.426
	Timber	2.302	1.068	2.220	1.320

Table 3-9. (cont.)

City	Material	1st-Order Approximation		2nd-Order Approximation	
		β	$P_f (x10^{-2})$	β	$P_f (x10^{-2})$
Jacksonville, FL	Structural (Hot-Rolled) Steel	2.323	1.010	2.346	0.949
	Cold-Formed Steel	2.430	0.755	2.454	0.706
	Aluminum	2.450	0.715	2.476	0.664
	Timber	3.098	0.098	3.008	0.132
Chicago, IL	Structural (Hot-Rolled) Steel	2.731	0.316	2.740	0.307
	Cold-Formed Steel	2.869	0.206	2.880	0.199
	Aluminum	2.919	0.176	2.935	0.167
	Timber	3.377	0.037	3.304	0.048
Rochester, NY	Structural (Hot-Rolled) Steel	2.041	2.062	2.041	2.061
	Cold-Formed Steel	2.190	1.425	2.192	1.420
	Aluminum	2.232	1.282	2.237	1.264
	Timber	2.913	0.179	2.830	0.233
Pocatello, ID	Structural (Hot-Rolled) Steel	1.590	5.592	1.596	5.524
	Cold-Formed Steel	1.730	4.185	1.736	4.125
	Aluminum	1.753	3.985	1.763	3.899
	Timber	2.576	0.500	2.473	0.670
Las Vegas, NV	Structural (Hot-Rolled) Steel	1.745	4.048	1.752	3.987
	Cold-Formed Steel	1.883	2.988	1.891	2.935
	Aluminum	1.908	2.820	1.919	2.746
	Timber	2.692	0.356	2.589	0.481
Spokane, WA	Structural (Hot-Rolled) Steel	2.014	2.199	2.025	2.144
	Cold-Formed Steel	2.145	1.597	2.157	1.550
	Aluminum	2.173	1.489	2.189	1.431
	Timber	2.889	0.194	2.787	0.266

Table 3-9. (cont.)

City	Material	1st-Order Approximation		2nd-Order Approximation	
		β	$P_f (x10^{-2})$	β	$P_f (x10^{-2})$
Savannah, GA	Structural (Hot-Rolled) Steel	2.401	0.818	2.424	0.767
	Cold-Formed Steel	2.508	0.606	2.533	0.566
	Aluminum	2.530	0.571	2.557	0.529
	Timber	3.164	0.078	3.073	0.106

Table 3-10. β and P_f for Temporary Structural Steel SSCs Located in Knoxville, TN and Subjected to a 50-Year Wind Load

Year	1st-Order Approximation		2nd-Order Approximation	
	β	P_f ($x10^{-2}$)	β	P_f ($x10^{-2}$)
2	3.048	0.115	3.076	0.105
3	2.966	0.151	2.992	0.139
4	3.034	0.121	3.056	0.112
5	2.905	0.183	2.927	0.171
50	1.849	3.221	1.859	3.152

The reliability index and P_f for structural (hot-rolled) steel will be used as the "target" values for temporary structures given that these yield the most conservative results. However, this author feels that the use of β and P_f values determined using basic wind speeds from Figure 1 in ASCE-7 (1993) for Denver, CO, Jacksonville, FL, Chicago, IL, Spokane, WA, and Savannah, GA, results in excessive conservatism. Thus, for these cities β and P_f values obtained using maximum (fastest-mile) wind speeds determined by Simiu et al. (1979) are used. These values are shown in parenthesis in Table 3-11 along with other target β and P_f values.

Determination of Wind Speeds for Temporary Structures

Design wind speeds may be determined for temporary structures which yield at or near the same reliability and probability of failure values as permanent SSCs subjected to 50-year wind loads. This is done by setting the β values for permanent and temporary structures equal and using the statistics for winds with return periods from 2 to 5 years. The nominal structural resistance may then be determined using the known mean structural resistance. Given that the nominal resistance is the product of the wind load multiplied by the factor of safety of actual to allowable load, the wind load is determined by:

$$F_W = R_n/FS \qquad 3.35$$

where: R_n = the nominal resistance

FS = the factor of safety

Wind speed is then determined by trial and error using a FOSM analysis. Statistical data for various speeds are input until a β is obtained at or near the target β.

Table 3-11. Target β and P_f Values for Temporary Structures

City	"Target" Values	
	β	P_f ($x10^{-2}$)
Knoxville, TN	1.849	3.221
Kansas City, MO	1.862	3.133
Albuquerque, NM	1.619	5.271
Dayton, OH	1.605	5.423
Denver, CO	2.648 (1.671)	0.405 (4.735)
Sacramento, CA	1.374	8.467
Jacksonville, FL	2.323 (1.548)	1.010 (6.081)
Chicago, IL	2.731 (1.605)	0.316 (5.424)
Rochester, NY	2.041	2.062
Pocatello, ID	1.590	5.592
Las Vegas, NV	1.745	4.048
Spokane, WA	2.014 (1.563)	2.199 (5.902)
Savannah, GA	2.401 (1.534)	0.818 (6.254)

Mean structural resistance statistics used in determining wind speeds for temporary SSCs have been obtained for structural (hot-rolled) steel under tension loading for various wind speeds and are shown in Table 3-12. Wind speed statistics for recurrence periods from 2 to 5 years are as shown in Table 3-4. Wind speeds satisfying the target β values listed in Table 3-11 for temporary structures with design lives of 2 to 5 years are provided in Table 3-13.

Base on the target wind speed hazard exceedance probabilities for temporary structures noted in Table 3-3, wind speeds may be determined by obtaining wind speeds at recurrence periods whose probabilities of maximum wind speed exceedance are less than or equal to 0.636. As was previously noted, this is the probability that a structure with a design life of 50 years (permanent structure) will experience the maximum wind speed with an annual exceedance probability of 0.02 (50-year return period). Wind speeds corresponding to target hazard exceedance probabilities have been determined for Knoxville, TN and are shown in Table 3-14. A comparison is made between these values and wind speeds determined using structural reliability analyses. Given that the wind load is proportional to the square of the wind velocity, wind loads have also been determined for Knoxville, TN and are compared to wind loads obtained from the reliability analyses. These values and their percent differences are shown in Table 3-15. This author feels that the wind speeds obtained using structural reliability methods are the more accurate values and should be used in the design of temporary SSCs. Wind speeds using these methods take into account the nature of the wind load and resistance of the structure.

Table 3-12. Resistance Statistics for Temporary Structures

Wind Speed (mph)	Nominal Resistance	Mean Resistance	Standard Deviation
45	3375.00	3543.75	389.81
46	3526.67	3703.00	407.33
47	3681.67	3865.75	425.23
48	3840.00	4021.00	443.52
49	4001.67	4201.75	462.19
50	4166.67	4375.00	481.25
51	4335.00	4551.75	500.69
52	4506.67	4732.00	520.52
53	4681.67	4915.75	540.73
54	4860.00	5103.00	561.33
55	5041.67	5293.75	582.31
56	5226.67	5488.00	603.68
57	5415.00	5685.75	625.43
58	5606.67	5887.00	647.57
59	5801.67	6091.75	670.09
60	6000.00	6300.00	693.00
61	6201.67	6511.75	716.29
62	6406.67	6727.00	739.97
63	6615.00	6945.75	764.03
64	6826.67	7168.00	788.48
65	7041.67	7393.75	813.31
66	7260.00	7623.00	838.53
67	7481.67	7855.75	864.13
68	7706.67	8092.00	890.12
69	7935.00	8331.75	916.49
70	8166.67	8575.00	943.25

Table 3-13. Wind Speeds (mph) for Temporary Structures

City	Design Life (yrs)			
	2	**3**	**4**	**5**
Knoxville, TN	54	55	56	57
Kansas City, MO	57	58	59	60
Albuquerque, NM	61	63	65	66
Dayton, OH	57	58	58	59
Denver, CO	50	51	51	52
Sacramento, CA	56	57	58	59
Jacksonville, FL	58	62	64	66
Chicago, IL	47	49	50	50
Rochester, NY	56	57	57	58
Pocatello, ID	56	57	58	59
Las Vegas, NV	58	60	61	63
Spokane, WA	49	51	52	53
Savannah, GA	55	58	61	63

Table 3-14. Comparison of Wind Speeds (mph) for Temporary SSCs in Knoxville, TN

Design Life (yrs)	Hazard Exceedance Probability Method	Structural Reliability Method	Percent Difference
2	51	54	5.88
3	53	55	3.77
4	54	56	3.70
5	58	57	1.72

Table 3-15. Comparison of Wind Loads $(IV)^2$ for Temporary SSCs in Knoxville, TN

Design Life (yrs)	Hazard Exceedance Probability Method(1)	Structural Reliability Method(1)	Percent Difference
2	2601	2916	12.11
3	2809	3025	7.69
4	2916	3136	7.55
5	3364	3249	3.54

(1) Wind load values do not include air mass density constant of 0.00256 and assumes 1.0 for K, G, C_p, and I factors

The design wind speeds presented in Table 3-13 are based on an importance factor, I, of 1.0. This corresponds to normal (PC-1) facilities. For DOE PC-2 type facilities, or commercial facilities meeting ASCE-7 (1993) Category II or III criteria, design wind speeds are to be multiplied by an I of 1.07.

Figure 3-3 illustrates the annual fastest-mile wind speeds observed for Knoxville, TN by Simiu, et al. (1979) from 1942 to 1974. For comparison, the design wind speed for permanent structures per ASCE-7 (1993) and the calculated wind speed for a temporary structure with a design life of 5 years (Table 3-13) are shown. As can be seen, during the years from 1942 to 1974, no annual fastest-mile wind speeds exceeded the 50-year design wind speed of 70 mph. The calculated wind speed of 57 mph for a 5-year temporary structure was exceeded only three times during the 33 year reporting period.

Related Codes, Guides, and Publications

ASCE-7 (1993) addresses the use of lesser wind speeds and, hence, wind loads by allowing the use of a lower importance factor, I. An I of 0.95 may be used for Category IV SSCs. Although Category IV includes certain temporary structures, it is unclear as to what ASCE-7 (1993) defines as a temporary structure. Multiplying the design wind speed by an I of 0.95 yields a wind speed that approximates a 25-year return period (0.04 annual exceedance probability) extreme fastest-mile wind speed. Table 3-16 provides wind speeds for the cities of interest using the basic wind speed from ASCE-7 (1993) multiplied by an I of 0.95. Wind speeds predicted by Simiu et al. (1979) for a 25-year return period are also included. For comparison, wind speeds obtained from the structural reliability analyses for SSCs with a 5-year design life are included.

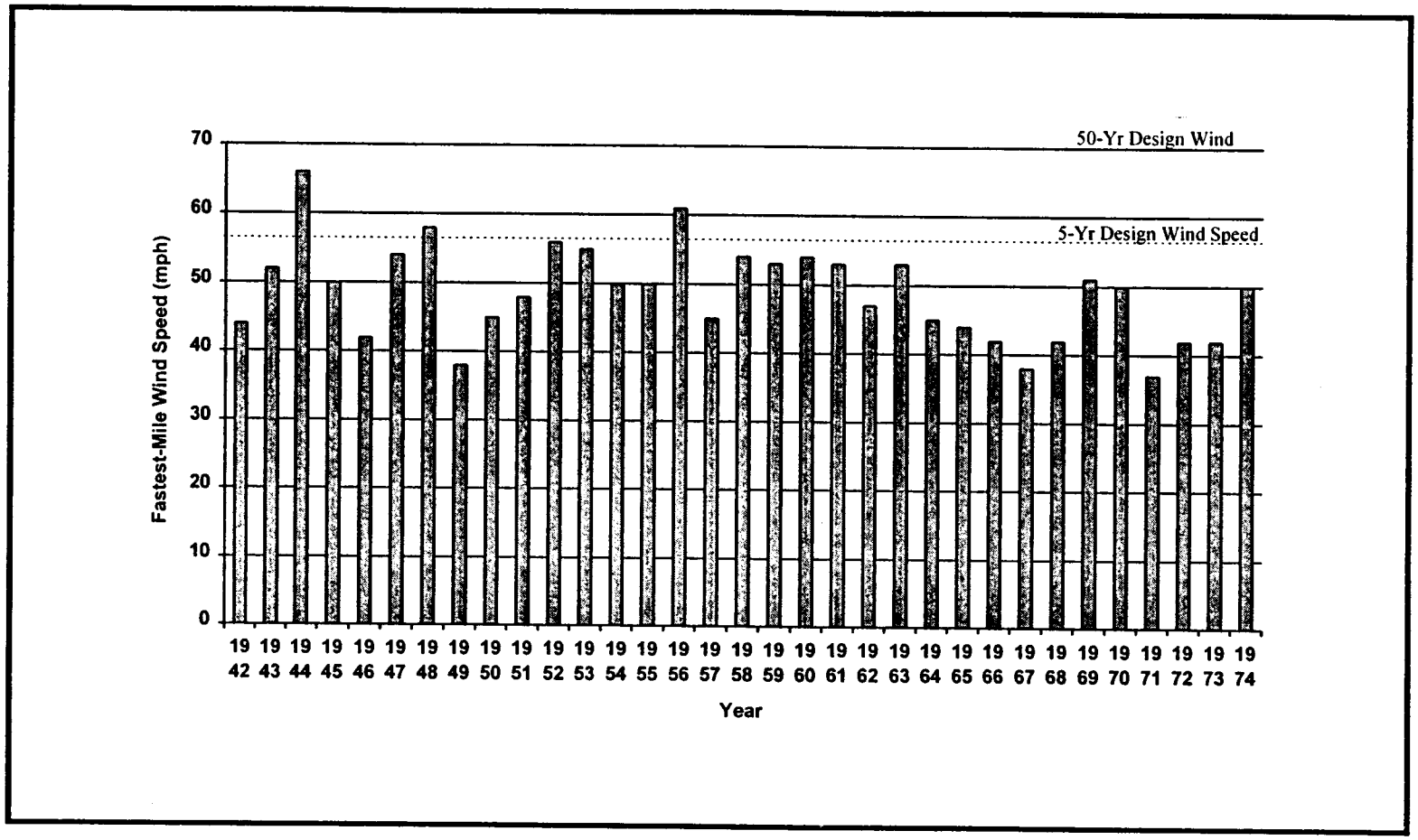

Figure 3-3. Fastest-Mile Wind Speeds for Knoxville, TN (1942 - 1974)

Table 3-16. Wind Speeds w/I=0.95 and for 25-Year Return Periods

City	Wind Speed w/I = 0.95 (mph)[1]	Wind Speed for 25-Yr Return Period (mph)[2]	Wind Speed Using Reliability Analyses 5-Yr Design Life (mph)
Knoxville, TN	66.5	63.5	57
Kansas City, MO	72.2	67.3	60
Albuquerque, NM	74.3	73.6	66
Dayton, OH	70.6	69.9	59
Denver, CO	66.5	59.5	52
Sacramento, CA	70.3	68.0	59
Jacksonville, FL	87.4	70.5	66
Chicago, IL	66.5	57.3	50
Rochester, NY	66.5	64.6	58
Pocatello, ID	68.0	67.6	59
Las Vegas, NV	71.3	70.7	63
Spokane, WA	66.5	61.5	53
Savannah, GA	85.5	68.1	63

(1) Obtained from ASCE-7 (1993) using an I of 0.95
(2) Predicted 25-year fastest-mile wind speed by Simiu et al. (1979)

As can be seen, wind speeds obtained using an I of 0.95 and wind speeds predicted for 25-year return periods are conservative when compared to wind speeds obtained using structural reliability methods. This conservatism is further magnified when the wind loads are examined.

International standards vary in their treatment of wind loads on temporary structures. For instance, the British Standard, BSI Code of Practice, CP3 (1970), calculates the design wind speed, V_s, using the following:

$$V_s = S_1 S_2 S_3 V \qquad 3.36$$

where: V = the basic wind speed from a 3-second gust

S_1 = the topography factor

S_2 = the exposure factor

S_3 = the statistical (importance) factor

The British Standard allows for the use of an S_3 less than 1.0 for temporary structures.

It appears, according to Liu (1991), that Australia has been in the forefront to modernize wind load provisions of building codes. Similar to the British Standard, the Australian Standard (SAA Loading Code, 1989) uses a basic wind speed determined from a 3-second gust. However, the Australian Standard allows for the use of varying return periods depending on structure classifications. For ordinary structures, a return period of 50 years is used. Structures having postdisaster functions such as hospitals and communications centers are to be designed using a return period of 100 years. Structures posing low hazard to life and property may be designed for a return period of 25 years. Wind speeds corresponding to a 5-year return periods may be used for the design of structures used in construction including formwork and scaffolding. Wind

speeds corresponding to a 5-year return period closely approximates wind speeds obtained using structural reliability methods.

IV. CONCLUSIONS

Wind speeds for temporary structures have been determined for 13 cities in the contiguous United States. These cities correspond to approximate locations of DOE project sites. Structural reliability methods using classical FOSM analyses were performed using fastest-mile wind speed data obtained by Simiu et al. (1979). Reliability indices and probabilities of failure were determined for permanent (50-year design life) SSCs subjected to predicted extreme fastest-mile wind speeds based on a 50-year return period (0.02 annual exceedance probability). These reliability indices and probabilities of failure were then set as target values for temporary structures. From these target values, nominal structural resistance was determined using known mean structural resistance data and statistical data calculated for winds with return periods from 2 to 5 years. Wind speeds were then determined based on an importance factor of 1.0. These correspond to DOE PC-1 and ASCE-7 (1993) Category I SSCs. Multiplying these wind speeds by an importance factor of 1.07 yields design wind speeds for use with DOE PC-2 and ASCE-7 (1993) Category II and III SSCs. Table 4-1 provides design wind speeds for PC-1/Category I and PC-2/Category II and III SSCs. Comparing the design wind speed for a temporary PC-1 structure with a 5-year design life located in Knoxville, TN with the design wind speed from the ASCE-7 (1993) wind speed map, 57 mph and 70 mph, respectively, one can readily see the conservatism of approximately 20 percent. Given that the wind load is proportional to the wind speed squared, this conservatism increases to approximately 35 percent. For cities whose design wind speeds per ASCE-7 (1993) exceed the predicted wind speeds per Simiu et al. (1979), such as Denver, CO, Jacksonville, FL, Chicago, IL, Spokane WA, and Savannah, GA, the conservatism is even greater.

Table 4-1. Design Wind Speeds for Temporary PC-1 and PC-2 SSCs

Design Life (yrs)	**2**		**3**		**4**		**5**	
Performance Category	**PC-1**	**PC-2**	**PC-1**	**PC-2**	**PC-1**	**PC-2**	**PC-1**	**PC-2**
Importance Factor	**1.0**	**1.07**	**1.0**	**1.07**	**1.0**	**1.07**	**1.0**	**1.07**
City								
Knoxville, TN	54	58	55	59	56	60	57	61
Kansas City, MO	57	61	58	62	59	63	60	64
Albuquerque, NM	61	65	63	67	65	70	66	71
Dayton, OH	57	61	58	62	58	62	59	63
Denver, CO	50	54	51	55	51	55	52	56
Sacramento, CA	56	60	57	61	58	62	59	63
Jacksonville, FL	58	62	62	66	64	69	66	71
Chicago, IL	47	50	49	52	50	54	50	54
Rochester, NY	56	60	57	61	57	61	58	62
Pocatello, ID	56	60	57	61	58	62	59	63
Las Vegas, NV	58	62	60	64	61	65	63	67
Spokane, WA	49	52	51	55	52	56	53	57
Savannah, GA	55	59	58	62	61	65	63	67

Wind speeds were also determined for temporary structures with design lives from 2 to 5 years located in Knoxville, TN using a target hazard exceedance probability method. This method used the probability of maximum wind speed exceedance for a permanent structure, approximately 64 percent, and set this as the target hazard exceedance probability for temporary structures. Wind speeds at return periods satisfying the target hazard exceedance probability were set as the design wind speeds. It was found that the percent differences in wind speeds and wind loads as compared to those obtained using structural reliability methods were relatively small. The differences tended to decrease as the design life increased. Although the differences were relatively small, it is felt that the structural reliability method yields more accurate results given that, not only is the nature of the wind load considered, but so is the resistance of the structure.

When international codes and criteria are examined, it is evident that Australia has performed research into wind loads on temporary structures. The Australian standard (SAA Loading Code, 1989) allows for the use of wind speeds based on a 5-year return period for temporary structures used in construction such as formwork and scaffolding. Wind speeds corresponding to a 5-year return period compare favorably with wind speeds obtained using structural reliability methods. Using wind speeds corresponding to a 5-year return period for temporary structures with design lives up to 5-years may be the most practical approach for this country as well.

The design wind speeds determined using structural reliability methods were based on annual fastest-mile wind speeds obtained over a period of 10 to 40 years. Using annual data tended to reduce the accuracy of the statistical wind speed data as the design life of the SSC increased.

The accuracy of the statistical data could be improved if shorter duration (e.g. monthly, weekly, etc.) wind speed data were used. This information may be obtained for many cities from the National Climatic Data Center located in Asheville, NC. Also, site-specific climatological data could be used to determine design wind speeds for temporary SSCs located at DOE project sites.

ASCE is in the process of issuing a revised ASCE-7. The revisions include the use of 3-second gust wind speeds in lieu of fastest-mile wind speeds. Although, typically, the 3-second gust wind speeds are greater than fastest-mile wind speeds, it is expected that the wind pressures yielded by the two wind speeds will be approximately the same. This will be accomplished by revising the current ASCE empirical formula for calculating wind pressure. Although specific wind speeds for temporary structures may vary using the 3-second gust, the methodology should be applicable. It is the hope of this author that additional studies will be made in the determination of not only wind speeds for temporary structures, but also other natural phenomena loading such as seismic loading. And he further hopes that, sometime in the near future, guidance will be provided in building codes, standards, and criteria that will prevent the continued typical overdesign, and sometimes the rare underdesign, of temporary SSCs subjected to these loads.

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VITA

Wayne S. Moore was born in San Diego, California on June 3, 1962. In 1964, his family relocated to North Charleston, South Carolina. He attended schools in the North Charleston Public School System, where he graduated from North Charleston High School in June 1980. He studied Civil Engineering Technology at Trident Technical College in Charleston, South Carolina from June 1981 to June 1982. After working as a draftsman for two years, he enrolled in the University of Tennessee at Knoxville where he received a Bachelor of Science degree in Civil Engineering in December 1987. Upon graduation he worked as a staff Structural Engineer for Lockwood Greene Engineers in Oak Ridge, Tennessee and then for Barge, Waggoner, Sumner, and Cannon in Miamisburg, Ohio. In January 1992, he took a position with PARSONS as a Project Engineer overseeing D&D projects at the DOE Fernald Environmental Management Project in Fairfield, Ohio. In January 1995, he returned to the University of Tennessee at Knoxville where, in May 1996, he received a Master of Science degree in Civil Engineering.

The author is presently acting as a consulting engineer to PARSONS Engineering Science and Lockwood Greene Technologies in Oak Ridge, Tennessee. He is a member of numerous technical societies including the American Society of Civil Engineers and the International Society for Decontamination and Decommissioning Professionals.