A UNIFORM APPROACH TO DESIGN OF WINDLOADS ON STRUCTURES

by

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CHAPTER 1

INTRODUCTION

1.1 Problem Statement

"Uniform" is defined as having the same manner, and "approach" is defined as a path, road or other means of reaching a certain thing. Therefore, combining these two words, "uniform approach," means having the same manner of reaching a certain thing in general. A uniform approach to design for wind loads implies using the same methodology when treating straight winds, hurricane winds and tornado winds.

1.2 Objective And Scope

The objective of this study is to propose a uniform approach for the design of low-rise buildings and facilities to assure that they are adequately constructed to resist the effects of all types of high winds. This study provides guidelines on how to design facilities in a manner such that the risk of adverse consequences is consistent with the criticality, function and cost of the facility. The risk of adverse consequences resulting from wind loads is a combined function of the hazard exceedance probability along with the margins of safety introduced by the design method and other sources of conservatism. An

acceptable design is accomplished by defining facility use based on characteristics such as the mission dependent purpose of a facility, its monetary value and the consequence of facility damage in terms of public safety.

1.3 Background

The three windstorms that engineers deal with in designing structures for are thunderstorms/gust fronts (designated "straight" winds), hurricanes and tornadoes.

Winds associated with the three types of windstorms have been treated somewhat differently in the past. The traditional way to design for straight and hurricane winds is to follow the provisions of the local jurisdictional building code. This approach works reasonably well for ordinary buildings and structures. Facilities requiring a higher margin of safety because of their contents, mission or hazard potential to the environment require consideration beyond local code provisions. One approach is to select a larger mean recurrence interval (lower annual probability of exceedance), which results in higher design wind speed. Provisions of ANSI A58.1-1982 are then applied to obtain design wind pressures.

A site specific hazard assessment is required for tornadoes. An acceptable probability level is selected and the design wind speed is determined from the hazard assessment model. Present day tornado resistant design

methods treat tornado wind speeds as peak gusts (2-3 seconds averaging time). Wind pressures are calculated with the gust response factor taken as unity. Pressure coefficients are obtained from ANSI A58.1-1972. Tornado wind speeds are assumed constant with height for low-rise buildings.

The uniform approach to design wind pressures proposed in this thesis treats the three types of winds (straight, hurricane and tornado) in an identical way by applying the provisions of ANSI A58.1-1982.

Chapter 2 addresses the effects of winds on buildings and describes how the ANSI A58.1-1982 Standard addresses these effects. The similarities of damage produced by the three kinds of winds are noted, which lead to the concept of a uniform approach. Chapter 3 demonstrates the feasibility of a uniform approach and points out its advantages. Details of the Uniform Approach are presented in Chapter 4. The present method (as opposed to the proposed method) for treating tornado wind loads is described in Chapter 5. Since the primary difference in the current approach and the proposed uniform approach is in the treatment of tornado wind pressures, results from the two methods are compared and evaluated in Chapter 6. The last chapter contains conclusions and recommendations.

CHAPTER 2

WIND EFFECTS ON BUILDINGS

2.1 Physical Effects of Straight Winds on Buildings

Wind speeds fluctuate about a mean and the mean varies with height above ground. The wind, the terrain and the structure can be modeled in a wind tunnel, which allows measurement of wind pressure distribution on the structure model. Full-scale measurements may be used to verify the wind pressure distributions found in the wind tunnel experiments.

2.1.1 Measured Characteristics of the Wind

Movement of air parallel to the ground is generally termed as "wind" for engineering purposes. The ground and man-made obstructions retard movement of air close to the ground surface. This retardation causes a reduction in wind speed. At some height above the ground, the movement of air is independent of the ground obstructions. This unobstructed wind speed is termed "gradient wind speed," and the height at which the air movement is not retarded is termed "gradient height." The effect of friction grows progressively smaller with distance from the surface, causing an increase in wind speed and a change of

direction with height up to the gradient height. Above the gradient height wind speed is constant at a value called the gradient wind speed. The air below the gradient height is called the boundary layer. The thickness of the boundary layer and the velocity profile are both largely dependent on turbulence generated by ground roughness. The resulting airstream is very disturbed, leading to the production of gusts of varying size, intensity and frequency.

2.1.2. Pressure Distributions from Wind Tunnel

Wind tunnels are used to measure the wind pressure distribution on buildings. The process is effective for very tall buildings and more difficult for low-rise buildings, because of the scale. Nevertheless, much has been learned about wind pressures on low-rise buildings in the wind tunnel [2, 3, 4]. The key to good wind tunnel results is to use a boundary layer wind tunnel.

Wind tunnel studies confirm that when the wind flows over and around a structure, its direction and speed change. If a building is regarded as an enclosed structure consisting of five surfaces (four walls and a roof) with wall corners, eaves, roof corners, and roof ridges in the case of a gabled roof, then the changes in direction and speed of the air flow influence the pressures

developed on each part of the building. These wind pressures can be broadly classified as pressures and local pressures.

The air flow produces an inward-acting pressure on the windward wall and outward-acting pressures on the side walls, the leeward wall and the roof. A very steep gable roof could have inward-acting pressures on the windward side. Locally, the air flow streamlines separate from the building surfaces at sharp corners. Relatively low outward acting pressures occur immediately downstream of the streamline separation points, which occur at wall corners, eaves, roof corners and ridges.

In a naturally ventilated building the internal pressure depends on the relationship between the external pressure distribution and the location and size of any openings through which air may flow into or out of the building. Considerable variation in internal pressure between different spaces within a building is possible. This variation affects loads on interior walls and may give rise to wind-induced loads on internal partitions.

Wind tunnel measurements on various types of low-rise buildings confirm the pressure of overall, local and internal pressure. The pressure variation is expressed as a function of free field wind speed in the form of a pressure coefficient

$$C_p = \frac{P_w - P_o}{0.5 \rho V_o^2} \qquad (2.1)$$

where

C_p = pressure coefficient

P_w = wake pressure

P_o = free stream pressure

V_o = free stream velocity

ρ = mass density of air at standard condition (0.00238 slugs/ft^3).

2.1.3 Full-Scale Measurements

Full-scale measurements were made on the Aylesbury House by the Building Research Establishment in England. Tests also were conducted by several wind tunnel facilities in Australia, Canada and the U.S. on models of the Aylesbury House over a range of sealing ratios. Results generally agreed with full-scale measurements when differences in velocity profile and static pressure references were taken into account [17].

Another comparison of wind tunnel and full-scale tests was made in 1982. The full-scale tests were performed at the Florida Solar Energy Center at Cape Canaveral, Florida. The wind tunnel studies were performed at Colorado State University. Comparison of results show quite good agreement. These comparisons lead to the conclusion that boundary layer wind tunnel testing

is a valid method for measuring wind pressure distributions on buildings.

2.2 ANSI Standard Addresses Effects of Winds on Buildings

Since publication of the 1972 edition of ANSI A58.1, the provisions of the Standard have addressed the physical effects of wind on structures as observed in wind tunnels, full-scale measurements and post-storm damage investigations. The 1982 edition reflected new knowledge, as does the edition which will be released in September, 1987.

The velocity Pressure Exposure Coefficient in ANSI A58.1 accounts for different degrees of terrain roughness and analytically defines the wind speed profile.

Since wind speed fluctuates randomly, its fluctuating properties need to be considered in statistical terms. Random fluctuations can be expressed as a frequency spectrum. In dynamic analysis of a structure subject to gust loading, significant dynamic amplification of response occurs at the resonance frequency, i.e., when natural frequencies of vibration of the structure and the wind match.

A structure will not respond fully to the impact of a gust which is only a small fraction of the size of the structure. A gust, to be fully effective, must have sufficient spatial extent to envelop both the structure

itself and the flow patterns on the windward and leeward sides which are responsible for the maximum loads on the structure. A correlation function has been defined [17] which accounts for the fact the wind gusts are not likely to act simultaneously over the full extent of the structure. The gust correlation function could vary from unity for completely correlated flow to zero for uncorrelated flow.

Gust response factors (GRFS) account for the additional loading effects due to wind turbulence over the fastest-mile of wind. Because of the relationship between gust size and building size, there are two reasons for separating determination of GRFS into two systems.

1) Main Wind-Force Resisting System:

   a) has larger periods (lower frequencies) that more nearly match gust frequencies,

   b) has larger tributary areas that are not always engulfed by a gust.

2) Components and Cladding:

   a) have shorter periods (higher frequencies) that do not match gust frequencies so well,

   b) have smaller tributary areas that are affected by gust size, i.e., the gust engulfs the tributary areas.

Pressure and force coefficient values provided in ANSI A58.1-1982 have been assembled from the latest boundary-layer wind-tunnel and full-scale tests and from the previously available literature. Some pressure

coefficients in ANSI do not come from boundary layer wind tunnels. However, their use over many years has justified the correctness of the values. Design pressures are determined for (a) the main wind-force resisting system and (b) components and cladding for each building and structure.

There are three types of design wind pressure coefficients overall, local and internal pressure coefficients provided in the ANSI A58.1-1982 Standard. Local and internal pressure coefficient are combined with gust response factors (GC_p & GC_{pi}). The coefficients are defined to match measured pressure distributions in the wind tunnel. See Appendix A for a summary of ANSI A58.1-1982 provisions.

The characteristics of wind and pressure distributions that act on structures, are reflected in damage from straight, hurricane and tornado winds. The commonality of damage from the three types of windstorms is discussed in the next section.

2.3 Observations of Damage from Straight, Hurricane and Tornado Winds

The phenomena of overall, local and internal pressures on buildings measured in the wind tunnel and in full-scale tests and addressed by ANSI A58.1-1982 can be observed in windstorm damage. The type of damage is

observed in all three types of windstorms, and appears to be similar. This leads to the contention that wind pressures on buildings as reflected in damage generally are independent of the type of wind storm. Material is presented in this section to support this conclusion.

2.3.1 Similarity of Damage

Observations of damage from straight, hurricane and tornado winds indicate that there is similarity of damage. Damage to windward walls, leeward walls, side walls and roofs is produced by overall pressures or the combination of overall and internal pressures; damage to eaves, roof corners and wall corners is produced by local pressures or the combination of local and internal pressures. This damage similarity suggests that wind pressures on buildings generally are independent of the type of windstorm. Damage photographs from the three types of windstorms are presented to show similarity of damage from overall, local and internal pressures.

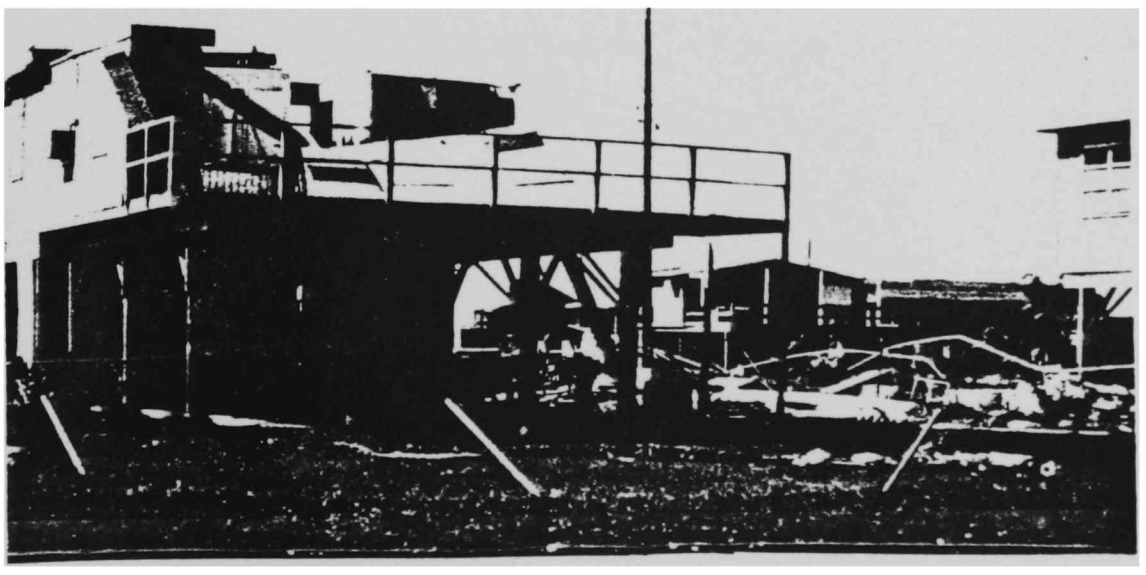
1) Overall Pressures:

Overall pressures affect the main wind-force resisting system at windward walls, leeward walls, side walls and roofs

a) Windward Walls

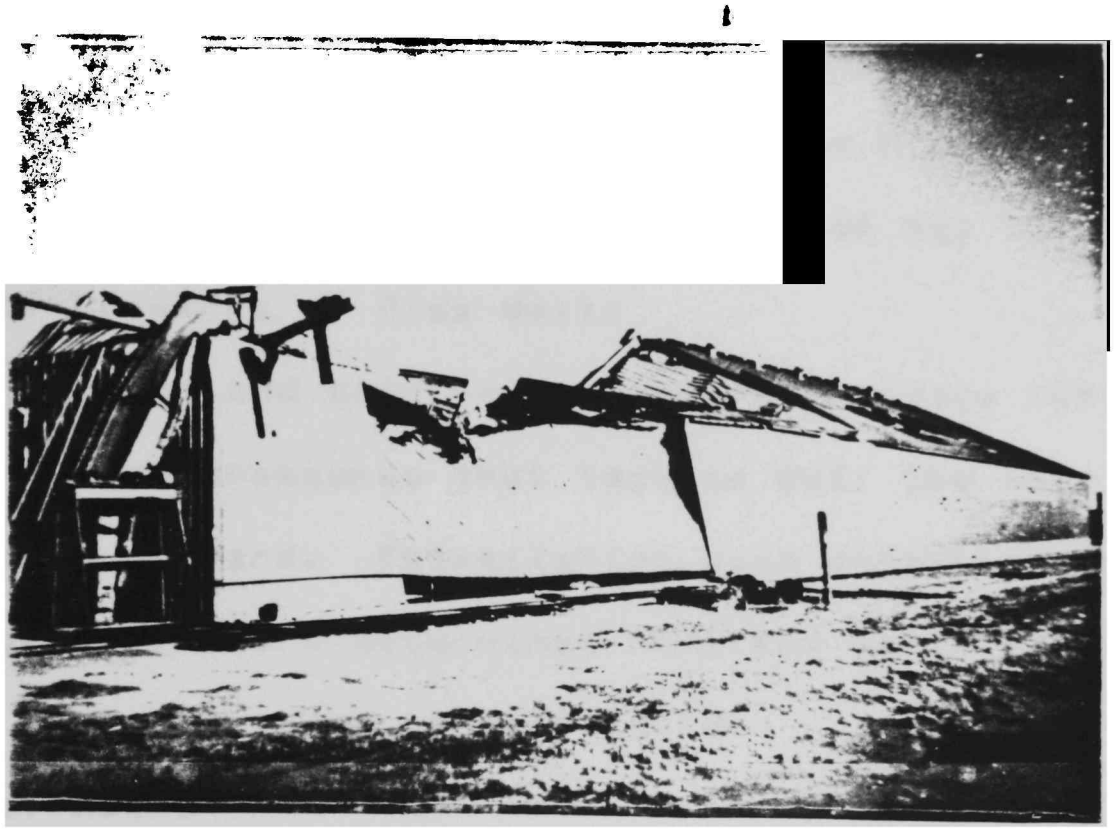
The windward wall of the gymnasium in Figure 2.1(a) fell inward from windward wall pressures produced by a

a) Straight Wind - Big Spring Texas, 1973



b) Hurricane Alicia, 1983

FIGURE 2.1 OVERALL EFFECTS ON WINDWARD WALL



c) Lubbock Tornado, 1970

FIGURE 2.1 (CONT.)

straight windstorm in Joplin, Missouri on May 11, 1973. Hurricane Alicia (Galveston, Texas, 1983) produced windward wall damage to the beach house shown in Figure 2.1(b). Windward wall damage shown in Figure 2.1(c) was caused by the Lubbock, Texas Tornado of May 11, 1970.

b) Leewarde or Side Walls

Leeward and side wall pressures produce outward acting wind pressures that tend to pull the wall or its cladding outward. Establishing wind direction from other damage indicators determined that the walls in Figure 2.2 were either leeward or side walls.

c) Roofs

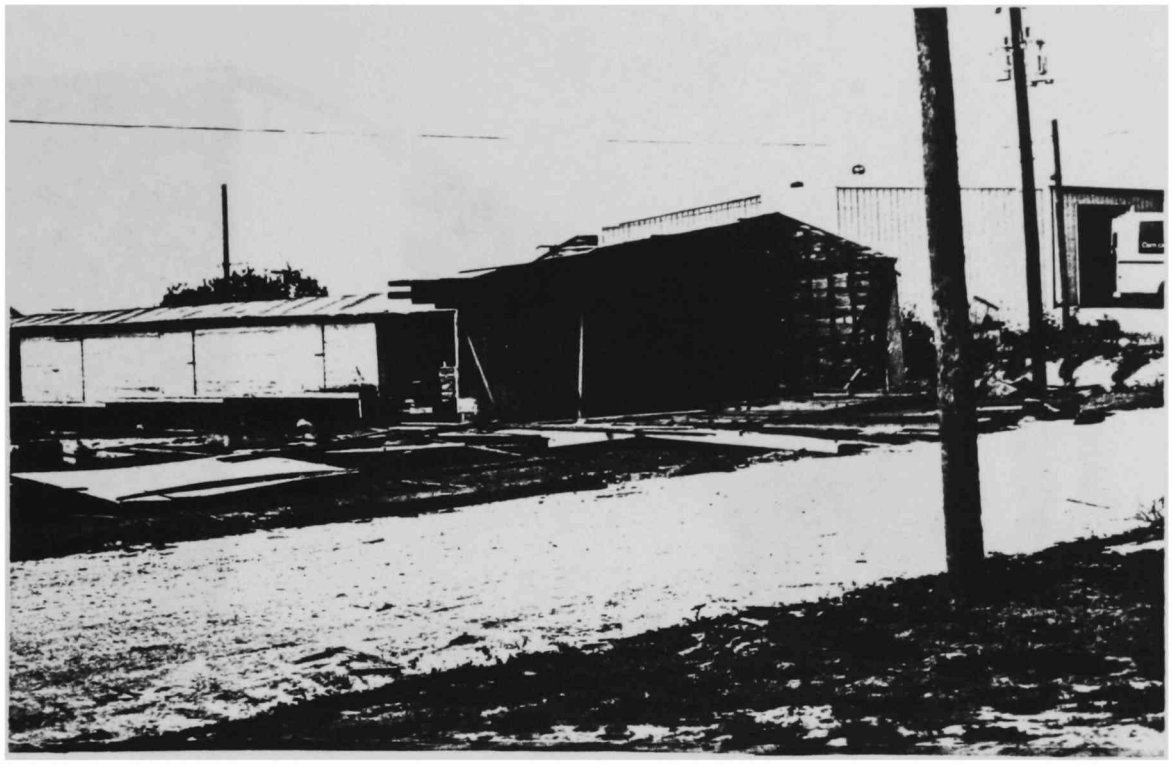
Overall upward pressures on relatively flat roofs are illustrated in Figure 2.3. Roofing material and roof decking were uplifted from the building in Figure 2.3(a) by the Joplin, Missouri Windstorm. Roof damage in parts (b) and (c) of the figure were caused by Hurricane Frederick (1979) and the Wichita Falls, Texas Tornado (1979).

2) Local Pressures

Local pressures affect components and cladding at eaves, roof corners and wall corners.

a) Eaves

The mobile home experienced eave damage by wind blowing essentially perpendicular to the long dimension of the unit in Figure 2.4(a). The damage occurred in the Big



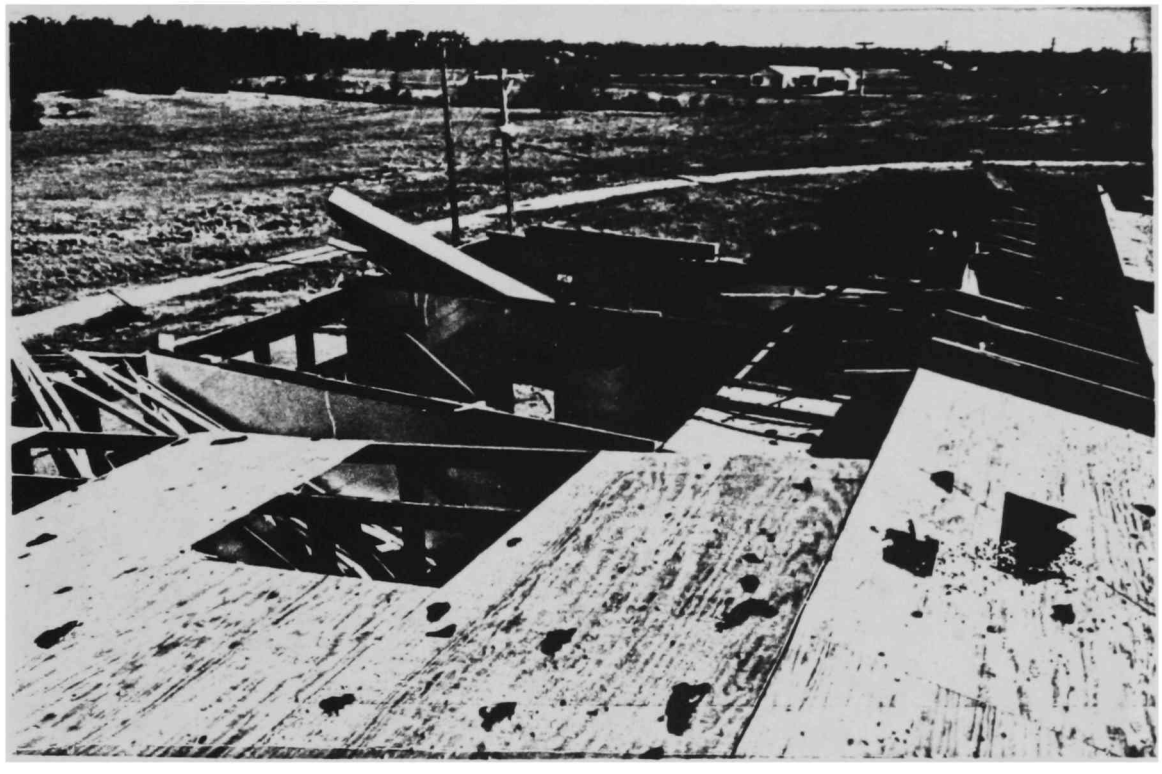
a) Straight Wind - Big Spring Texas, 1973

b) Hurricane Diana, 1984

FIGURE 2.2 OVERALL EFFECTS ON LEEWARD OR SIDE WALL

c) Lubbock Tornado, 1970

FIGURE 2.2 (CONT.)



a) Straight Wind - Joplin Missouri, 1973

b) Hurricane Frederic, 1979

FIGURE 2.3 OVERALL EFFECTS ON ROOF

c) Wichita Falls Tornado, 1979

FIGURE 2.3 (CONT.)

a) Straight Wind - Big Spring Texas, 1973

b) Hurricane Diana, 1984

FIGURE 2.4 LOCAL EFFECTS ON EAVE

c) Lubbock Tornado, 1970

FIGURE 2.4 (CONT.)

Spring, Texas Windstorm (1973). Hurricane Diana produced the eave (and roof corner) damage on the gabled roof of the building in Figure 2.4(b). Damage on both edges of the roof suggests there was a wind direction change during the course of the storm. Eave damage to metal buildings in Figure 2.4(c) was produced by the Lubbock, Texas Tornado of 1970.

b) Roof Corners

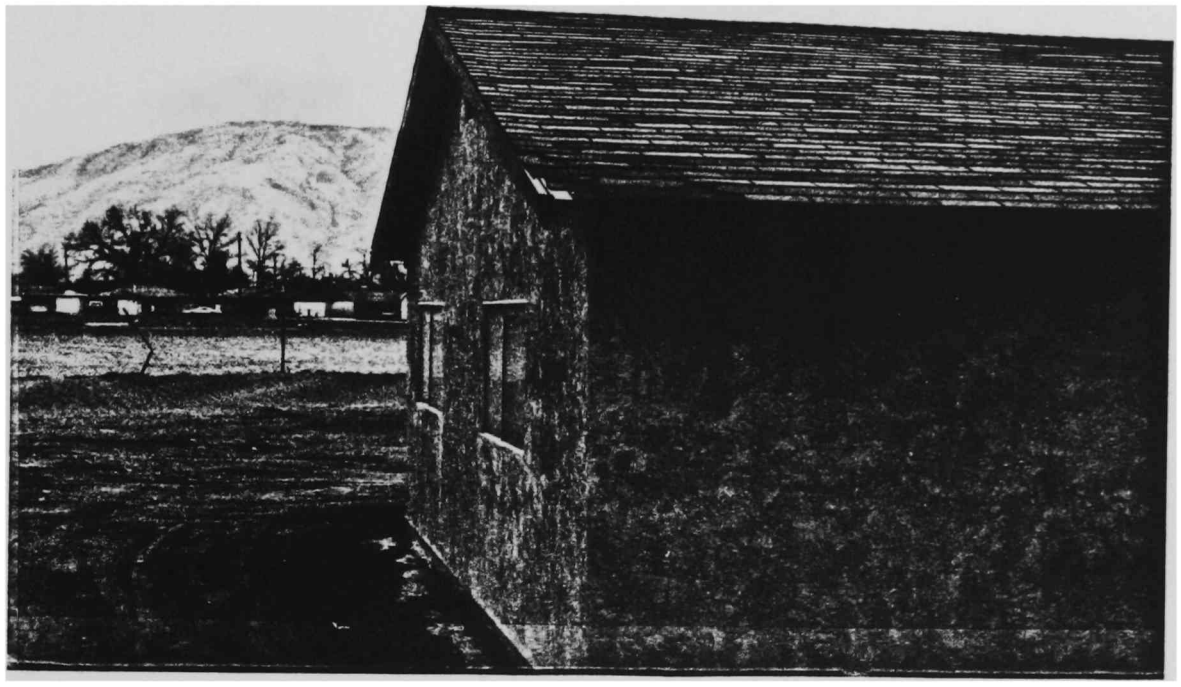
Roof corner damage produced straight winds (Arvin, California, 1977) and a tornado (Omaha, Nebraska,1975) are illustrated in Figure 2.5.

c) Wall Corners

Outward acting pressures are produced by winds flowing around a wall corner. Examples of wall corner damage from Arvin, California (straight winds), Galveston, Texas (hurricane winds) and Lubbock, Texas (tornado winds) are illustrated in Figure 2.6.

3) Internal Pressures

The effects of internal pressure combining with external pressure to produce a worst case are illustrated in Figure 2.7. In part (a) of the figure, the windward liter broke at the second floor level, which is a windward wall. The addition internal pressure that destroyed on the roof and side walls produced failure of these wall elements. At the first floor level, where the windows did not break, the side walls remain intact. The damage was

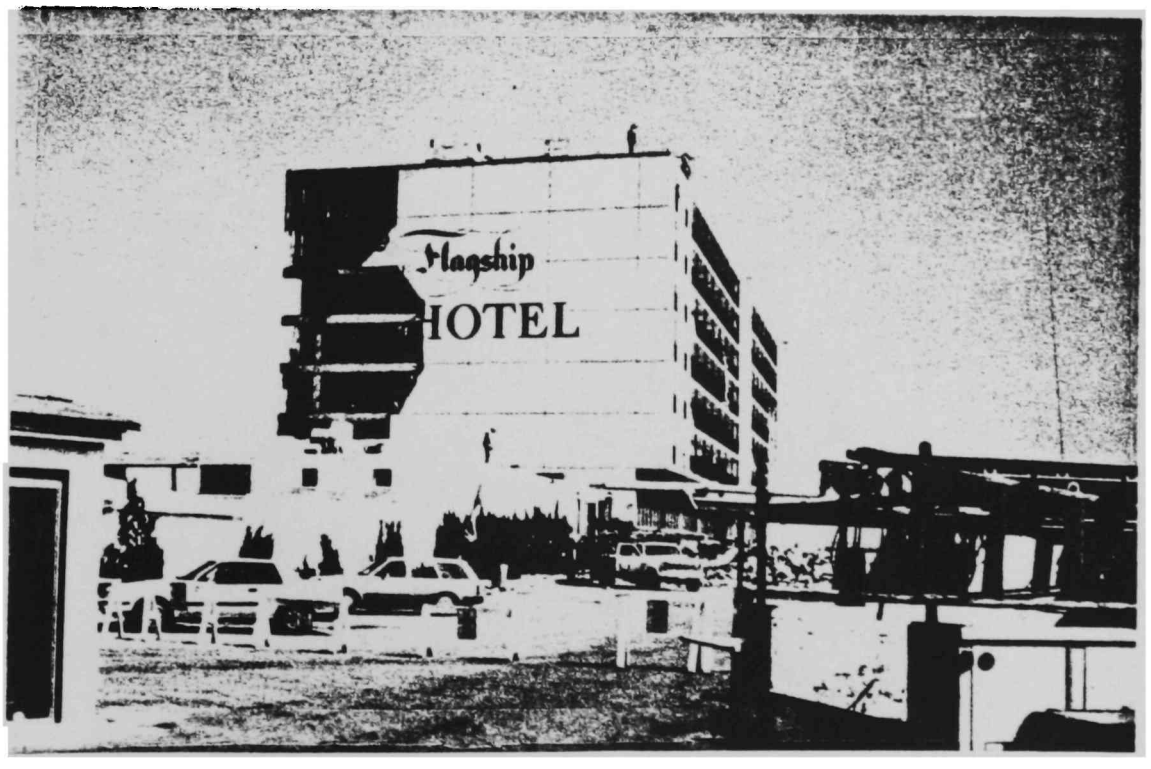


a) Straight Wind - Arvin California, 1977

c) Omaha Tornado, 1975

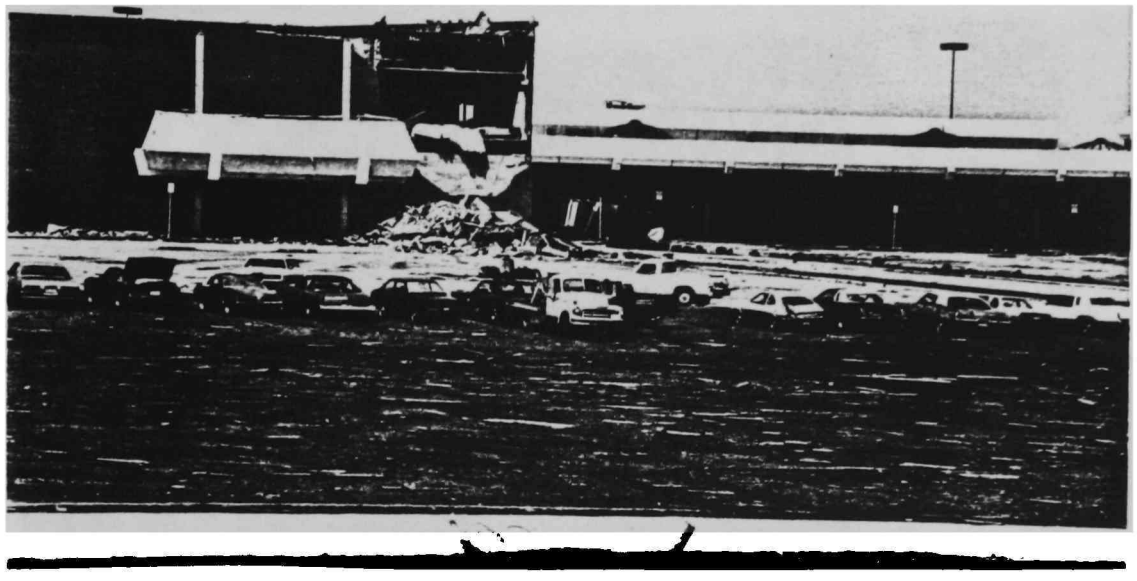
FIGURE 2.5 LOCAL EFFECTS ON ROOF CORNER

a) Straight Wind - Arvin California, 1977



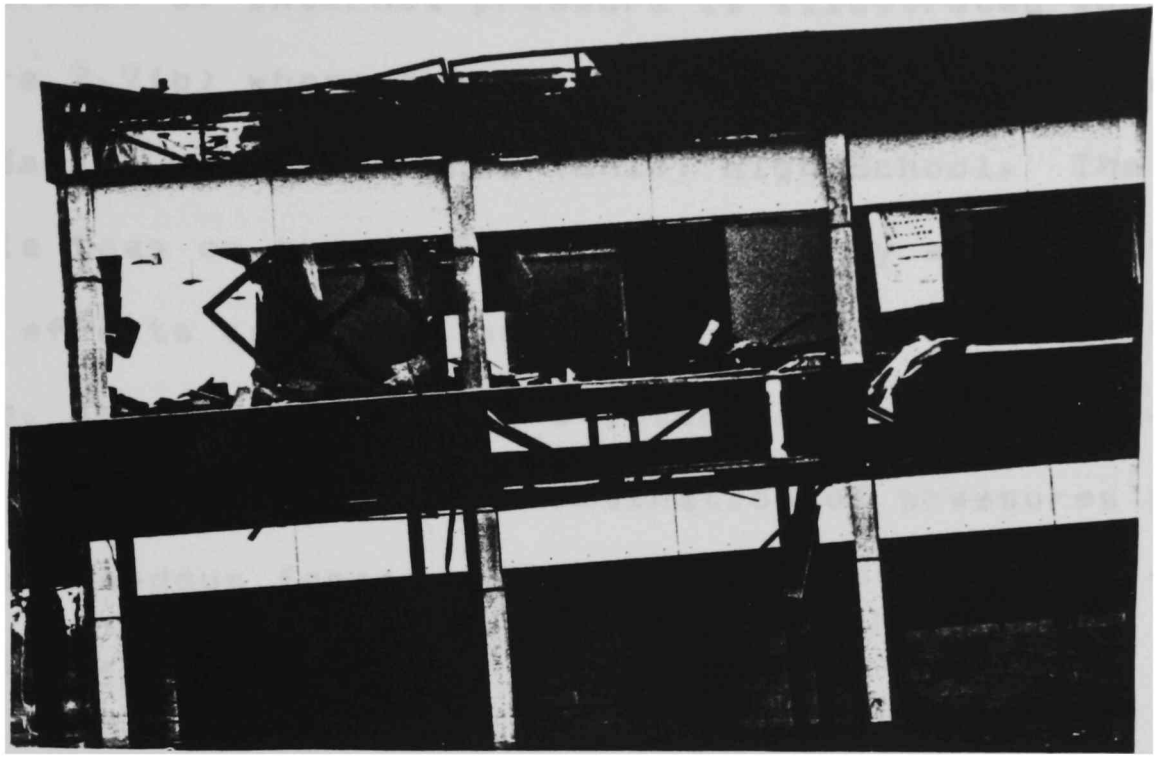
b) Hurricane Alicia, 1983

FIGURE 2.6 LOCAL EFFECTS ON WALL CORNER

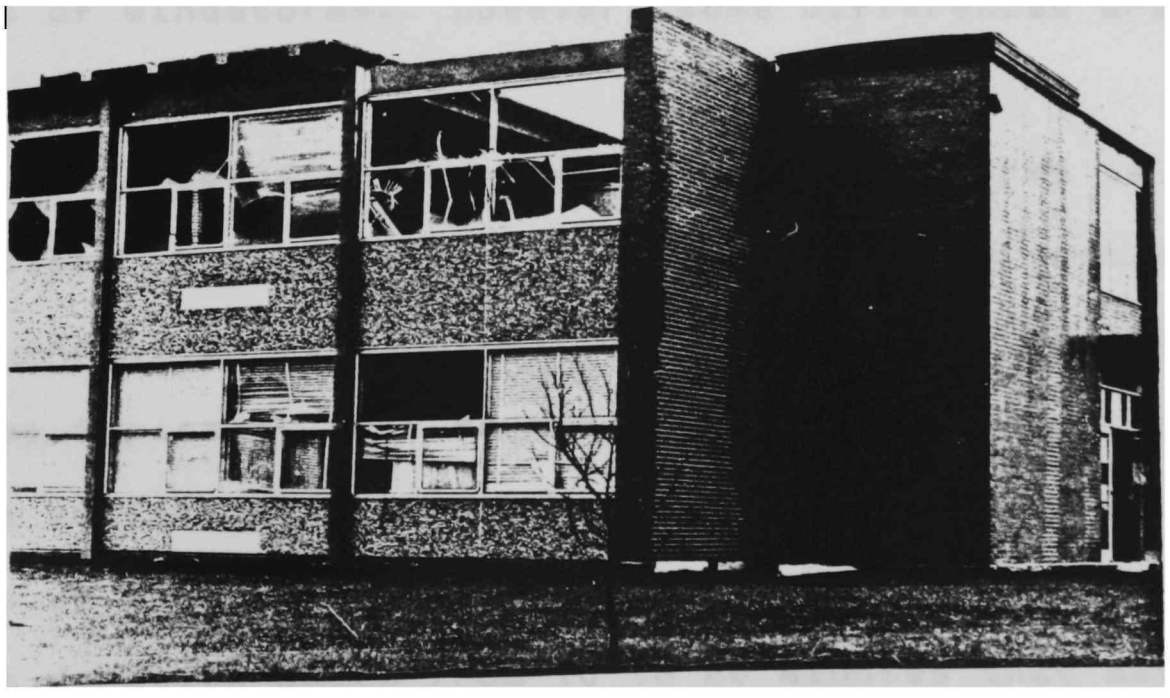


c) Lubbock Tornado, 1970

FIGURE 2.6 (CONT.)



a) Hurricane Wind - Darwin Australia, 1974



b) Tornado Wind - Xenia Ohio, 1974

FIGURE 2.7 INTERNAL EFFECTS ON BUILDING

caused by Cyclone Tracy (hurricane) in Darwin, Australia. The effect of internal pressure is illustrated again in Figure 2.7(b) where the tornado of April 3, 1974 produced the damage shown to Xenia (Ohio) High School. The precast double tees on the roof were uplifted by a combination of wind effects including pressures on the overhang, the eaves, the roof corners and internal pressures because of the broken window. This combination of pressures produced the tremendous forces needed to uplift the double tee sections.

2.3.2 Differences

The material presented in the previous section makes a good case for similarity of damage produced by the three types of windstorms. However, some differences are apparent.

The primary difference in the three storms is their duration. Tornadoes are short lived, rarely lasting more than one to three minutes. Straight winds from thunderstorms typically last no more than 10 to 15 minutes. Large surface lows produce winds that may blow for several hours at speeds up to 50 mph. They may be accompanied by short duration gusts from 10 to 20 minutes that may reach 80 mph. Hurricanes move inland at translational speeds less than 10 mph. Sustained hurricane winds in a large storm can last for several hours.

Tornadoes and hurricanes both have a reduced atmospheric pressure at their centers. Because hurricanes move very slowly there is little chance for rapid application of the pressure difference on a structure. In the case of tornadoes atmospheric pressure change (APC) is a factor on buildings that are specifically dealed. Ordinary buildings that are not purposely sealed have enough openings or natural porosity to prevent rapid pressure buildup from APC.

Objects and debris can be picked up by high winds. The tendency is more predominant with tornadoes, because of the rapid change in velocity of the winds and because of the upward component of wind velocity. Hence wind-borne missiles are of more concern with tornadoes than with hurricanes and straight winds.

2.3.3 Conclusion from Damage Observations

Observations of damage from straight, hurricane and tornado winds support the conclusion that wind pressures as reflected in damage to buildings generally are independent of the type of wind storm. The three kinds of storms produce similar damage. Thus, it appears reasonable to use the same procedures for obtaining design wind-loads.

CHAPTER 3

FEASIBILITY AND ADVANTAGES OF A UNIFORM APPROACH

Evidence was presented in the previous chapter to support the conclusion that damage caused by straight, hurricane and tornado winds appears to be similar. This conclusion implies that the damage mechanisms generally are independent of the type of windstorm.

There are obvious differences in the three windstorms, including duration, APC and wind-borne missiles. These differences do not affect the approach for determining wind pressures on buildings or other structures. Hence, the differences that do exist do not affect the magnitude or distribution of wind induced pressures.

Since the windload provisions of ANSI A58.1-1982 attempt to address the types of observed damage and because they represent a consensus of latest wind technology, the proposed uniform approach uses the general provisions of the ANSI Standard for straight, hurricane and tornado winds.

Straight winds and hurricane winds are already treated in a similar manner in the ANSI Standard. Thus, all that is needed for a uniform approach is to adopt the ANSI procedures for tornado wind pressures. It should be

emphasized that treatment of tornado wind pressures are not mentioned in the ANSI Standard. The methods in the Standard are adopted for determining tornado wind pressures.

Subsequent calculations in two example problems in Chapter 6 will demonstrate that the wind pressure obtained by the proposed uniform approach and not significantly different from those obtained by current tornado design procedures. Where differences occur they can be explained as being more rational or at best more conservative.

The advantage of the uniform approach is that only one method is required to deal with all these windstorms. At a given site the probabilities of the three types of storms are different. The type of windstorm considered in design depends on the type of facility, the geographic location and the exceedance probability associated with the design criteria.

A uniform approach to wind design is needed to assure a consistent design of facilities that have similar functions and values. It further allows for consistent treatment of buildings or facilities that range from ordinary to those that pose a high hazard to people and the environment.

CHAPTER 4

A UNIFORM APPROACH FOR WINDLOADS BY ANSI A58.1-1982

Wind design criteria are based on probability. The approach is to first determine the importance of a particular building or facility under consideration. Factors such as number of people, function of the facilities, value of contents and hazard to people and environment should be taken into account in determining the importance of a facility. By placing a particular facility in a particular category, criteria for wind-resistant design can be established which are consistent with design goals of the category and the grographic location of the site. The criteria are expressed in the form of a wind speed associated with an annual probability of exceedance and an importance factor.

One approach is to consider four building categories that represent different levels of importance of the facilities. The building category descriptions are based on the four categories defined in "Design and Evaluation Guidelines for DOE facilities Subjected to Natural Hazard phenomena" [19].

These categories are used in this report to establish the criteria and to illustrate the uniform approach to

wind-resistant design. The first two categories of the DOE Guidelines are essentially identical to the ordinary and essential facilities as defined in ANSI A58.1-1982. The other two categories relate to moderate and high hazard facilities where containment of dangerous materials is essential. The four building use categories are described in Table 4.1. The DOE Criteria for the four building use categories are summarized in Table 4.2 [11].

Adaptations of the ANSI A58.1-1982 provisions to the three types of windstorms are described in subsequent paragraphs.

4.1. Straight Winds

To determine wind pressures on buildings produced by straight winds, the provisions in ANSI A58.1-1982 are used. The basic wind speeds for buildings or facilities in categories I and II come directly from the ANSI map, which shows the geographic variation of wind speeds associated with a 0.02 annual probability of exceedance (50 year mean recurrence interval). Importance factors of 1.0 and 1.05 are used for Category I and II facilities respectively. A value greater than 1.0 for Category II has the effect of specifying a basic wind speed associated with an annual exceedance probability of 0.01 (100 year mean recurrence interval).

TABLE 4.1

BUILDING USE CATEGORIES

Building Category	Description
I General Use Facilities	All buildings and structures except those listed below
II Essential use Facilities	Buildings and structures where the primary occupancy is one in which 300 people congregate in one area; buildings and structures designated as essential facilities, including, but not limited to: (1) hospital and other medical facilities having surgery or emergency treatment areas (2) fire or rescue and police stations (3) primary communication facilities and disaster operation centers (4) power stations and other utilities required in an emergency (5) structures having critical national defense capabilities
III Moderate Hazard Facilities	Buildings where containment of contents is necessary for public or employee protection, e.g. facilities with high explosives or involving the handling or storage of significant quantities of radioactive or toxic material
IV High Hazard Facilities	Buildings where operating continuity and public and employee protection are of paramount importance, such as facilities handling substantial quantities of in-process plutonium

TABLE 4.2

DOE WIND DESIGN CRITERIA

Building Category	General Use (I)	Essential Use (II)	Moderate Hazard (III)	High Hazard (IV)
Annual Probability of Exceedance	$2x10^{-2}$	$2x10^{-2}$	$1x10^{-3}$	$1x10^{-4}$
Importacne Factor	I=1.0-S I=1.05-H	I=1.07-S I=1.11-H	I=1.0-S I=1.05-H	I= 1.0-S I=1.05-H
Missile Criteria			2x4 timber plank 15 lb @ 50 mph (horiz.); max. height 30 ft.	2x4 timber plank 15 lb @ 50 mph (horiz.); max. height 30 ft.
(This portion is for straight winds and hurricanes only)				
Annual Probability of Exceedance			$2x10^{-5}$	$2x10^{-5}$
Importacne Factor			I=1.0	I= 1.35
APC			40 psf @ 20 psf/sec	125 psf @ 50 psf/sec
Missile Criteria			2x4 timber plank 15 lb @ 100 mph (horiz.); max. height 150 ft; 70 mph (vert.)	2x4 timber plank 15 lb @ 100 mph (horiz.); max. height 200 ft; 100 mph (vert.)
			3 in. dia. std. steel pipe, 75 lb @ 50 mph (horiz.); max. height 75 ft, 35 mph (vert.)	3 in. dia. std. steel pipe, 75 lb @ 75 mph (horiz.); max. height 100 ft, 50 mph (vert.)
				3000 lb automobile @ 25 mph, rolls and tumbles
(This portion of the table is for tornodes only)				

* Importance factor "S" for sites 100 miles from hurricane oceanline and other areas. Importance factor "H" for sites at hurricane oceanline. For sites between hurricane oceanline and 100 miles inland, the importance factor shall be determined by linear interpolation.

Basic wind speeds for facilities in Category III and IV are associated with probabilities less than or equal to 1×10^{-3}. ANSI A58.1-1982 does not provide this type of wind speed data [15]. Site specific hazard assessments are required (see for example Ref [16]). However, once the wind speeds associated with the lower porbabilities are obtained, facilities in Category III and IV can be designed by the procedure of ANSI A58.1-1982. The steps of the ANSI A58.1-1982 procedure include

1) obtain basic wind speed and importance factor
2) calculate effective velocity pressures
3) calculate pressures on main wind-force resisting system
4) calculate pressures on components and cladding.

Details are given in Appendix A.

4.2. Hurricane Winds

The ANSI basic wind speed map is based on measurements of straight winds only. Hurricane hazard assessment has been performed for the region along the Gulf and Atlantic coastlines [20] using Monte Carlo simulation techniques. Comparing of the straight wind and hurricane hazard models showed that for buildings in Category I and II, equivalent hurricane design wind speeds can be obtained simply by multipling the basic 50 year mean recurrence interval wind speeds from ANSI A58.1-1982 by an

appropriate importance factor value. The approach avoids the need for a separate hurricane hazard map.

Basic hurricane wind speeds for design of Category III and IV buildings or facilities must come from a site specific hazard assessment. Data for hurricane hazard assessment are available [27]. An example of hurricane hazard assessment is contained in Ref [31].

Hurricane wind speeds reduce quickly after the hurricane moves inland, because of increased surface roughness and because the hurricane's source is no longer present. Hurricane hazard assessments are made at the coastlines. At 100 miles inland, hurricane winds are no longer considered a threat. A linear reduction of wind speed, because of the reduction of importance factor, from coastline to 100 miles inland is permissible. Once the basic hurricane wind speed is established, wind pressures are obtained following the ANSI A58.1-1982 procedures. Wind speeds are fastest mile.

4.3 Tornadoes

A tornado hazard probability model is determined from statistical analysis of records of tornadoes that have occurred in the region surrounding the site of interest. The tornado data used are expressed in terms of the Fujita-Pearson scales [9]. Various tornado hazard probability models have been published in the literature. They

tend to be of two types: those that consider a point probability and those that consider the aerial extent of a site. Point probability models have been proposed by Thom (1963) [21], Markee et al. (1974) [22], Abbey and Fujita (1975) [23] and McDonald (1978) [24]. Aerial probability models have been suggested by Cornell et al. (1975) [24], Twisdale (1978) [25] and Reinhold and Ellingwood (1982) [26]. If probability interpreted, either a point probability or the aerial probability model can be used for design. Each approach has advantages and disadvantages. The DOE criteria in Table 4.2 is based on the point probability assessment using the method proposed by Fujita (1975).

Because tornadoes are relatively rare events, straight wind speeds control except when the criteria call for annual exceedance probabilities less than approximately $1x10^{-4}$. Hence, tornadoes do not govern the design of Category I and II facilities. The geographical location of a site determines if tornado-resistant design must be provided for Category III and IV facilities.

4.3.1 Tornado Wind Speeds

Wind speeds in ANSI A58.1-1982 for straight and hurricane winds are based on fastest-mile wind speeds. Therefore, in the uniform approach, tornado wind speeds also must be fastest-mile wind speeds. Tornado wind

speeds are estimated from appearance of damage using the Fujita-Scale, which is interpreted to be peak gust speeds (2-3 seconds averaging time). Thus, tornado wind speeds must be converted to equivalent fastest mile wind speeds for use in the uniform approach. The differences between the peak gust and fastest-mile wind speed is because of the averaging time. A relationship between tornado wind speed V_{tor} and equivalent fastest-mile wind peed V_{fmw} is given by

$$V_{fmw} = 0.9575\ V_{tor} - 11.3430. \qquad (4.1)$$

Equation (4.1) is developed in the following manner. A relationship developed by Durst is first used to convert the peak gust tornado wind speed to mean hourly. A trial and error procedure is then required to convert from mean hourly to fastest-mile wind speed, because of the variable averaging time. After doing this for a range of tornado wind speeds from 90 through 300 mph, the relation between tornado wind speed and fastest-mile wind speed (Eq. [4.1]) is obtained by linear regression. The details for converting tornado wind speeds to fastest-mile wind speeds are described in Appendix C.

4.3.2 Tornado Wind Pressures

Tornado wind pressures are obtained in a four-step procedure analogous to that for straight winds and hurricane winds in ANSI A58.1-1982.

1) Obtain basic wind speed for tornado resistant design from hazard probability model in terms of equivalent fastest-mile wind speed. Obtain importance factor that is appropriate for building-use category III or IV from Table 4.2.

2) Calculate effective velocity pressure q using equations in ANSI A58.1-1982.

3) Calculate tornado wind pressures for Ordinary Buildings and Structures (as defined in ANSI A58.1-1982) for main wind-force resisting systems.

4) Calculate tornado wind pressures for components and cladding.

The basic wind speed for tornado resistant design is obtained from a site-specific tornado hazard assessment. The peak gust tornado wind speed is converted to fastest-mile wind speed using Equation (4.1). The importance factor is given in Table 4.2.

Selection of the importance factor for Category III and IV buildings is a new concept for tornado design and requires an explanation. Because of large uncertainties in tornado hazard assessments, the tendancy of regulatory agencies has been to set very low annual exceedance probabilities for hazardous facilities of the order $1x10^{-4}$ to $1x10^{-7}$. The approach requires considerable extrapolation of the data from existing tornado records. The philosophy introduced in the DOE Guidelines is to specify a relatively high annual exceedance probability, which minimizes the extropolation and then apply an arbitrary importance factor to the resulting wind speed to obtain additional conservatism in the criteria.

The effective velocity pressure q is calculated using appropriate equations as specified by ANSI A58.1-1982. The value of q may be determined at 30 ft, at any height z or at mean roof height h. The variation of wind speed with height and terrain roughness is adjusted by using velocity pressure exposure coefficient K. For determining tornado wind pressures, values of K for terrain Exposure C is recommended in the DOE Guideline, regardless of actual terrain condition.

To obtain the wind pressures on tributary areas of main wind force resisting systems, the effective velocity pressure is multiplied by a gust response factor G and a pressure coefficient C_p. The gust response factor comes from a table in ANSI A58.1-1982 and depends on terrain roughness and height z. The pressure coefficients are obtained from ANSI and depend on which part of the building the wind pressure is desired i.e., windward wall, leeward wall or roof.

If the building is vented (not speccificially sealed), it may be necessary to consider effects of internal pressure following the guidance of ANSI. If the building is sealed, the atmospheric pressure change associated with the tornado vortex must be taken into account (see next section).

Tornado wind pressures on components and cladding are obtained using the appropriate ANSI equations. Values of

the gust response factor and the pressure coefficient are combined in a single value (GC_p). Figures in ANSI give values, depending on the specific location on the building and the size of the tributary area. They are independent of height above ground.

4.3.3 Atmospheric Pressure Change

Atmospheric pressure change (APC), is a function of the radial distance from the tornado core. It is a reduction in pressure from ambient at a large distance from the tornado center. The maximum pressure change occurs at the center of the core and is given by

$$P = \rho V_{em}^2 \qquad (4.2)$$

where

P = pressure change from ambient (psf)

ρ = the mass density of air at standard conditions (0.00238 slug/ft^3)

V_{em} is the maximum tangential wind speed (ft/sec)

The value of V_{em} is approximated by the equation

$$V_{em} = (V_{tor} - 50)(0.89)(1.47)$$
$$= (V_{tor} - 50)(1.305) \qquad (4.3)$$

V_{tor} = maximum tornado design wind speed (peak gust, not fastest-mile) in mph

The term V_{em} is the main tangential wind speed. Equation (4.3) comes from Ref [17]. It is based on an assumed relationship between the vertical, radial and tangential components of the tornado wind velocity vector. The

constant 50 is an assumed tornado translational velocity of 50 mph. The term 1.47 converts wind speed in mph to ft/sec.

4.3.4 Load Combinations

The wind speed at the center of a tornado vortex is theoretically zero. Thus, when a large tornado engulfs a relatively small sealed building, the pressure on the outside of the building is less than on the inside by an amount P given by Equation (4.2). The pressures are outward acting and are not combined with wind pressures.

Another situation that should be checked is the combination of tornado wind pressure plus one-half maximum atmospheric pressure change pressure. This situation occurs at the radius of maximum wind speed and may be a worse case than atmospheric pressure change alone. All of these conditions should be checked when the building is sealed.

4.3.5 Tornado-Generated Missiles

High winds are capable of picking up and transporting various types of objects and debris that can range in size from small objects such as roof gravel to large, heavy objects such as planks, pipes or steel beams. Tornadoes are more likely to pick up debris, because of their vertical wind component and higher wind speeds. Missiles may

break windows, perforate exterior walls or cause stability-type failures of walls, frames or columns. Openings created by missile perforation of an enclosed building can lead to the buildup of internal pressure. Thus, the potential for missile impact may require a building to withstand internal pressure.

The DOE Design Guidelines [11] specify the missiles to consider for building use Categories III and IV. Because straight winds or weak tornadoes may generate missiiles, a minimum missile criterion is specified even when tornado resistant design is not required.

For the treatment of missile impacts, see Baily (1984) [28], White (1986) [29] and McDonald (1976) [30] for guidance on design missile for impacts.

CHAPTER 5

CURRENT METHOD FOR TORNADO DESIGN

The current tornado design method is one that has evolved from research at the Institute for Disaster Research (IDR), Texas Tech University. It is generally accepted for tornado resistant design of all categories of buildings and facilities. The current method is described in detail in this chapter so that the results from the current method and the proposed uniform method can be compared in Chapter 6.

5.1 Design Wind Speeds

The tornado design wind speed must be obtained from a site-specific tornado hazard probability model. The various models mentioned in Section 4.3 can be used with the current design procedure. Appropriate probability of exceedance must be specified for the type of building being considered. Values in Table 4.2 should not be used. The peak gust wind speeds obtained directly from the tornado hazard model are used in the analysis. Importance factors do not come into play.

Tornado wind pressures are obtained in a four-step procedure that is somewhat analogous to the procedure in ANSI A58.1-1972. The exceptions are noted below.

1) Obtain appropriate basic wind speed from hazard probability model. The model gives the wind speed in terms of peak gust.

2) Calculate the effective velocity pressure using the equation for staganation pressure.

3) Calculate the overall tornado wind pressures on windward, leeward, side walls and roof. Use values of C_p in ANSI A58.1-1972.

4) Calculate local tornado wind pressures on wall corners, eaves, ridges and roof corners using pressure coefficients from ANSI A58.1-1972.

5.2 Tornado Wind Pressures

The effective velocity pressure q is given by the equation

$$q = 0.00256\ V^2 \qquad (5.1)$$

No distinction is made for terrain roughness or variation with height. The tornado wind speed is assumed to be constant with height up to 200 ft. Thus, Equation (5.1) does not contain a K term.

Overall wind pressures are calculated using the equation

$$p = qC_p \qquad (5.2)$$

where C_p is given in ANSI A58.1-1972 for windward, leeward, side walls and roof. These pressures are used to determine loads on the main wind force resisting system.

Local pressures on wall corners, eaves, roof corners and roof ridges are calculated using an equation similar to Equation (5.2) with appropriate values of C_p. These

pressures act over areas whose width is 0.1 times the least width of the building (ft). Local pressures are not combined with overall pressures. They are used to design connections and anchorage as well as components and cladding.

5.3 Atmospheric Pressure Change

The expressions for APC are the same as those given in Section 4.3.

5.4 Load Combinations

The combinations are the same as described in Section 4.4.

5.5 Missile Impacts

Tables 5.1 and 5.2 list the tornado generated missiles that are considered for the design of structures by the current method.

This list of missiles is not the same as the one specified by the DOE Guidelines. The DOE list does not contain the utility pole.

TABLE 5.1

WINDBORNE MISSILE PARAMETERS

Missile	Weight (lb)	Maximum Projected Area (ft^2)	Maximum Cross Sectional Area (ft^2)
Timber Plank 2 in x 4 in x 12 ft	15.0	3.63	0.0410
3 in Dia. Std. Steel pipe x 10 ft	75.8	2.92	0.0155*
Utility Pole 13.5 in Dia. x 35 ft	1490.0	39.40	0.9900
Automobile	4000.0	100.00	20.0000

* Value given is metal area. In penetration calculations the gross cross sectional area may be used

TABLE 5.2

WINDBORNE MISSILE IMPACT SPEEDS

	Horizontal Missile Speed, mph						Maximum
Design Wind Speed	100	150	200	250	300	350	Height, ft
Timber Plank	40	70	100	150	200	250	250
3 in Dia. Std. Pipe	**	50	65	85	110	140	100
Utility Pole	**	**	**	80	100	130	30
Automobile	*	**	**	25	45	70	30

* A vertical missile speed equal to 2/3 the specified horizontal speed should be used for impact on roofs. Do not combine horizontal and vertical missile speeds vectorially

** Missile will not be picked up or sustained by the wind

CHAPTER 6

COMPARISON OF CURRENT AND PROPOSED METHODS FOR TORNADO LOADS

This chapter presents a comparison of results from the current approach and the proposed uniform approach for determination of tornado pressures. Two cases are presented for comparison. Case I considers a moderate hazard facility based on the DOE criteria consisting of a one-story rectangular building with a flat roof. Results are compared for the main wind-force resisting system and components and cladding. Case II considers a building in the high hazard category. The rectangular building is similar to Case I except it is 60 ft high instead of 20 ft.

6.2 Case I: Moderate Hazard Facility

Because the current method does not consider an importance factor, it is necessary to select equivalent wind speeds so that a fair comparison is obtained. Rather than select a wind speed from a hazard model, a value of 200 mph (peak gust) is arbitrarily assumed for use with the current method. The corresponding wind speed for the uniform approach is obtained by first converting to equivalent fastest-mile wind speed. Two hundred mph peak gust

converts to 180 mph fastest-mile [Equation (4.1)]. For a moderate hazard facility, the importance factor is 1.0. Thus, the corresponding design wind speed for the uniform approach is 180 mph (fastest-mile).

Figure 6.1 shows details of the structure. The wind blows from from south to north in the example. Rigid steel frames span in the 100 ft direction. They are spaced at 25 ft on center. Braced frames contained in the exterior walls are used in the east-west direction. The roof is a 22 gage composite metal deck with a 4-in concrete topping. The walls are constructed of 12 in. reinforced concrete blocks. The walls span 20 ft vertically from roof to floor. The surrounding terrain is assumed to be flat and open corresponding to Exposure C.

The building has doors and windows located in each of the four walls, which could be perforated by missiles or broken by excessive wind pressures. Therefore, the internal pressure should be considered in determining worst case wind loading. With this situation, APC is not a factor. Details of the calculations for Case I by the current and proposed uniform method are given in Appendix B.

Results from the two methods are compared in Table 6.1. The pressures on windward, leeward, side wall and roof are compared in the upper part of the Table. Pressures from the current method are slightly more than 20

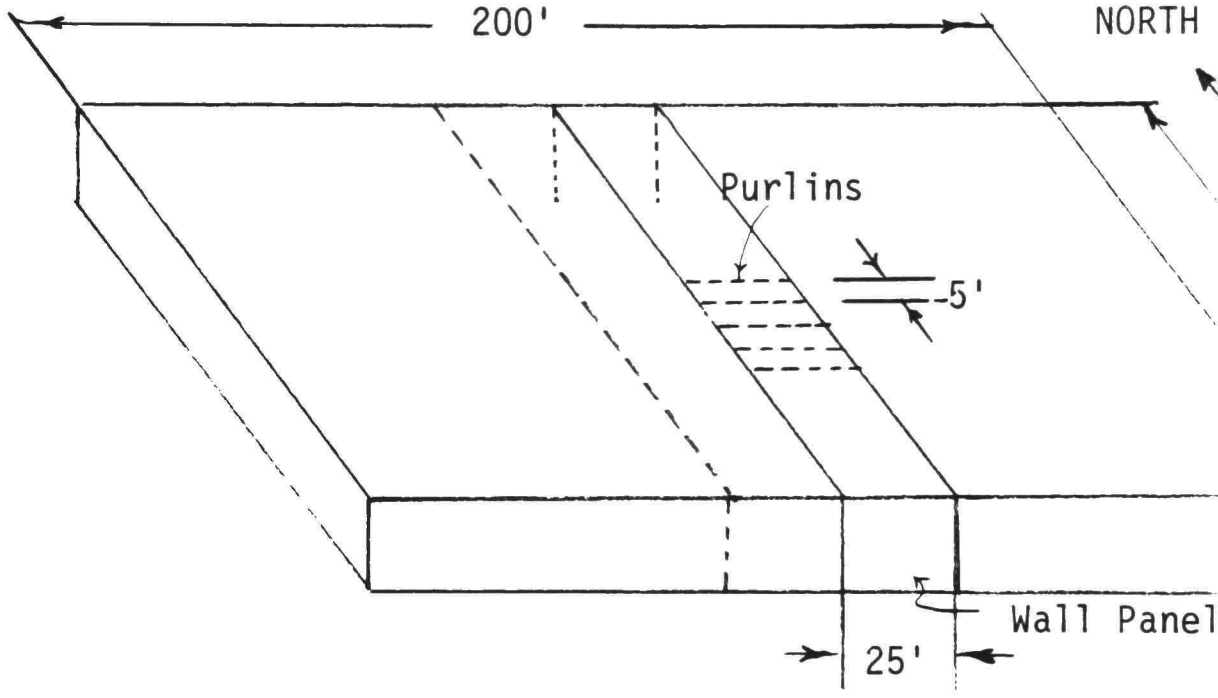


FIGURE 6.1 BUILDING FOR CASE I

TABLE 6.1

COMPARISON OF RESULTS FROM
CURRENT AND PROPOSED METHODS FOR CASE I

System	Location	Design Pressures (psf)	
		current method	proposed uniform method
Main Wind-Force Resisting System	Windward Wall	113	92
	Leeward Wall	-82	-64
	Side Walls or Roof	-102	-83
Components & Cladding	Eaves	-276	-141
	Roof Corners	-542	-160
	Wall Corners	-235	-131

* The comparison is based on a flat building (Exposure C, Category III, h=20 ft, tornado gust wind speed = 200 mph)

* The design pressure values for each method are the maximum values calculated from the same part of the building

* Uniform openings are assumed, internal pressures are considered

percent higher than those given by the proposed uniform method. The effective velocity pressures are 102 and 72 psf for the current and proposed uniform methods, respectively. The value obtained by the current method is 1.42 times larger than the value by the proposed uniform method. The product of $K_h G_h$ is only 0.87 (1.29) =1.12. Thus, the current method gives values of overall pressures 20-30 percent greater than the proposed uniform method. The difference in peak gust tornado wind speed and fastest-mile wind speed decreases with increase in tornado wind speed. Therefore the differences in pressure by the current and proposed methods will decrease at higher wind speeds. The pressures are proportional to the square of the wind speed. The product of $K_h G_h$ is about constant, because K increases with height, but G decreases. The difference in overall pressures obtained by the two methods are not enough to seriously affect the safety of a building designed by the proposed uniform method.

For local pressures a direct comparison of values is not easy, because the pressures used in the proposed uniform approach are obtained in a different manner and also depend on tributary areas. Pressure coefficients for components and cladding in ANSI A58.1-1982 and hence in the proposed uniform approach are measured directly in the wind tunnel as the product of pressure coefficient and gust response factor, (GC_p). The values are a function of

tributary area. Difference in these measured values and the tabulated values in ANSI-1972 are the main reasons for differences in the local pressures by the two methods. Because the ANSI A58.1-1982 values are believed to be more representative of actual pressures (since they are measured directly in the wind tunnel), the lower values obtained with the uniform approach are believed to be safe and acceptable. The largest differences are at roof corners. The pressure coefficient value of -5.0 at flat roof corners in ANSI 1972 has been criticized by several organizations as being too high, which explains why the difference in pressures at roof corners by the two methods is the largest of the three values.

6.2 Case II: High Hazard Facility

The basic wind speed for use with the current method is arbitrarily assumed to be 250 mph (peak gust). The corresponding wind speed for use with the proposed uniform appproach is obtained by first converting the 250 mph peak gust to a 228 mph fastest-mile wind speed. This value is then divided by 1.35, which is the importance factor for high hazard facilities according to the DOE Guideline Criteria, to obtain the basic wind speed of 169 mph for use with the proposed uniform method.

The structure for Case II is identical in size to that of Case I except the roof height is 60 ft instead of

20 ft. Story heights are assumed to be 20 ft (see Figure 6.2).

Because of the need for containment, this building is constructed of reinforced concrete. Reinforced concrete frames span 20 ft in each direction. Two-way slabs are used as floor and roof panels as well as wall panels.

The surrounding terrain is assumed to be Exposure C. The building is sealed and the walls are capable of resisting design missiles without perforation. Thus, APC should be considered as a possible loading condition.

To calculate the APC, refer to Eq. (4.2) and (4.3) in Chapter 4. Details of the calculations for Case II by the current and proposed uniform method are given in Appendix B.

Results from the two methods are compared in Table 6.2. The pressures on windward, leeward, and side walls and roof are compared in the upper part of the Table. Design pressures from the proposed uniform method tend to increase more than those of the current method. As a result, the design pressures of the proposed uniform method appear to be about 20 percent larger than those from the current method. Since the differences in design pressures from these methods are very small, they can be ignored. Therefore, the uniform approach are believed to be safe and acceptable again.

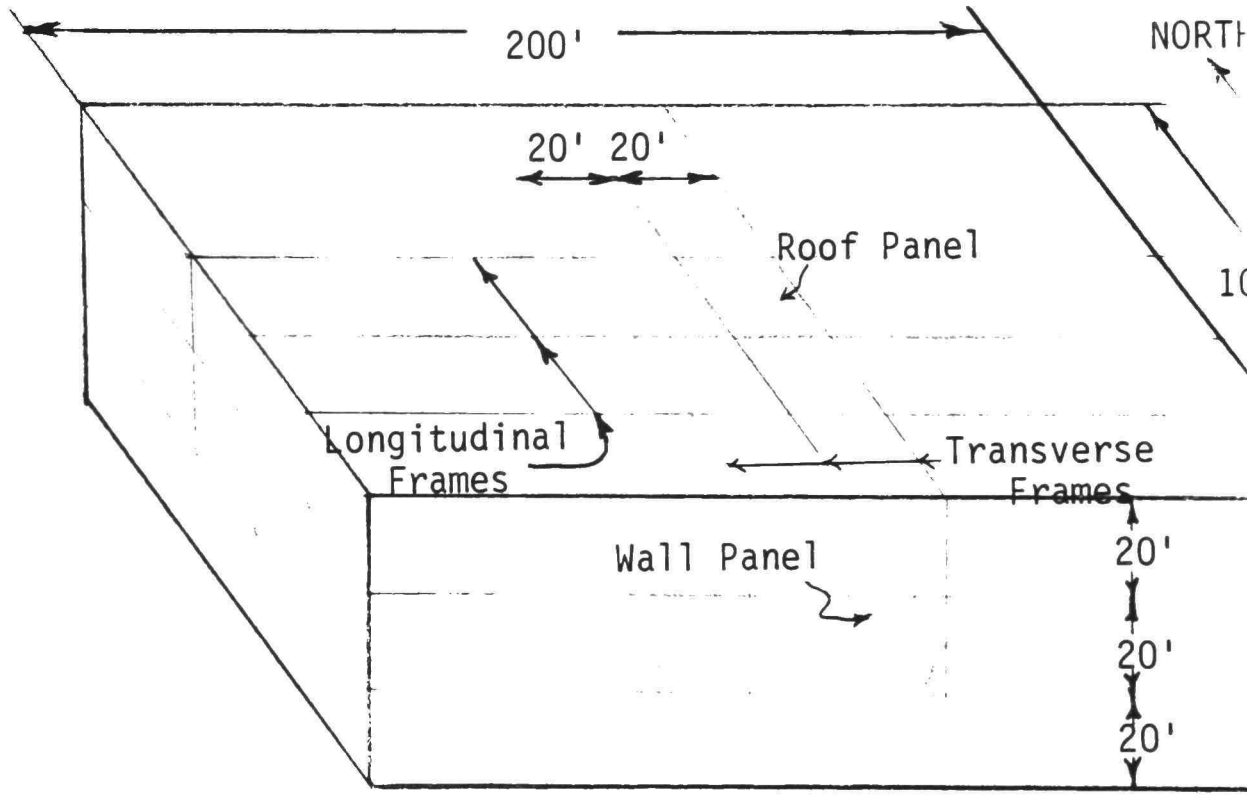


FIGURE 6.2 BUILDING FOR CASE II

TABLE 6.2

COMPARISON OF RESULTS FROM
CURRENT AND PROPOSED METHODS FOR CASE II

System	Location	Design Pressures (psf)	
		current method	proposed uniform method
Main Wind-Force Resisting System	Windward Wall	128	152
	Leeward Wall	-161	-176
	Side Walls or Roof	-193	-214
Components & Cladding	1. eaves	-465	-372
	2. roof corners	-881	-415
	3. wall corners	-401	-350

* The comparison is based on a flat building (Exposure C, Category IV, h=60 ft, tornado gust wind speed = 250 mph)

* The design pressure values for each method are the maximum values calculated from the same part of the building

* Since the building is sealed, APC is considered

For local pressures, the comparison is very similar to the Case I example. Because the design pressures of the proposed uniform method tend to increase much more than those of the current method, the differences in design pressures for these two methods appear to be smaller. The largest differences are at the roof corners once again. The differences in local pressures from these two methods are considered to be satisfactory. Therefore, the uniform method can be concluded as safe and acceptable for local pressures.

6.3 Conclusion

There are many differences in factors affecting the tornado design between these two methods. For low-rise structures, the variation of tornado wind speed in the current method is constant while that of the proposed method is not; the gust response factor of the current method is taken as unity while that of the suggested method is varied with height; and the ways to obtain the wind speed of the two methods are different. Thus, different results are naturally expected.

If the design pressures of the proposed method are much larger than those of current one, the proposed method will be more expensive than the current one. Since the current method is considered to be conservative, a new,

more conservative and more expensive method would not make sense.

If results indicate that the design pressures of the proposed method are much smaller than the current one, the proposed method would be less conservative and less expensive than the current one, but the design might be unsafe.

If results indicate that the design pressures of the proposed method are almost the same as those of current one, the proposed method would be preferred over the current one, because the methodology of this new method is rational and similar to the method for dealing with other windstorms.

Tables 6.1 and 6.2 show that values of overall wind pressure on the windward wall, leeward wall, side walls or roof, and local pressures on eaves, roof corners and wall corners of the building are only slightly different for the two tornado design methods. The results of these two examples, with different building categories, heights and design wind speeds, confirm that the proposed method is as good as the current one, and for reasons mentioned above is the preferred method.

CHAPTER 7

CONCLUSION

A uniform approach for design of structures for wind-loads is proposed in this study. The current design procedure for straight and hurricane winds is addressed by the provisions of ANSI A58.1-1982. However, tornado winds are not considered in the ANSI Standard. Therefore, the main point is to propose a new tornado design method that essentially follows the ANSI A58.1-1982 Standard.

The provisions of ANSI A58.1-1982 attempt to address the physical characteristics of wind effects on structures as observed in wind tunnels, full-scale measurements and post-storm damage investigations. Because the damage from straight, hurricane and tornado winds appears to be similar, it appears to be feasible to present a tornado design approach that is also based on the ANSI A58.1-1982 Standard, thus providing a uniform approach to wind design for straight, hurricane and tornado winds.

Comparing the current and proposed tornado design methods, the wind pressures and load combinations of wind pressure and APC and missile impacts appear to be similar. Two examples are presented to verify that wind pressures on buildings are comparable when calculated by these two design methods. This implies that the proposed tornado

design method is as good as the current one. However, for the purpose of having only one method deal with all types of windstorms, the suggested tornado design method is better than the current one. Therefore, the uniform approach to design of windloads on structures is strongly proposed.

A recommendation is suggested to the coming researchers. Somewhere in the earth, people are suffering from natural phenomena not only for the windstorms but also from the earthquakes and floods. If a uniform approach to design loads on structures is possible, it will be easier and more convenient for Civil Engineers.

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APPENDICES

APPENDIX A

DETAILED PROVISIONS OF ANSI A58.1-1982 FOR WINDLOAD DESIGN

1) Design Pressure

The format of the design pressures, which are determined through analytical procedure using equations, is simplified by putting the basic equations for different types of buildings and structures in a single table (Table 4 of the ANSI Standard). The format of equations for design pressures and forces are:

$$p = qGC \qquad (A.1)$$

and

$$F = qGCA \qquad (A.2)$$

because of different definitions of the terms in the above equations and different combinations of the terms applicable to a specific structure, there are several equations in Table 4 of ANSI Standard. Definitions of the terms in Equations (A.1) and (A.2) are specified as following:

p = design pressure in psf; positive value means acting toward the surface; negative value means action away from the surface.

F = design force in lbs.

q = velocity pressure in psf; q_z is the value edtermined at height z (ft) above ground, q_h is the value determined at mean roof height h (ft).

G = gust response factor (dimensionless); G_z is the value determined at mean roof height h (ft) above ground, G_h is the value determined at mean roof height h (ft).

C = pressure coefficient (dimensionless); positive value means acting toward the surface, negative value means acting away from the surface; C_p is external pressure coefficient; C_{pi} is internal pressure coefficient, and C_f is force coefficient.

With mean roof heights of 60 ft or less, there is an individual equation to determine design pressure for components and cladding of buildings. Because of the extensive wind tunnel and limited full-scale data available for buildings with heights of less than 60 ft, this special category is necessary.

2) Velocity Pressure q

The velocity pressure, q, in psf is determined by following equations:

$$q_z = 0.00256K_z(IV)^2 \qquad (A.3)$$

$$q_h = 0.00256K_h(IV)^2 \qquad (A.4)$$

where V is wind speed in mph, I is the importance factor, K_z and K_h are the velocity pressure exposure coefficients at heights z and h, respectively.

3) Wind Speed V

The basic wind speed V used in the determination of design windloads on buildings and other structures shall be as given in ANSI Fig. 1 for the contiguous United States and Alaska.

4) Importance Factor I

The importance factor I accounts for the degree of hazard to human life and damage to peoperty. It is used to modify the wind speed.

5) Velocity Pressure Exposure Coefficient K

The velocity pressure exposure coefficient takes into consideration changes in wind speed with height above ground and with the nature of the surroundings (types of terrain). The wind speed varies with height because of ground friction, and that the amount of friction varies with the ground roughness. There are four roughness categories or exposures defined in the ANSI sstandard; shown as following:

a) Exposure A represents centers of large cities and very rough terrain;

b) Exposure B represents suburban areas, towns, city outskirts, wooded areas, and rolling terrains;

c) Exposure C reprents flat open country and grass-land; and

d) Exposure D reprents flat, unobstructed coastal areas directly exposed to wind blowing over bodies of water.

The wind speed variations with height and exposure are determined by the following equation:

$$V_z = V_{33}(z/33)^{1/a} \qquad (A.5)$$

where

z = elevation in feet; practical limits $15' < z < z_g$

V_{33} = wind speed in mph at 33 ft

a = coefficient depending on exposure (ANSI Table A6)

Wind velocity is constant at some gradient height, z_g. The gradient heights z_g for different exposure are given in ANSI Table A6. Since the data used in developing wind speed contour maps were obtained primarily at airports (Exposure C), the wind speeds for other exposures should be obtained from the gradient wind speed. Therefore, the wind speed at any height and for any exposure can be obtained by utilizing the equation:

$$V_z = [V_{33} \ (900/33)^{1/7}] \ (z/z_g)^{1/a} \qquad (A.6)$$

where z_g is the gradient height in feet (ANSI Table A6). The term within brackets converts the basic wind speeds to gradient wind speed; the second term (in parenthesis) determines the wind speed for any exposure and height.

K_z is a pressure coefficient rather than wind speed coefficient; hence, the wind speeds have to be squared to obtain the pressure coefficients. The equation for K_z becomes:

$$K_z = (V_z/V_{33})^2 = (900/33)^{2/7} \ (z/z_g)^{2/a} \qquad (A.7)$$

or

$$K_z = 2.58 \ (z/z_g)^{2/a} \qquad (A.8)$$

Values of K_z are tabulated in the ANSI-1982 Table 8. 500 ft above ground. For values of height z greater than 500 ft, Equation 8 can be used. Below the height of 15 ft, the values of K_z are taken as constant (obtained at 15 ft) because of increased turbulence near the ground. Even

though the ground friction tends to reduce the mean wind speed closer to the ground, the turbulence tends to increase. Although K_z is assumed to be constant, quantitative effects of this turbulence are not known.

6) Gust Response Factor G

Gust response factors (GRF) are used to account for the fluctuating nature of wind and its interaction with buildings and other structures. GRF do not include allowances for the effects of acorss-wind deflection, vortex shedding, or instability due to galloping or flutter. In some cases GRF are combined with pressure coefficients to produce values of GC_p and GC_{pi}; in these cases GRF shall not be determined separately.

For the purposes of clarity and simplification in the ANSI Standard, GRF are specified as G_z and G_h. The functions of each one are specified as shown below:

G_h = for the use of main wind-force resisting systems; the value of G_h is determined by the mean height (h) of the building (see ANSI Table 8),

G_z = for the use of components and cladding, the value of G_z shall be determined by the height (z) above ground (see ANSI Table 8).

7) Pressure Coefficients C

The values of pressure coefficient are based on recent turbulent boundary layer wind tunnel tests. Values obtained under smooth flow conditions may not be valid under turbulent flows. For enclosed buildings, pressure

coefficient values for components and cladding are based on recent boundary layer wind tunnel tests; the values have good validity.

The values of pressure coefficient are given in the ANSI Tables 9 through 16 and Figures 2 through 4. It is very important to use the proper sign of the pressure coefficient values. Whenver the sign of $\pm$ is specified, both positive and negative values should be checked to obtain controlling loads. "+" (plus sign) means positive pressure acting toward the surface, and "-" (minus sign) means negative pressure acting away from the surface. For building components and cladding, the values of pressure coefficient (ANSI Figures 3) are dependent on location of component on the building surface and on the tributary area of the component. It is recognized that peak pressures occur over a small area at any given time; average pressures become smaller as the area gets larger. Besides, the values shown in the ANSI Table 9 and Figuers 3 are combined gust response factor and pressure coefficient (GC_p). The gust response factors were measured with pressure coefficient values in wind tunnel experiments; G should not be separated from C_p in these cases.

APPENDIX B

CALCULATIONS BY CURRENT AND PROPOSED UNIFORM METHODS

The current method and proposed uniform method are used to calculate tornado wind and APC pressures (where applicable) for the two example problems in Chapter 6.

B.1 Design Pressures by Current Method

Steps involved in the current method are described in Section 5.1.

B.1.1 Case I: Current Approach

STEP 1: Basic Wind Speed

V = 200 mph (peak gust - north/south direction)

STEP 2: Velocity Pressure

$q = 0.00256\ V^2$

$= 0.00256 \times 200^2$

$= 102$ psf

STEP 3: Overall Design Pressures

$p = q\ (C_p - C_{pi})$

C_p : External Pressure Coefficients from ANSI A58.1-1972, Section 6.5.3

Windward Wall, $C_p = +0.8$

Leeward Wall, (h/L and h/B < 2.5), $C_p = -0.5$

Side Walls, $C_p = -0.7$

Roof, (h/L and h/B (2.5), $C_p = -0.7$

C_{pi} : Internal Pressure Coefficient from ANSI A58.1-1972 Table 11 $C_{pi} = \pm 0.3$

Windward Wall

$$p = 102\ (+0.8 \pm 0.3)$$

$$= 82 \pm 31$$

$$= 113 \text{ psf (inward acting)}$$

$$= 51 \text{ psf (inward acting)}$$

Leeward Wall

$$p = 102\ (-0.5 \pm 0.3)$$

$$= -51 \pm 31$$

$$= -20 \text{ psf (outward acting)}$$

$$= -82 \text{ psf (outward acting)}$$

Side Walls & Roof

$$p = 102\ (-0.7 \pm 0.3)$$

$$= -71 \pm 31$$

$$= -40 \text{ psf (outward acting)}$$

$$= -102 \text{ psf (outward acting)}$$

STEP 4 Local Design Pressures

$$p = q\ (C_p - C_{pi})$$

C_p : External Pressure Coefficients from ANSI A58.1-1972, Section 6.5.3

Wall Corners, $C_p = -2.0$

Eaves, $C_p = -2.4$

Roof Corners; $C_p = -5.0$

C_{pi} : Internal Pressure Coefficients
Uniform Openings, ANSI A58.1-1972, $C_{pi} = \pm 0.3$

Wall Corners

$$p = 102\ (-2.0 \pm 0.3)$$

$$= -204 \pm 31$$

$$= -173 \text{ psf (outward acting on a strip 10 ft wide)}$$

$$= -235 \text{ psf (outward acting on a strip 10 ft wide)}$$

Eaves

$$p = 102\ (-2.4 \pm 0.3)$$

$$= -245 \pm 31$$

$$= -214 \text{ psf (outward acting on a strip 10 ft wide)}$$

$$= -276 \text{ psf (outward acting on a strip 10 ft wide)}$$

Roof Corners

$$p = 102\ (-5.0 \pm 0.3)$$

$$= 510 \pm 31$$

$$= -479 \text{ psf (outward acting on a strip 10 ft wide)}$$

$$= -542 \text{ psf (outward acting on a strip 10 ft wide)}$$

B.1.2 Case II: Current approach

STEP 1: Basic Wind Speed

$V = 250$ mph (peak gust - north/south direction)

$V_t = 50$ mph (assumed translational speed)

$V_{om} = (V_{tor} - 50)(1.305)$ [Eq. (4.3)]

$= (250 - 50)(1.305)$

$= 261$ fps

STEP 2: Velocity Pressure

$q = 0.00256\ V^2$

$= 0.00256 \times 250^2$

$= 160$ psf

STEP 3: Overall Design Pressures

$P_{APC} = -\rho V_{em}^{\ 2}$

$= -0.00238 \times (261)^2$

$= -162$ psf

$p = qC_p$ (for windward wall)

$p = qC_p + 0.5\ P_{APC}$ (for other walls and roof)

Windward Wall

$p = 160\ (+0.8)$

$= +128$ psf (inward acting)

Leeward Wall

$p = 160\ (-0.5\ ) - (0.5)(162)$

$= -80 - 81$

$= -161$ psf (outward acting)

Side Walls & Roof

$p = 160\ (-0.7) - (0.5)(162)$

$= -112 - 81$

$= -193$ psf (outward acting)

STEP 4: Local Design Pressures

$p = qC_p + 0.5\ P_{APC}$

Wall Corners

$$p = 160\ (-2.0) - (0.5)(162)$$

$$= -320 - 81$$

$$= -401 \text{ psf (outward acting on a strip 10 ft wide)}$$

Eaves

$$p = 160\ (-2.4) - (0.5)(162)$$

$$= -384 - 81$$

$$= -465 \text{ psf (outward acting on a strip 10 ft wide)}$$

Roof Corners

$$p = 160\ (-5.0) - (0.5)(162)$$

$$= -800 - 81$$

$$= -881 \text{ psf (outward acting on a strip 10 ft wide)}$$

B.2 Design Pressures by Proposed Uniform Method

Steps in the proposed uniform method are described in Section 4.3 and Appendix A.

B.2.1 Case I: Proposed Uniform Approach

STEP 1: Basic Wind Speed and Importance Factor

V = 180 mph (fastest-mile wind speed)

I = 1.0

STEP 2: Velocity Pressure

K_h = 0.87 (h=20 ft, ANSI A58.1-1982, Table 6, Exposure C)

$q_h = 0.00256\ K_h\ (IV)^2$

$= 0.00256 \times 0.87\ (1.0 \times 180)^2$

$= 72$ psf

STEP 3: Overall Design Pressures

$p = q_h\ G_h\ C_p - q_h(GC_{pi})^*$ (ANSI 1982, Table 4)

* For one story buildings, ANSI 1982 requires consideration of internal pressure.

G_h : Gust Response Factor
Evaluate at roof height, ANSI A58.1-1982, Table 8, $h = 20$ ft, $G_h = 1.29$

C_p : External Pressure Coefficients from ANSI A58.1-1982, Section 6.7

Windward Wall, $C_p = +0.8$

Leeward Wall, $C_p = -0.5$

Side Walls, $C_p = -0.7$

Roof, $C_p = -0.7$

GC_{pi} : Internal Pressue Coefficients
Uniform Openings, ANSI A58.1-1982, Table 9, $(GC_{pi}) = \pm 0.25$

Windward Wall

$p = (72)(1.29)(+0.8)-(72)(\pm 0.25)$

$= 74 \pm 18$

$= +92$ psf (inward acting)

$= +56$ psf (inward acting)

Leeward Wall

$p = (72)(1.29)(-0.5)-(72)(\pm 0.25)$

$= -46 \pm 18$

$= -28$ psf (outward acting)

$= -64$ psf (outward acting)

Side Walls & roof

$$p = (72)(1.29)(-0.7)-(72)(\pm 0.25)$$

$$= -65 \pm 18$$

$$= -47 \text{ psf (outward acting)}$$

$$= -83 \text{ psf (outward acting)}$$

STEP 4: Local Design Pressures

$$p = q_h(GC_p - GC_{pi})$$

GC_p : External Pressure Coefficients from ANSI A58.1-1982, Figure 3

GC_{pi} : Internal Pressure Coefficients
Uniform Openings, ANSI A58.1-1982, Table 9,
$GC_{pi} = \pm 0.25$

The design pressures are tabulated in Table B.1

B.2.2 Case II: Proposed Uniform Approach

STEP 1: Basic Wind Speed and Importance Factor

$V = 169$ mph (fastest-mile wind speed)

$V_{em} = 162$ fps (same as current method)

$I = 1.35$

STEP 2: Velocity Pressure

$K_h = 1.19$ (h=60 ft, ANSI 1982-Table 6)

$$q_h = 0.00256\ K_h\ (IV)^2$$

$$= 0.00256(1.19)(1.35 * 169)^2$$

$$= 158 \text{ psf}$$

TABLE B.1

DESIGN PRESSURES FOR COMPONENTS AND CLADDING
CASE I: PROPOSED UNIFORM APPROACH

Component	Tributary Area, sq ft	Location*	(GC_p)	Design Pressure psf
Roof Purlin	5x25=125	1	-1.2	-104
		2 & 3	-1.5	-126
Roof Corners	8x8=64	1	-1.24	-107
		2	-1.71	-141
		3	-1.98	-160
Wall Panel	20x25=500	4 & 5	-1.10	-97
			+1.00	89
Wall Corner	8X8=64	4	-1.31	-112
			+1.21	105
		5	-1.57	-131
			+1.21	105

* From ANSI 1982, Figure 3

"1" Represents roof

"2" Represents ridge and eaves

"3" Represents roof corners

"4" Represents wall

"5" Represents wall corners

K_z = 0.87 (z=20 ft)

= 0.98 (z=30 ft)

= 1.06 (z=40 ft)

= 1.13 (z=50 ft)

= 1.19 (z=60 ft)

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

$= 0.00256\ K_z\ (1.35 \times 169)^2$

= 116 psf (z=20 ft)

= 130 psf (z=30 ft)

= 141 psf (z=40 ft)

= 150 psf (z=50 ft)

= 158 psf (z=60 ft)

P_{APC} = -162 psf (same as current method)

STEP 3: Overall Design Pressures

$p = q_z G_h C_p$ (for windward wall at height z)

$p = q_h G_h C_p$ (for other walls and roof at mean roof height)

$p = q_h G_h C_p + 0.5\ P_{APC}$ (see Section B.1.2)

G_h : 1.20 (h=60 ft)

Windward Wall (see Table B.2)

Leeward Wall

$p = (158)(1.20)(-0.5) - (0.5)(162)$

$= -95 - 81$

= -176 psf (outward acting)

Side Walls & Roof

$$p = (158)(1.20)(-0.7) - (0.5)(162)$$

$$= -133 - 81$$

$$= -214 \text{ psf (outward acting)}$$

STEP 4: Local Design Pressures

$$p = q_h(GC_p) + 0.5 \times P_{APC} \text{ (from Section 4.3)}$$

The design pressures are tabulated in Table B.3.

TABLE B.2

DESIGN PRESSURES FOR WINDWARD WALL
CASE II: PROPOSED UNIFORM APPROACH

Height (ft)	K_z	q_z (psf)	C_p	G_h	Design pressure (psf)
20	0.87	116	+0.80	1.20	+107
30	0.98	130	+0.80	1.20	+125
40	1.06	141	+0.80	1.20	+135
50	1.13	150	+0.80	1.20	+144
60	1.19	158	+0.80	1.20	+152

TABLE B.3

DESIGN PRESSURES FOR COMPONENTS AND CLADDING
CASE II: PROPOSED UNIFORM APPROACH

Component	Tributary Area, sq ft	Location* Zone	(GC_p)	Design Pressure psf
Roof Panel	20x20=400	1	-1.2	-294
		2 & 3	-1.5	-339
Roof Corner	8x8=64	1	-1.24	-298
		2	-1.71	-372
		3	-1.98	-415
Wall Panel	20x20=400	4	-1.18	-288
			+1.08	69
		5	-1.27	-303
			+1.08	69
Wall Corner	64	4	-1.31	-309
			+1.21	89
		5	-1.57	-350
			+1.21	89

* From ANSI 1982, Figure 3

"1" Represents roof

"2" Represents ridge and eaves

"3" Represents roof corners

"4" Represents wall

"5" Represents wall corners

APPENDIX C

RELATIONSHIP BETWEEN FASTEST-MILE WIND SPEED AND TORNADO WIND SPEED (3 SEC. GUST)

The purpose of this section is to develope a relationship between fastest-mile wind speed and tornado wind speed (3 second gust). The differences between the two wind speed measurements are due to averaging times. Information gathered by Durst (1960) was used to determine a relationship between maximum winds peed and averaging time, t (sec). The relationship developed by Durst (1977) is shown in Figure C.1.

Tornado wind speeds are first converted to mean hourly wind speeds using Figure C.1. Then, a trial-and-error procedure is required to convert mean hourly wind speeds to fastest-mile wind speeds because of the variable averaging time associated with fastest-mile wind speeds. The conversion factor from tornado wind speeds to mean hourly wind speeds is 1.51 (See Figure C.1). The tornado wind speed is divided by 1.51 to obtain the corresponding mean hourly wind speed. Next, one estimates the corresponding fastest-mile wind speed. The averaging time for the estimated fastest-mile wind speed is obtained from the following relationships:

$$\frac{V\ (\text{mile/hour})}{3600\ (\text{sec/hour})}\quad t\ (\text{sec}) = 1.0\ \text{mile}$$

$$t = \frac{3600}{V}\ (\text{sec}) \qquad (C.1)$$

The conversion factor can then be obtained for converting the mean hourly wind speed to fastest-mile wind speed from Figure C.1. If multiplication of the mean hourly wind speed times the conversion factor gives the same values as estimated, then the iterative procedure is complete. Otherwise, a new estimated fastest-mile wind speed is considered and the procedure is repeated. Table C.1 summarizes the iterative procedure for tornado wind speeds from 90 to 300 mph.

A plot of fastest-mile wind speed versus tornado wind speed is given in Figure C.2 based upon the data of tornado and fastest mile wind speeds assembled from Table B.1. A linear regression is made as following:

$$y = 0.9575x - 11.3430 \qquad (C.2)$$

where

y = fastest-mile wind speed

x = tornado wind speed

Using a more obvious notation

$$V_{(fmw)} = 0.9575\ V_{(tor)} - 11.3430 \qquad (4.1)$$

TABLE C.1

TRIAL-AND-ERROR PROCEDURE FOR RELATING
TORNADO WIND SPEEDS TO FASTEST-MILE WIND SPEEDS

Tornado Wind Speed	Mean Hourly W. S.	Esti-mated FMW	Avg. Time, Sec	Conver-sion Factor	Calcu-lated FMW	FMW
90	59.60	72	50.00	1.2600	75.10	
		75.10	47.94	1.2639	75.33	
		75.33	47.79	1.2641	75.35	
		75.35	47.78	1.2642	75.35	75.4
100	66.23	80	45.00	1.275	84.44	
		84	42.86	1.28	84.77	
		85	42.35	1.285	85.11	85.1
110	72.85	88	40.91	1.2776	93.07	
		93.07	38.68	1.2842	93.55	
		93.55	38.48	1.2849	93.61	93.6
120	79.47	96	37.50	1.2887	102.41	
		102.41	35.15	1.2980	103.15	
		103.15	34.90	1.2991	103.24	103.2
130	86.09	111	32.43	1.3097	112.75	
		112.75	31.93	1.3119	112.94	
		112.94	31.87	1.3122	112.97	112.9
140	92.72	121	29.75	1.3214	122.52	
		122.52	29.38	1.3226	122.63	
		122.63	29.36	1.3227	122.64	122.6
150	99.34	122	29.57	1.3220	131.33	
		131.33	27.41	1.3293	132.05	
		132.05	27.26	1.3299	132.11	132.1
160	105.96	140	25.65	1.3358	141.54	
		141.54	25.44	1.3366	141.62	
		141.62	25.42	1.3366	141.63	141.6
170	112.58	150	23.93	1.3425	151.14	
		151.14	23.82	1.3429	151.19	
		151.19	23.81	1.3429	151.19	151.2
180	119.21	161	22.36	1.3490	160.81	
		160.81	22.39	1.3489	160.79	
		160.79	22.39	1.3489	160.79	160.8

TABLE C.1 (CONT.)

Tornado Wind Speed	Mean Hourly W. S.	Esti-mated FMW	Avg. Time, Sec	Conver-sion Factor	Calcu-lated FMW	FMW
190	125.83	169	21.30	1.3537	170.33	
		170.33	21.14	1.3544	170.43	
		170.43	21.12	1.3545	170.43	170.4
200	132.45	178	20.16	1.3590	180.00	
		180.00	20.00	1.3598	181.10	
		180.10	19.99	1.3600	180.13	180.1
210	139.07	189	19.05	1.3640	189.70	
		189.70	18.98	1.3644	189.74	
		189.74	18.97	1.3644	189.74	189.7
220	145.70	198	18.18	1.3681	199.32	
		199.32	18.06	1.3687	199.41	
		199.41	18.05	1.3687	199.41	199.4
230	152.32	207	17.39	1.3719	208.97	
		208.97	17.23	1.3728	209.10	
		209.10	17.22	1.3728	209.10	209.1
240	158.94	216	16.67	1.3756	218.64	
		218.64	16.46	1.3767	218.81	
		218.81	16.45	1.3768	218.82	218.8
250	165.56	228	15.79	1.3973	230.74	
		230.74	15.60	1.3947	230.90	
		230.90	15.59	1.3949	230.94	230.9
260	172.19	237	15.19	1.3970	240.55	
		240.55	14.96	1.3980	240.72	
		240.72	14.96	1.3980	240.72	240.7
270	178.81	247	14.57	1.3990	250.16	
		250.16	14.39	1.3995	250.24	
		250.24	14.39	1.3995	250.24	250.2
280	185.43	256	14.06	1.3999	259.58	
		259.58	13.87	1.4005	259.70	
		259.70	13.86	1.4006	259.71	259.7

TABLE C.1 (CONT.)

Tornado Wind Speed	Mean Hourly W. S.	Esti-mated FMW	Avg. Time, Sec	Conver-sion Factor	Calcu-lated FMW	FMW
290	192.05	266	13.53	1.4090	270.60	
		270.60	13.30	1.4094	270.70	
		270.70	13.30	1.4094	270.70	270.7
300	196.68	274	13.14	1.4100	280.14	
		280.14	12.85	1.4105	280.23	
		280.23	12.85	1.4105	280.23	280.2

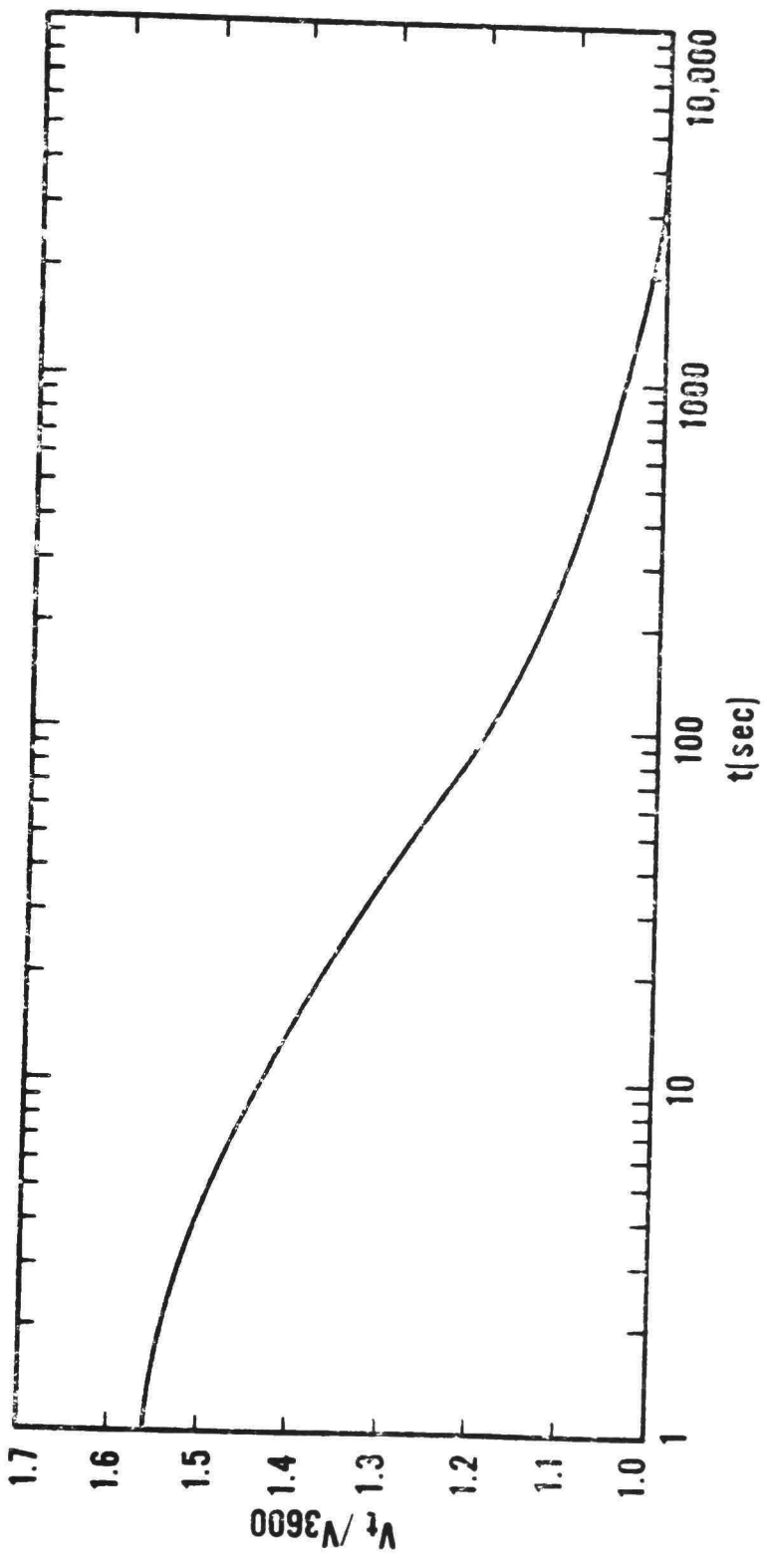

FIGURE C.1 VARIATION OF MAXIMUM WIND SPEED WITH AVERAGING TIME (DURST, 1977)

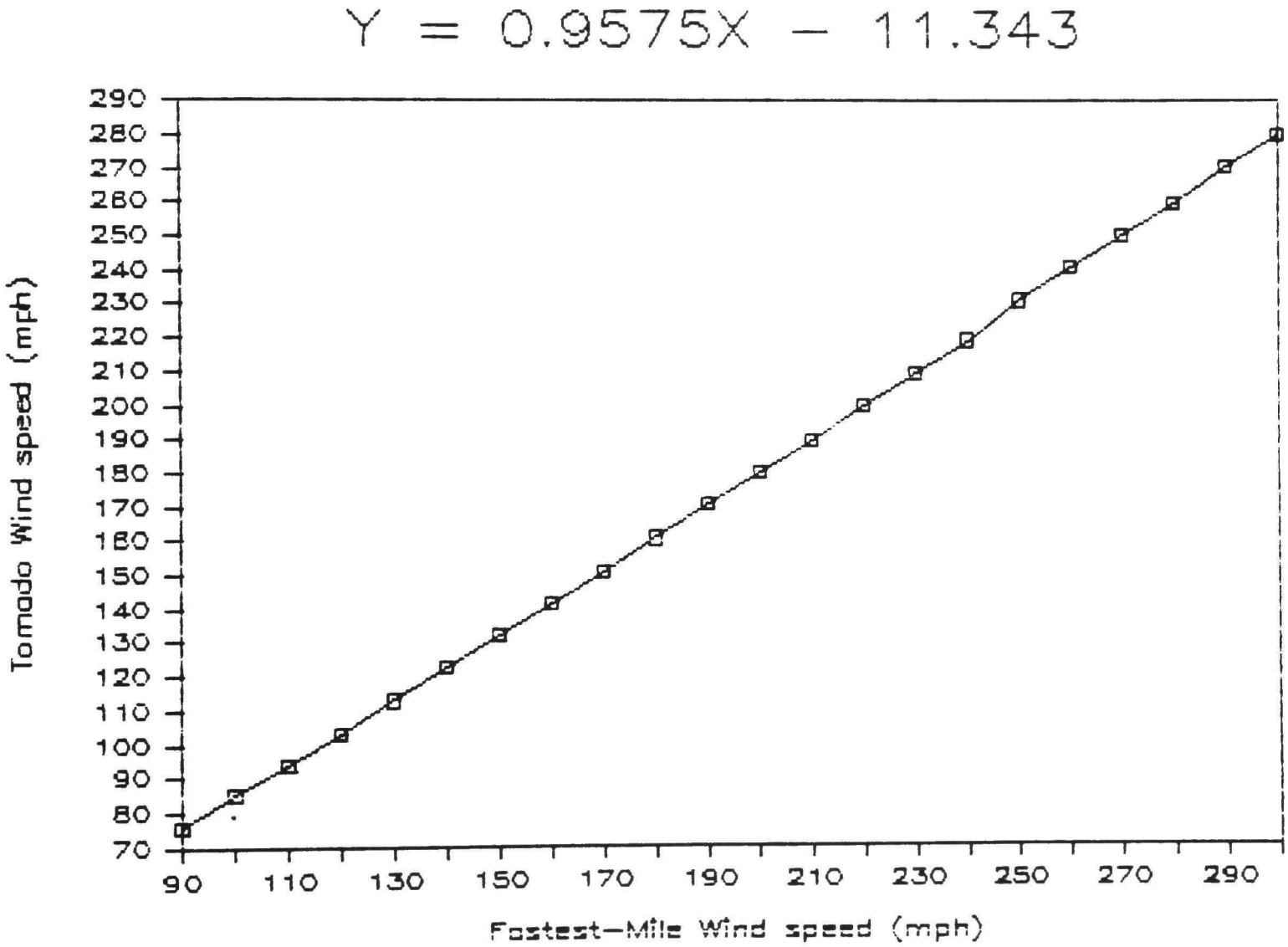

FIGURE C.2 RELATIONSHIP BETWEEN TORNADO WIND SPEED AND FASTEST-MILE WIND SPEED

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