To the Graduate Council:

I am submitting herewith a thesis written by Christopher A. Belk entitled "Wind Speed Modelling for Structural Load Combination Methods." I have examined the final 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.

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WIND SPEED MODELLING

FOR

STRUCTURAL LOAD COMBINATION METHODS

A Thesis

Presented for the

Master of Science

Degree

The University of Tennessee, Knoxville

Christopher A. Belk

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ABSTRACT

Many of today's reliability-based specifications utilize somewhat outdated methods to handle the effects of load combinations. While more accurate load combination methods exist, they are seldom used due to a lack of necessary statistical data on various types of loadings.

Pertinent statistics of wind were obtained from continuous gust charts. The wind at nine sites across the United States was modelled as a Poisson square wave pulse process with clustering effects being taken into consideration. Wind intensity was fitted with an Extreme Type I distribution. The data were verified by comparing extreme annual and 50-year winds obtained from the developed model with those statistics previously obtained by other researchers.

Estimates of composite statistics for the contiguous United States were made. These statistics include the mean rate of occurrence, mean duration, and mean number of winds per cluster. Parameters for an Extreme Type I distribution of wind speed intensity and wind pressure were developed in terms of the 50-year design wind speed and the design wind pressure respectively.

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I. INTRODUCTION

Significant advances have recently been made in the study of structural reliability. The American Institute of Steel Construction Load and Resistance Factor Design manual is an excellent example of a recent specification which utilizes current knowledge in reliability-based design. However, since the publication of even the most recent specifications, researchers have developed improved methods for the solving of many problems associated with structural reliability.

Evaluation of the maximum combined load effects over a given period of time is one of the most important considerations in the design of a structure and the development of probability-based specifications. Current building specifications utilize somewhat outdated methods to handle this load combination problem. More accurate methods have been developed but are not widely used due to a lack of important statistical data on various types of loadings.

There is a particular dearth of data with respect to wind loads. Accurate statistics and parameters of the wind process for use in load combination analyses are needed in order to develop more accurate design specifications. This need is accentuated by the development of new construction techniques and building materials which has brought about the design of structures that are much more flexible and lightweight than was previously possible. These structures are increasingly susceptible to the effects of wind loads.

The Wind Process

The wind is the motion of air with respect to the earth's surface. It is primarily caused by thermal currents in the atmosphere as it is heated by the sun. Other factors which influence the wind include the earth's rotation, the compression and expansion of air particles, and the horizontal drag force caused by friction between the atmosphere and earth's surface. A detailed discussion of these and other factors, as well as descriptions of extreme wind phenomenon such as hurricanes, monsoons, extratropical cyclones, and tornadoes can be found in Sachs (1978) and Simiu and Scanlan (1986).

Load Modelling and Load Combination Methods

Significant progress has recently been made in the development of stochastic models for various load processes. Several methods have been developed for obtaining statistics of the maximum combined load using probability theory (Pearce and Wen, 1984; Larrabee and Cornell, 1981; Grigoriu, 1984). Loads and load effects have been modelled as both static and dynamic problems taking into account both nonlinear limit states and dynamic responses (Pearce and Wen, 1984). Winterstein and Cornell (1984) estimated both first and second order reliabilities of structures subjected to several individual and combined loadings. Pearce and Wen (1984) concluded that the method of load coincidence (Wen, 1981) gives good results for load combination problems of many types. The widely used load combination rule developed by Turkstra (1970) was found to be consistently unconservative due to it's greater dependence on load process characteristics (Pearce and Wen, 1984). Turkstra's rule considers the maximum combined load to occur when one load is at its lifetime maximum value and all other loads are at their arbitrary-point-in-time value. It thus ignores the possibility of the maximum combined load occurring at times other than when one load is at its maximum.

The majority of load combination methods generally consider the loads on a structure to follow a poisson square wave pulse process. This process is described in detail in Section II. Pearce and Wen (1984) determined that the pulse process is appropriate for both static and dynamic loadings. Other models, such as a clipped normal process (Ditlevsen and Madsen, 1983), have been studied. However, due to its relative simplicity, the poisson pulse process is used for the modelling of loads in most load combination analyses.

Several building codes recommend a reduction factor to recognize the small chance of the simultaneous occurrence of maximum loads. An example of one such reduction is the factor of 0.75 required by the American Concrete Institute (1983) for the combination of live loads and wind or earthquake loads. Such factors are based largely on judgement and experience. Several investigations of the load combination problem based on random process theory (Wen, 1977; Bosshard, 1975; Grigoriu, 1975) have shown that such reduction factors tend to be quite conservative. Further research on the development of probability-based load factors and load combinations suitable for use with the loads specified in ANSI.A58 was performed by Galambos et al. (1982). This study also summarized available data on structural capacity and loads and assessed current standards from a reliability viewpoint.

Another concern of the structural engineer is the tendency of loads to cluster around common points in time. The occurrence dependence then becomes an important factor in load combination analysis. Wen (1981) presents an analytical model based on a multivariate point process that takes this occurrence dependence into consideration. Dependence within loads and between loads are both modelled for use in the load coincidence method.

Previous Load Analysis

Probabilistic models for the determination of building design floor live loads have been developed in detail for spatial and temporal load variability (Chalk and Corotis, 1980; Peir and Cornell, 1973; Hasofer, 1968). Live load surveys in several office buildings (Culver, 1976) have provided valuable data which have been applied to probabilistic live load models (Ellingwood and Culver, 1977). Ellingwood and Culver (1977) compared the results of a United States survey with the results of a study in the United Kingdom and the design loads specified in ANSI.A58-1972. Another comprehensive study to compare the effects of actual gravity loads with building design loads was reported by McGuire and Cornell (1974).

Since most live load surveys are conducted over a relatively short period of time, the statistics of extreme live loads must be analytically developed. Wen (1981) has presented methods for relating the mean value of the lifetime maximum sustained live load to the statistics of the short term loadings. Corotis and Tsay (1983) have developed statistics for both sustained and extraordinary live loads.

Previous to this research there had been no modelling of wind data from actual records. A number of studies have assumed certain values of wind statistics. Der Kiureghian (1979) assumed a mean rate of occurrence of 50 wind storms per year (0.00571 per hour), a mean duration of 1 x 10^{-4} years (0.875 hours), and a lognormal distribution for the intensity. Turkstra and Madsen (1980) assumed that there is always a non-zero wind with a mean duration of six hours that follows a gamma distribution. Kim and Wen (1987) assumed that the wind follows an Extreme Type I distribution with a mean rate of occurrence of 10 storms per year (0.00114 per hour) and a mean duration of 0.01 years (87.6 hours). These assumed values have been used in various load combination problems.

Objective and Scope

The objective of this research is to obtain necessary statistics of wind speed so that it may be modelled as a poisson pulse process. Appropriate parameters will thus be available for use in load combination analysis. The data used throughout this report were obtained from the analysis of continuous strip gust charts made available by the National Climatic Data Center.

No effort was made to differentiate between the different types of wind. Although data have been truncated at 20 knots to eliminate morning breezes and other winds which are not structurally significant, hurricanes, extratropical cyclones, thunderstorms, tornadoes, and other extraordinary meteorological events have not been separated. It is felt though, that the results are not applicable to hurricanes or tornadoes, partially due to the geographic regions examined.

The duration of individual and clustered wind storms and the time lag between wind storms have been fitted to an exponential distribution. Several distributions, including the gamma and Extreme Type I, were examined with regard to suitability for modelling intensity. The data have been verified by comparing one-year maximum wind statistics and 50-year maximum winds with values given in Simiu et al. (1979).

Composite United States statistics have been developed from data obtained at various sites. Appropriate parameters for the Extreme Type I distribution of individual winds are developed in terms of the 50-year design wind speed. Parameters have also been developed for the individual wind pressure intensities in terms of the nominal design pressure. These values can then be used in an appropriate load combination method.

II. DATA COLLECTION AND DISTRIBUTION ANALYSIS

Data Collection From Continuous Gust Charts

Wind speed data are recorded by the National Weather Service in various forms. The data used in the majority of this report were recorded on continuous strip charts. An example of a typical segment of one such chart is shown in Figure 1. The goal of this research is to model actual wind speed records as a Poisson square wave pulse process. The Poisson pulse process has been widely used for time varying loadings and is characterized by a mean rate of occurrence λ, a mean load duration, and a random intensity (Wen and Pearce, 1981). An example of the Poisson square wave pulse process is shown in Figure 2. The time lag is defined as the period between the start of one loading and the start of the following loading, and is the reciprocal of the mean rate of occurrence. The duration and intensity of each load are represented respectively by the width and height of each Poisson square pulse.

The Poisson square wave model has been extended to include loadings that have a tendency to cluster around common points in time (Wen, 1981; Winterstein and Cornell, 1984). This clustering may be an important factor in load combination analysis. In Figure 2, a Poisson square wave pulse model for a three-wind cluster storm surrounded by two single-wind storms is illustrated. The time lag and duration are defined in the same manner as they are for a single-wind storm. Within a cluster, the number of winds is n, and each wind has a random individual duration. All appropriate wind storms have been modelled

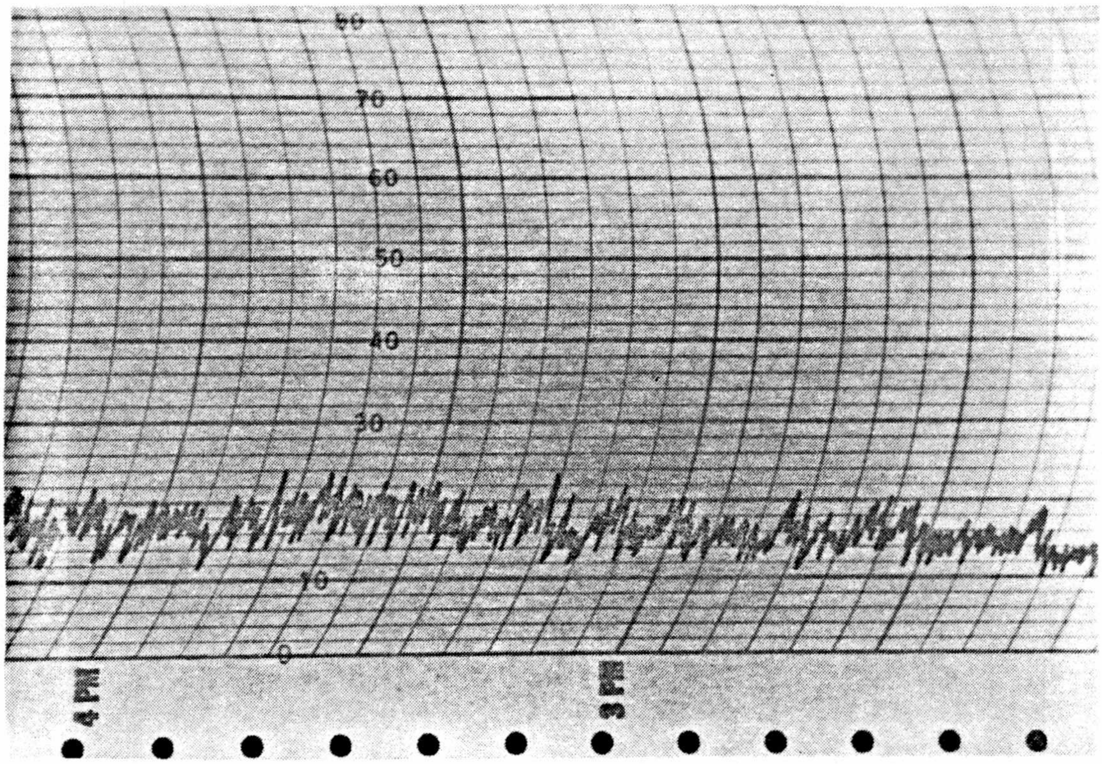


FIGURE 1. Typical Segment of Continuous Gust Chart

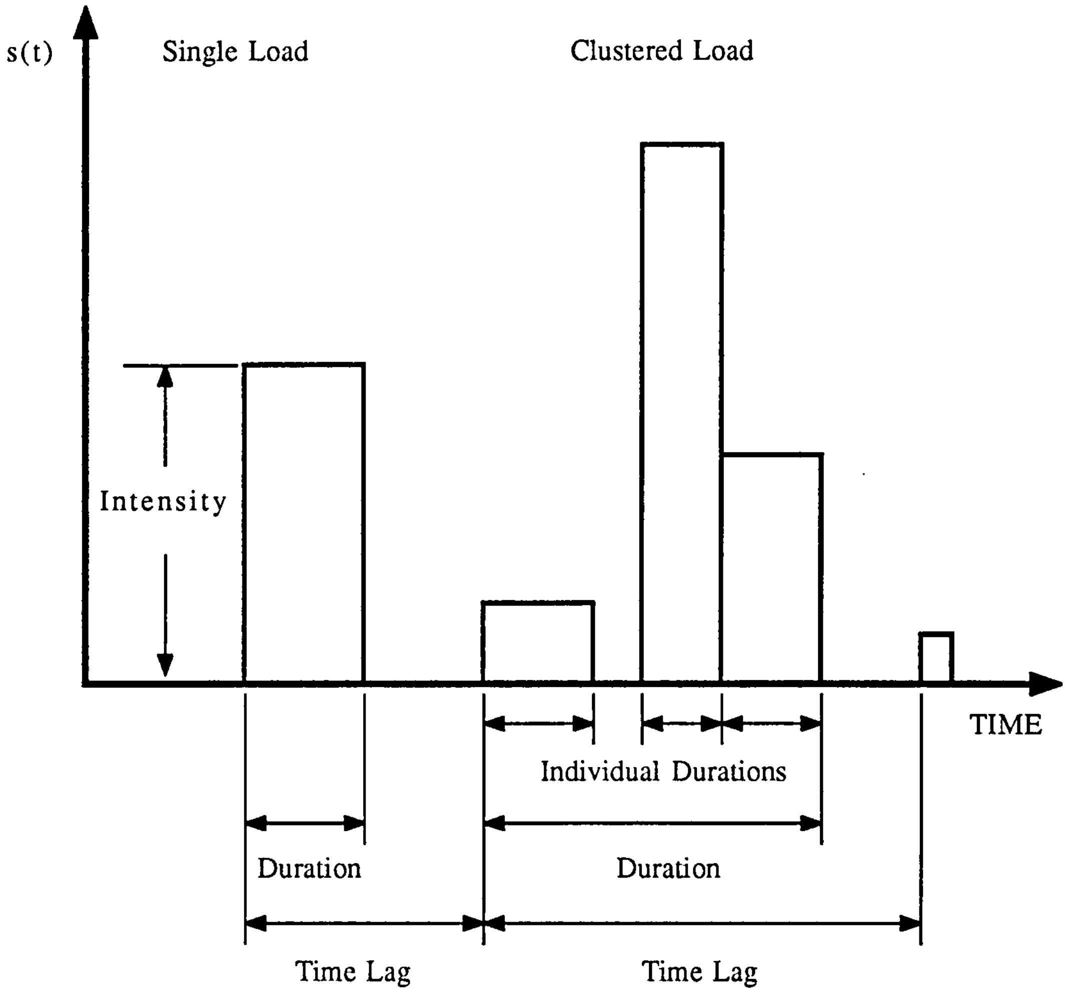


FIGURE 2. Poisson Square Wave Pulse Process

as clusters of occurrences of structurally significant winds. Single winds have been considered as 'one-wind' clusters.

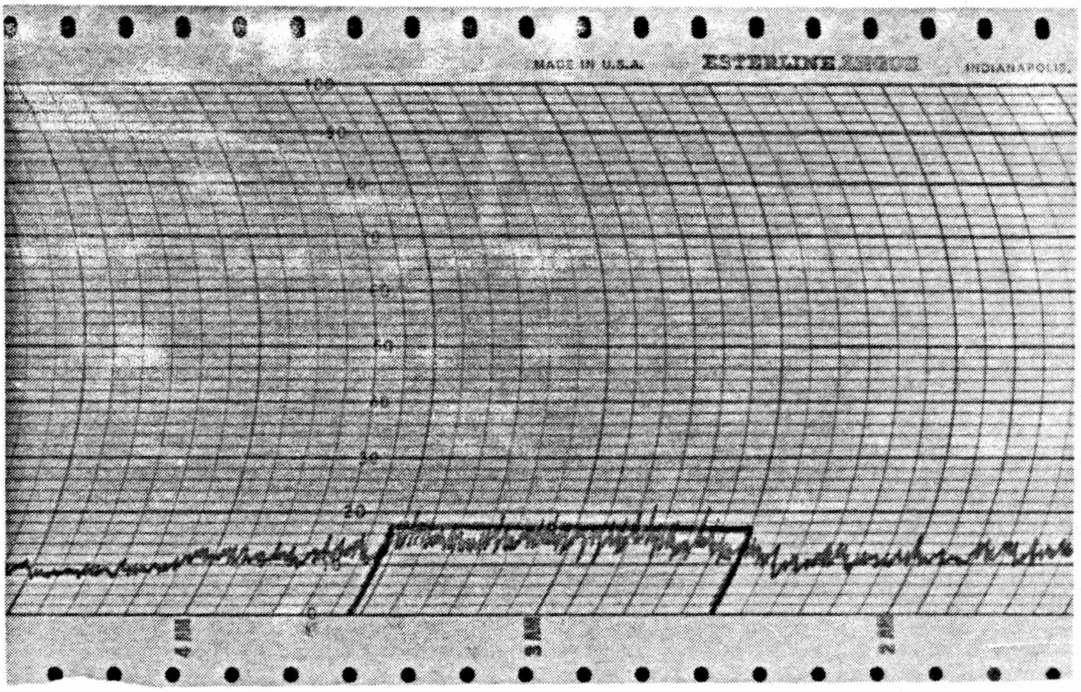
Nonzero winds are almost a constant occurrence. However, a great deal of the wind speeds are very low and are of small structural significance. The data used in this report reflect only the modelling of structurally significant winds. A formal definition of what constitutes a structurally significant wind is not in existence.

The American National Standards Institute (ANSI, 1982) requires a minimum design pressure equivalent to that caused by a 70 mile per hour wind. Since the force on a structure is proportional to the square of the wind velocity, a wind speed of approximately 22 miles per hour causes a force that is 10 percent of the minimum required design force. This force is considered to be a definition of a structurally significant load for load combination analyses. Wind speeds are generally recorded in knots, with 22 miles per hour equaling 19.2 knots. For convenience 20 knots was chosen as the cutoff value. Therefore, any winds with peak velocities at or above 20 knots have been modelled as a square wave pulse while those below 20 knots have been modelled as zero.

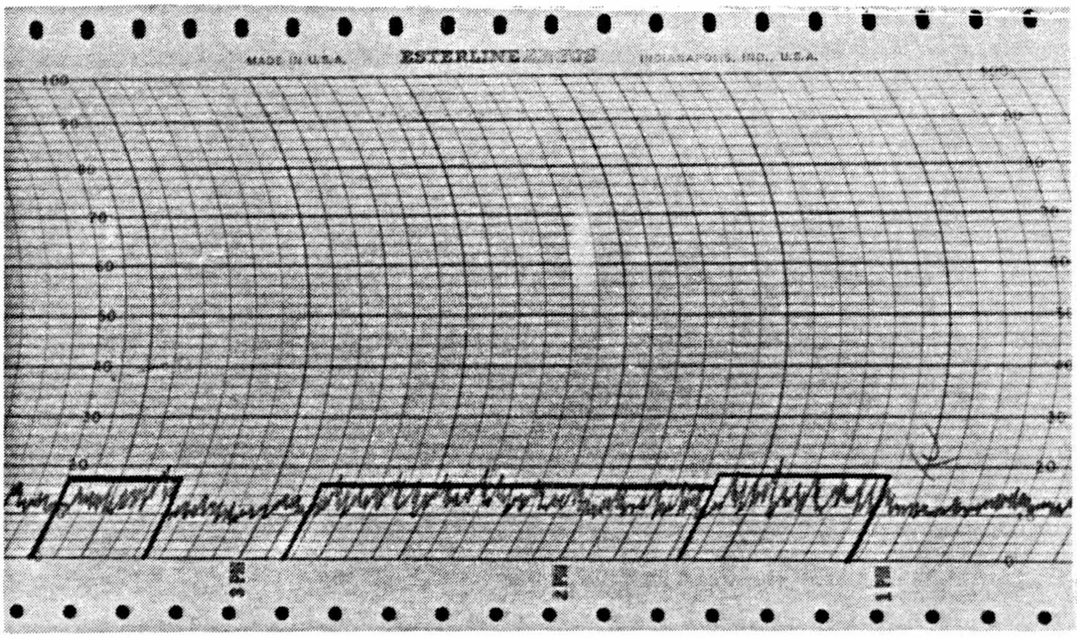
Further justification for this truncating of data is the elimination of many meteorological events that are unrelated to extreme winds. It has been shown (Sachs, 1978) that extreme wind data cannot be predicted from the annual frequency distribution of mean hourly winds. This is due to the number of events reflected in hourly data which are meteorologically unrelated to the storms associated with extreme winds. Simiu and Scanlan (1986) observe that these unrelated events obscure information relevant to the description of the extreme wind climate. The truncation of data collection at 20 knots has eliminated many meteorological events, such as morning breezes, which are not significant to the analysis and prediction of extreme winds, and do not impose significant forces on structures.

A potential difficulty in the modelling of structurally significant winds involves the determination of the duration of a storm. Generally, there are no well defined beginning and ending points of each occurrence. Figures 3A and 3B show the modelling process for two separate segments of a continuous gust chart. In certain instances, as in 3A, the significant wind is reasonably well defined and the duration can be accurately approximated. However, in most cases, especially when clustering occurs (Fig 3B), it is much more difficult to accurately ascertain the durations of the significant wind occurrences. This leads to a degree of subjectivity which depends on the individual researcher's interpretation of the wind speed records. The actual amount of error that this subjectivity has induced into the results of this research is unclear. However, the consistency of the results presented in this report indicate that they are not greatly effected by the subjectivity of the collection procedure.

Wind speed can be divided into two components: the mean wind, which remains constant over the duration, and a fluctuating component about the mean. Simiu and Scanlan (1986) have presented a detailed study of the fluctuating wind using random process theory. The mean wind intensity cannot be readily obtained from the continuous strip charts. However, based on Equation 2.3.37 in Simiu and Scanlan (1986) the mean hourly wind speed can be related to the peak wind speed as follows:



A



B

FIGURE 3. Poisson Modelling Process for Segments of Continuous Gust Chart

$$V_{hour} = \frac{1}{[1 + 0.185c(t)]} V_{peak} \qquad 2.1$$

in which V_{hour} = mean hourly wind speed and V_{peak} = peak wind speed. Some assumption used to obtain Equation 2.1 are a constant $\beta = 6.0$, an anemometer elevation z = 10m, and a roughness length z_o = 0.05m. The coefficient c(t) is based on statistics of wind speed reported by Durst (1960) and is related to the average duration length. Table 1 lists the values of c(t) for various durations. From information supplied by the National Weather Service, it is apparent that the recording equipment is unable to record gusts of a duration less than 10 seconds. From Table 1, the appropriate value of c(t) is 2.32. Therefore, Equation 2.1 becomes:

$$V_{hour} = 0.700 \ V_{peak} \qquad 2.2$$

To obtain the average wind speed for a specified duration, Equation 2.2 must be altered as follows:

$$V_{avg} = [0.7 \ \{1 + 0.185 \ c(\tau)\}] \ V_{peak} \qquad 2.3$$

in which V_{avg} = average speed and τ = duration. If τ is one hour or greater, then the value $c(\tau)$ from Table 1 is zero and V_{avg} is simply 0.7 (V_{peak}). If τ is less than one hour, then the appropriate value of $c(\tau)$ must be used.

For an example of the analysis procedure, refer back to Figures 3A and 3B. Both strip segments shown are from actual records for May, 1988. They represent wind speeds as recorded by the National Weather Service at McGhee Tyson Airport in Knoxville, Tennessee.

TABLE 1

VALUES OF COEFFICIENT c (t)

Duration (seconds)	c (t)
1	3.00
10	2.32
20	2.00
30	1.73
50	1.35
100	1.02
200	0.70
300	0.54
600	0.36
1000	0.16
3600	0.00

Figure 3A indicates a single wind occurrence beginning at 2:30 A.M. and ending at 3:32 A.M. The duration is therefore one hour and two minutes. The peak wind gust is 21 knots or 24.2 miles per hour. Using Equation 2.2, the average wind speed is determined to be 16.9 miles per hour.

Figure 3B represents a storm of an unspecified duration containing several single wind occurrences. This is an excellent example of the clustering effect. The mean wind speed of the entire storm duration is not considered. Calculations of the average speeds of each individual occurrence are identical to those described earlier.

The wind speed records used for this research were obtained through the National Climatic Data Center (NCDC) in Asheville, North Carolina. Two years of records from eight sites throughout the continental United States were analyzed. Seven of these sites, Austin, Baltimore, Detroit, Rochester, Sacramento, St. Louis, and Tucson, were chosen because they were used in previous studies on the development of a probability based load criterion (Ellingwood et al., 1980). The eighth site, Billings, Montana, was chosen to provide geographical representation for the northwest United States as shown in Figure 4.

To examine stationarity from year to year, a ninth site was more intensely studied. Due to local interest, seven continuous years of data from Knoxville, Tennessee are analyzed. Pertinent statistics from all nine sites are presented in Tables 2 and 3. Year by year statistics from Knoxville, as well as the combined seven year statistics are listed in Tables 4 and 5. It appears that more than one

TABLE 2

DATA FOR STRUCTURALLY SIGNIFICANT
WIND STORMS - VARIOUS SITES

City	Sample Size	Time Lag (Hours)		Duration (Hours)		# Per Cluster	
		Mean	St. Dev.	Mean	St. Dev.	Mean	St. Dev.
Austin	373	47.12	55.94	5.52	6.65	1.25	0.61
Baltimore	326	51.29	58.05	4.77	5.35	1.33	0.74
Billings	446	38.96	43.02	5.29	5.80	1.26	0.67
Detroit	324	54.14	61.55	6.42	7.36	1.60	1.31
Knoxville	500	114.85	165.80	3.42	4.06	1.33	0.80
Rochester	271	61.19	81.71	5.73	6.36	1.27	0.71
Sacramento	128	120.87	163.09	8.42	6.89	1.39	0.79
St. Louis	222	78.88	101.37	5.80	6.34	1.39	0.90
Tucson	491	35.68	44.14	3.45	3.68	1.41	0.84

TABLE 3

DATA FOR INDIVIDUAL WINDS WITHIN CLUSTERS - VARIOUS SITES

City	Sample Size	Individual Duration (Hours)		Maximum Speed (Miles Per Hour)		Average Speed (Miles Per Hour)	
		Mean	St. Dev.	Mean	St. Dev.	Mean	St. Dev.
Austin	466	4.19	4.63	28.63	5.66	20.21	3.99
Baltimore	435	3.33	3.67	29.20	5.43	20.66	3.78
Billings	563	4.03	4.41	32.70	6.76	22.98	4.73
Detroit	519	3.78	4.19	29.87	6.10	21.01	4.27
Knoxville	663	2.23	2.52	29.98	5.97	21.25	4.32
Rochester	343	4.28	4.80	29.91	6.70	21.02	4.66
Sacramento	178	5.62	4.35	27.16	4.89	19.08	3.49
St. Louis	309	4.07	4.52	31.09	6.43	21.90	4.58
Tucson	690	2.30	2.41	28.53	5.59	20.15	3.94

TABLE 4

DATA FOR STRUCTURALLY SIGNIFICANT
WIND STORMS - KNOXVILLE, TN

Year	Sample Size	Time Lag (Hours)		Duration (Hours)		# Per Cluster	
		Mean	St. Dev.	Mean	St. Dev.	Mean	St. Dev.
1981	92	86.55	172.95	3.24	3.58	1.37	0.78
1982	67	132.11	141.44	3.72	4.15	1.39	0.93
1983	74	108.33	146.86	4.08	5.05	1.46	1.27
1984	74	113.99	153.04	2.60	2.62	1.27	0.65
1985	68	128.48	158.97	4.02	3.95	1.28	0.65
1986	47	154.34	266.79	3.20	4.89	1.30	0.62
1987	78	105.03	128.38	3.15	4.13	1.18	0.50
Combined	500	114.85	165.80	3.42	4.06	1.33	0.80

TABLE 5

DATA FOR INDIVIDUAL WINDS WITHIN CLUSTERS - KNOXVILLE, TN

Year	Sample Size	Individual Duration (Hours)		Maximum Speed (Miles Per Hour)		Average Speed (Miles Per Hour)	
		Mean	St. Dev.	Mean	St. Dev.	Mean	St. Dev.
1981	126	2.11	2.31	29.28	5.21	20.87	3.75
1982	93	2.47	2.38	31.24	7.01	22.11	4.89
1983	108	2.50	2.47	31.32	6.50	22.05	4.57
1984	94	1.94	1.69	30.62	6.15	21.47	4.24
1985	89	2.81	2.73	31.13	5.96	21.96	4.35
1986	61	2.03	3.11	26.72	3.87	19.01	2.75
1987	92	2.52	2.97	28.51	5.15	20.54	4.51
Combined	663	2.23	2.52	29.98	5.97	21.25	4.32

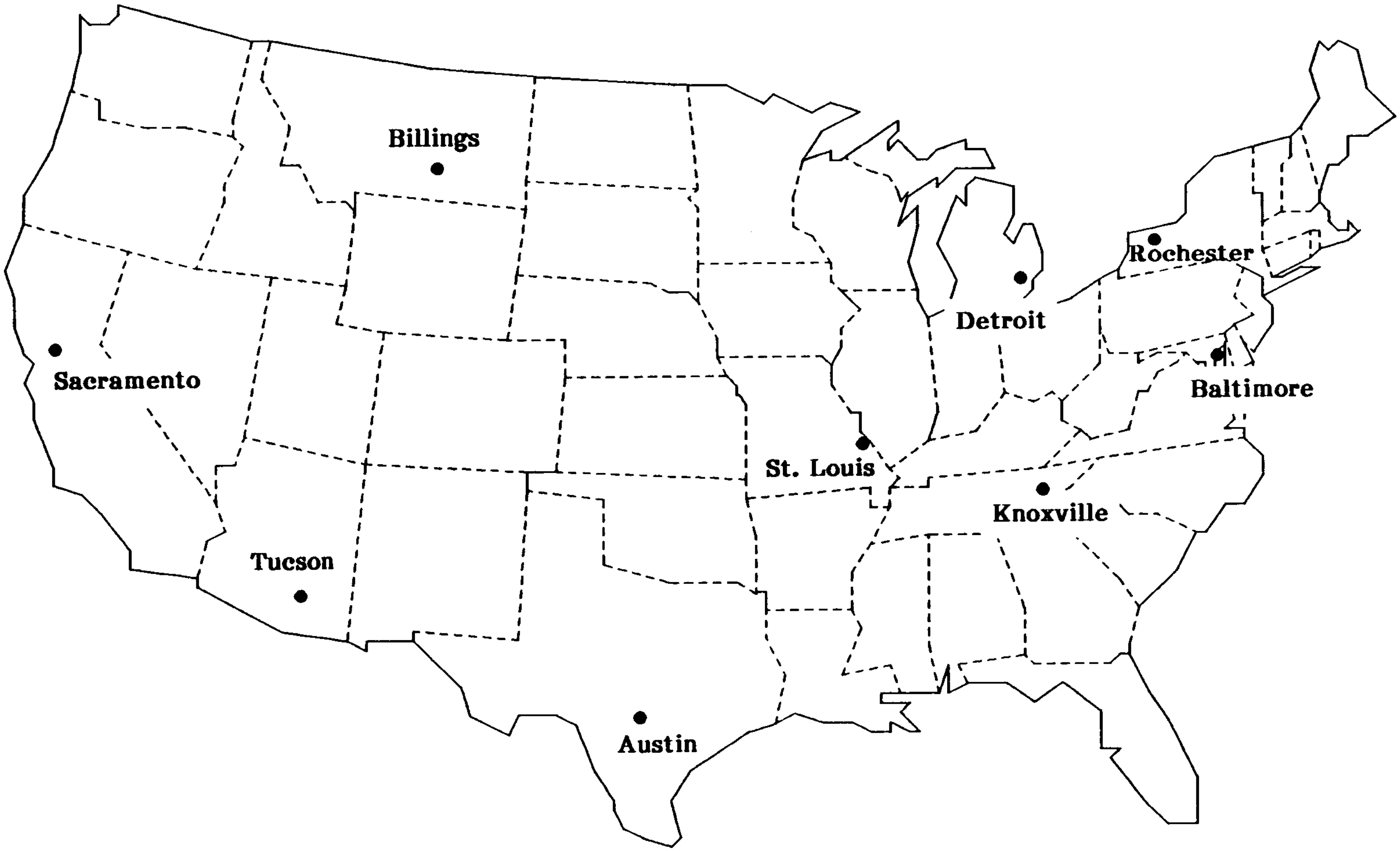


FIGURE 4. Sites Analyzed

year of records is necessary to define the wind at a site, but two years are probably sufficient.

It should be noted that thoroughly complete records were not obtained. Presented in this report is the most complete set of data available. However, certain blocks of records have been misplaced or destroyed. A complete list of missing data is shown in Table 6.

To examine the effects of correlation between intensity and duration, coefficients of correlation for average speed versus individual duration and maximum speed versus individual duration were calculated for each of the nine sites analyzed. These values are listed in Table 7. Previous work on the effects of correlation (Wen and Pearce, 1981) show that for coefficients less than 0.5, the error factor remains well below 1.5 for the coincident term in load combination analysis. Neglecting the effects of correlation significantly simplifies the Poisson square pulse model. It therefore seems reasonable to ignore the slight correlation between intensity and duration, and consider them to be statistically independent.

If a process follows a Poisson process, then the time lag and duration should follow an exponential distribution (Ang and Tang, 1975). A fundamental property of an exponential distribution is that the mean and standard deviation should be equal. Referring to Tables 2, 3, 4, and 5 it can be seen that this is approximately true for time lag, cluster duration, and individual duration. The greater discrepancies for the time lag probably are caused by the modelling of winds with intensity of less than 20 knots as zero. While this truncating of data is a necessary and theoretically correct step, it conceivably could cause

TABLE 6

MISSING DATA

City	Period Missing
Baltimore	September 1986
Knoxville	December 1981 July 1983 March 1-26, 1986 September 1-5, 1986
Rochester	May 1986
Sacramento	December 1985

TABLE 7

CORRELATION COEFFICIENTS FOR
INDIVIDUAL DURATION VS. WIND SPEED

City	Correlation Coefficient	
	Average Speed vs. Individual Duration	Maximum Speed vs. Individual Duration
Austin	0.2134	0.2489
Baltimore	0.3202	0.3661
Billings	0.1363	0.1479
Detroit	0.2838	0.3027
Knoxville	0.0929	0.1408
Rochester	0.4358	0.4630
Sacramento	0.4742	0.4825
St. Louis	0.1359	0.1639
Tucson	0.1506	0.1894

problems with the probability distribution due to a large majority of the overall data being ignored. Frequency histograms and the resulting exponential distributions are shown in Figures 5-7, for time lag, cluster duration, and individual duration statistics from Knoxville, Tennessee. Results from a Chi-Square test are included in Table 8. These results indicate the fundamental correctness of the Poisson process assumption.

Various distributions were examined to determine a proper probability distribution for the maximum and average wind speed data. Chalk and Corotis (1980) showed that the Extreme Type I or Gumbel distribution gave the most accurate results for maximum sustained live loads. Simiu (1979) and Kauffman (1977) have concluded that the Extreme Type I distribution also provides an excellent fit for extreme wind data. Chou and Corotis (1983) showed that the Rayleigh distribution provides a good approximation for nominal one-minute average wind speeds. Both the Extreme Type I and Rayleigh distributions were fitted to the data obtained through this research. A gamma distribution and a lognormal distribution have also been included. Frequency histograms fitted with these distributions are shown in Figures 8 and 9 and Chi-Square values are listed in Table 9 for data from Knoxville, Tennessee.

The Extreme Type I distribution provides the best fit for both maximum and average wind speed data. The lognormal and gamma distributions provided poorer fits. The Rayleigh distribution provided the worst fit of any of the distributions examined. Truncating all wind speeds below 20 knots and assuming them to be zero is probably the main cause of the inadequacy of the Rayleigh distribution.

TABLE 8

CHI-SQUARE VALUES FOR
OCCURRENCE VARIABLES FOR KNOXVILLE, TN

Variable	Chi-Square Value for Exponential Distribution
Time Lag	103.75
Cluster Duration	30.73
Individual Duration	48.61

TABLE 9

CHI-SQUARE VALUES FOR
WIND SPEED DISTRIBUTION FOR KNOXVILLE, TN

Variable	Chi-Square Value			
	Gamma	Lognormal	Rayleigh	Extreme Type I
Maximum Speed	216.80	234.22	1319.65	163.22
Average Speed	161.79	128.04	1014.19	72.54

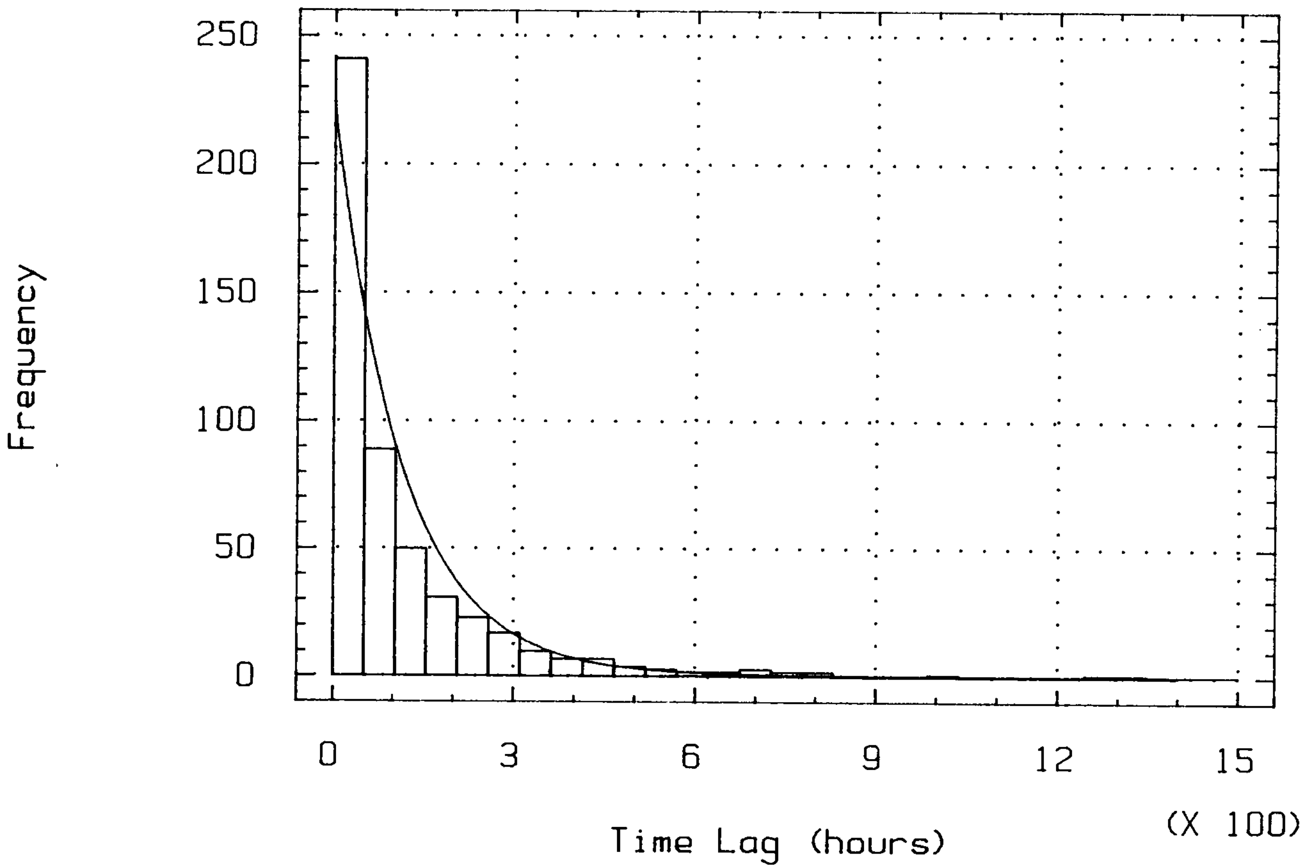


FIGURE 5. Exponential Distribution of Time Lag for Knoxville, TN

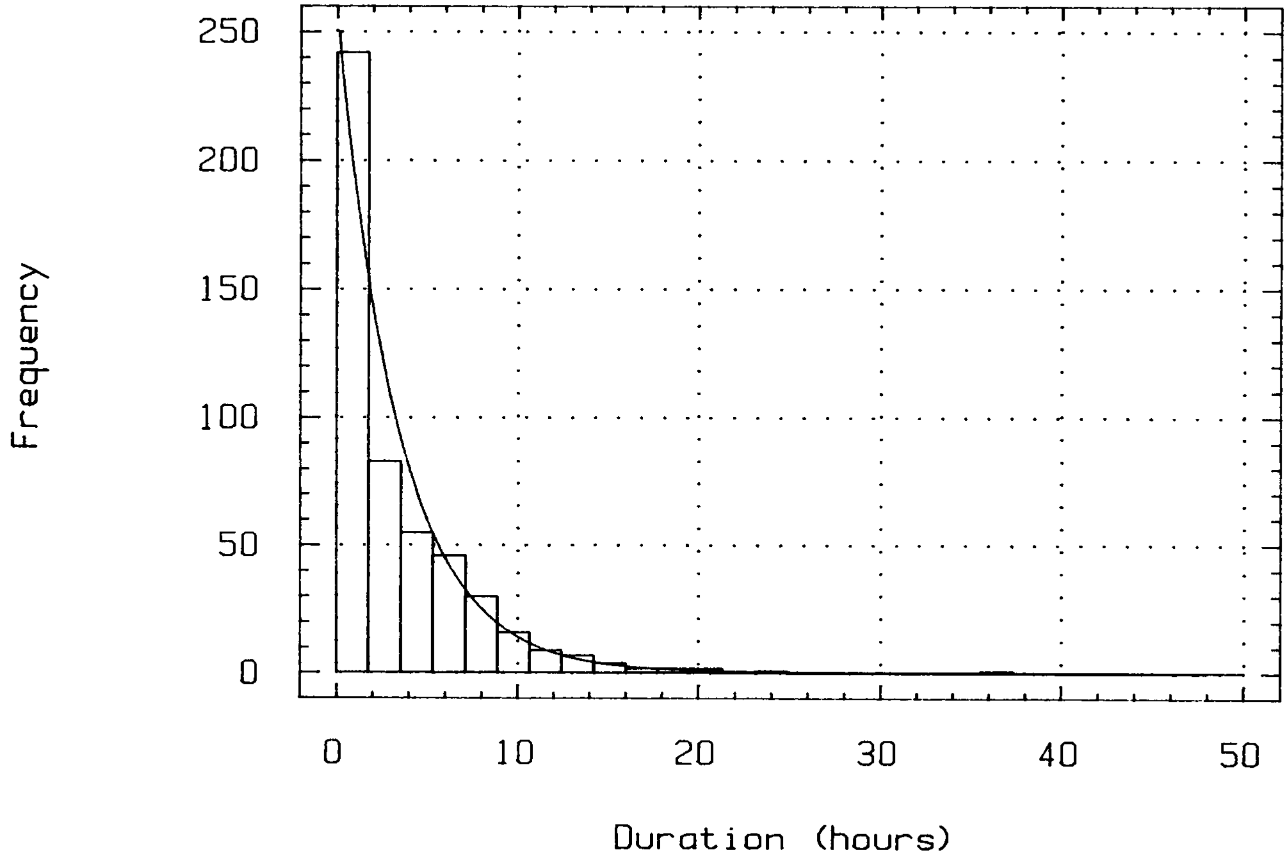


FIGURE 6. Exponential Distribution of Cluster Duration for Knoxville, TN

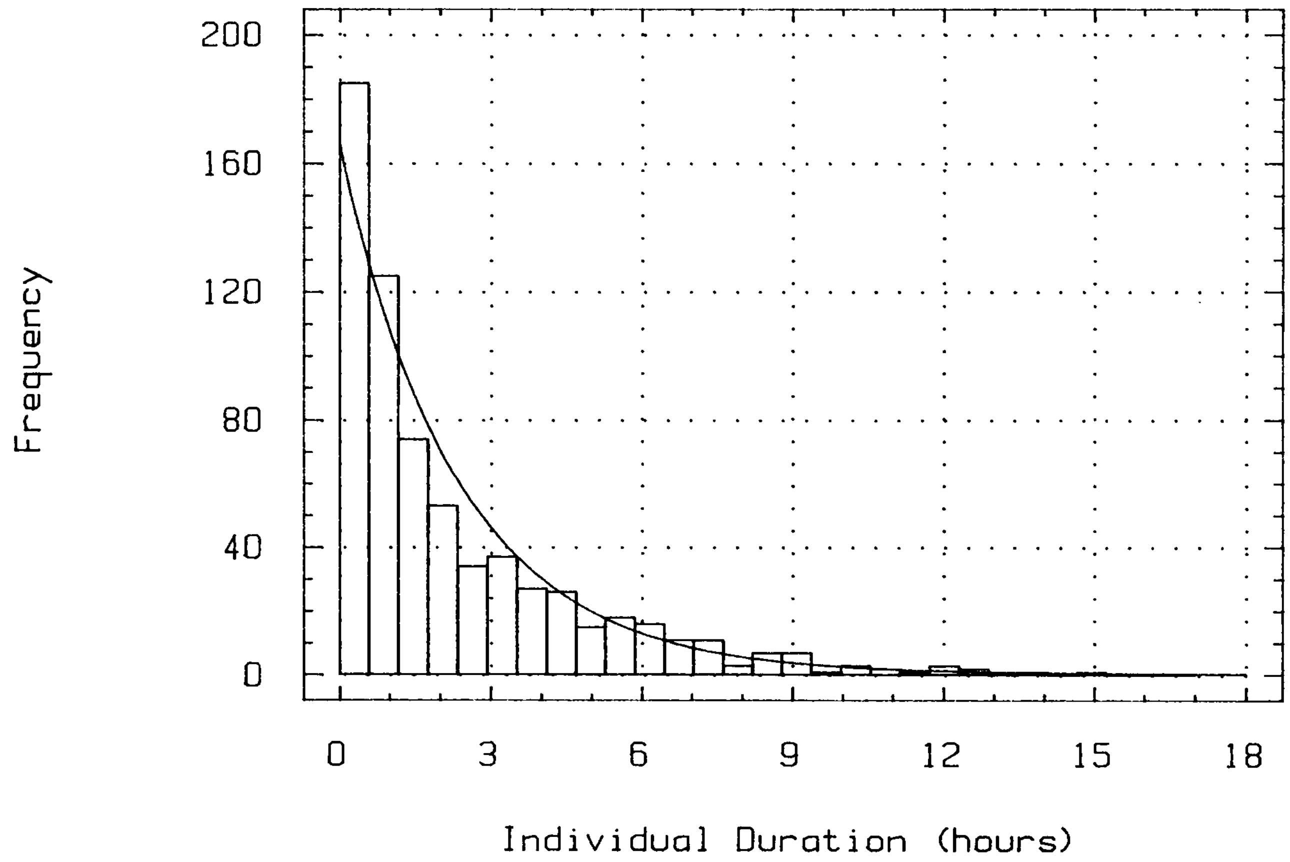


FIGURE 7. Exponential Distribution of Individual Duration for Knoxville, TN

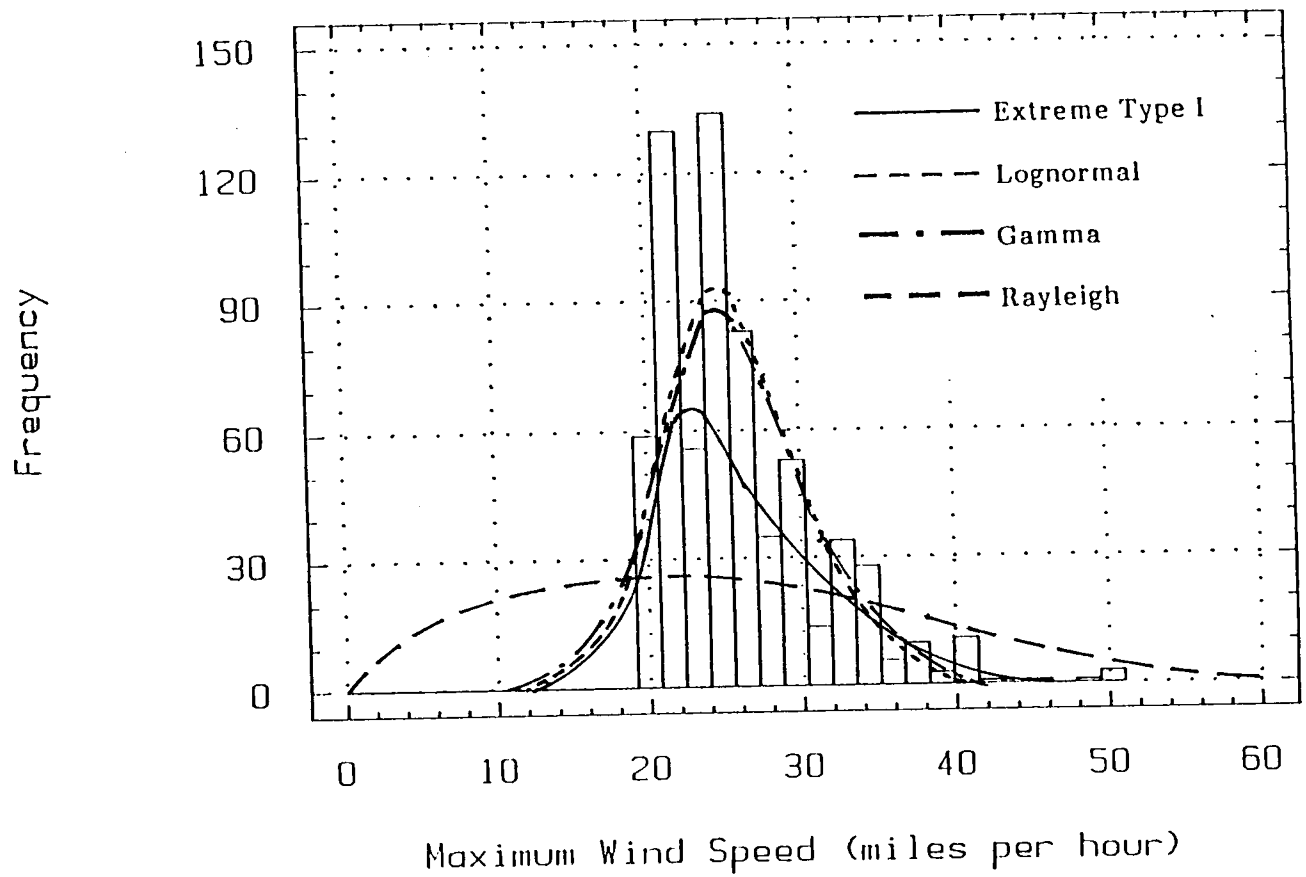


FIGURE 8. Various Distributions of Maximum Wind Speed for Knoxville, TN

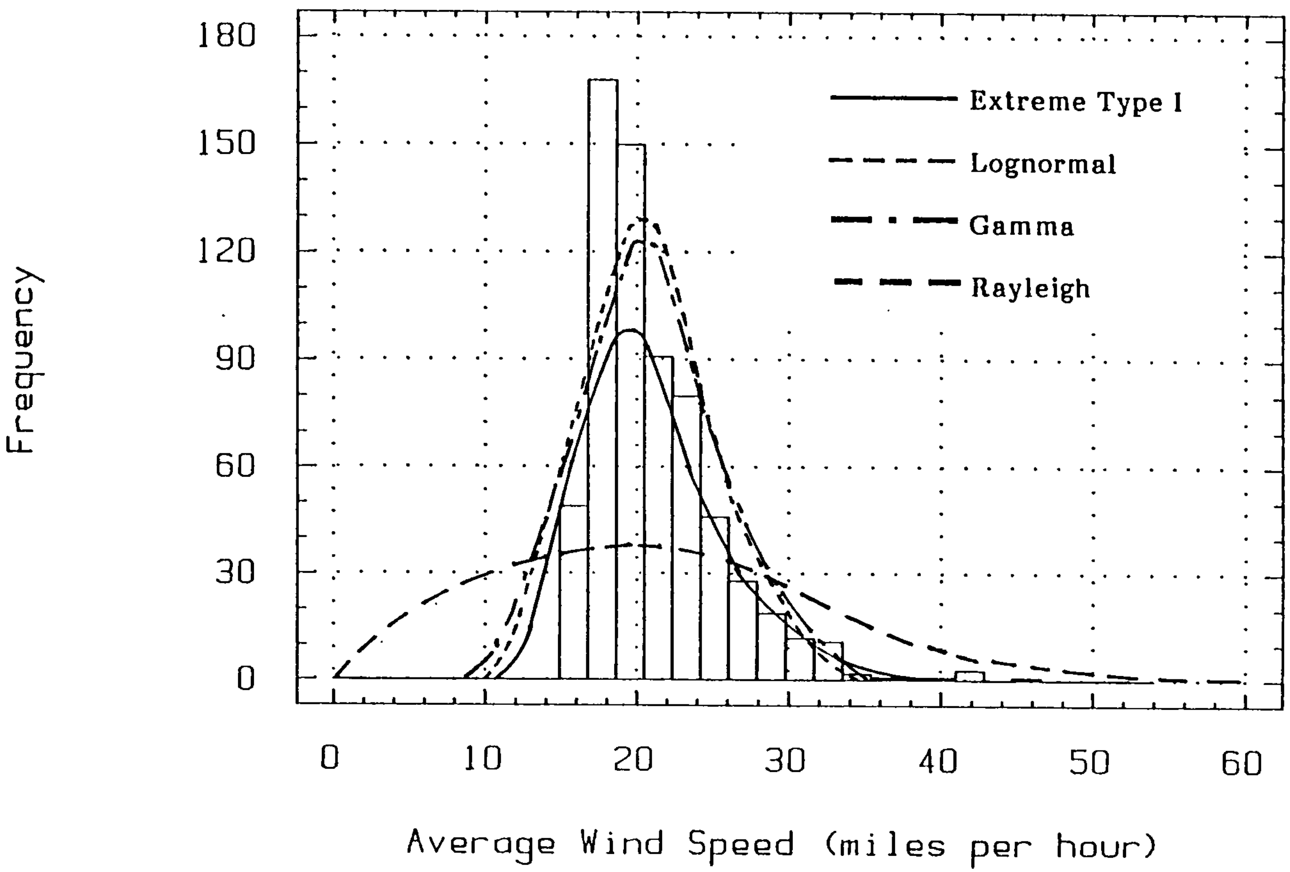


FIGURE 9. Various Distributions of Average Wind Speed for Knoxville, TN

Data Collection From Local Climatological Data Summaries

Another form in which the National Weather Service publishes wind speed data is the monthly Local Climatological Data Summaries (LCDS). These reports contain a listing of the average one minute wind speed recorded at three hour intervals. Local Climatological Data Summaries are more easily and quickly analyzed than the continuous strip charts. They are also more readily obtainable and much less costly to analyze. To analyze the continuous strip charts at any location other than NCDC in Asheville, North Carolina is highly cost preventative. The LCDS's, however, are available at only a nominal cost.

One year of LCDS's from Knoxville was analyzed (January-December, 1986). The process was indeed much quicker and easier than the analysis of the same year of continuous strip charts. However, only twelve data points were obtained. This small data set is not sufficient to accurately represent wind behavior. Because of the three hour recording durations, storm durations of less than three hours are impossible to obtain. It is also impossible to determine any clustering effects. Therefore the Local Climatological Data Summaries do not provide the necessary information for thorough wind speed analysis.

III. VERIFICATION OF RESULTS

For a clustered process, the cumulative distribution function of the maximum wind in time T is (Wen and Pearce, 1981):

$$F_{X_m}(x_m) = exp\left\{-\lambda_c T\left[\frac{n\left(1-F_X(x_m)\right)}{n\left(1-F_X(x_m)\right)+1}\right]\right\} \qquad 3.1$$

in which λ_c is the mean rate of occurrence of clustered winds, n is the average number of winds occurring in each cluster of significant winds, $F_X(x_m)$ is the cumulative distribution function of the maximum individual wind velocity, and $F_{X_m}(x_m)$ is the cumulative distribution function of the maximum wind in time T. This equation is developed based on the assumption that load duration and intensity are independent of each other and also independent of the occurrence time. Wen and Pearce (1981) indicate how dependencies between duration and intensity could be included in the formulation. The derivation of Equation 3.1 relies on the exponential distribution of the time of occurrence. It is therefore reasonable to assume that Equation 3.1 is applicable to the wind speed data collected in this research.

The maximum one year (8760 hour) wind speed for Knoxville, Tennessee is evaluated using Equation 3.1 and plotted on Extreme Type I paper in Figure 10. The straight-line plot indicates that the yearly maximum wind approaches an Extreme Type I distribution.

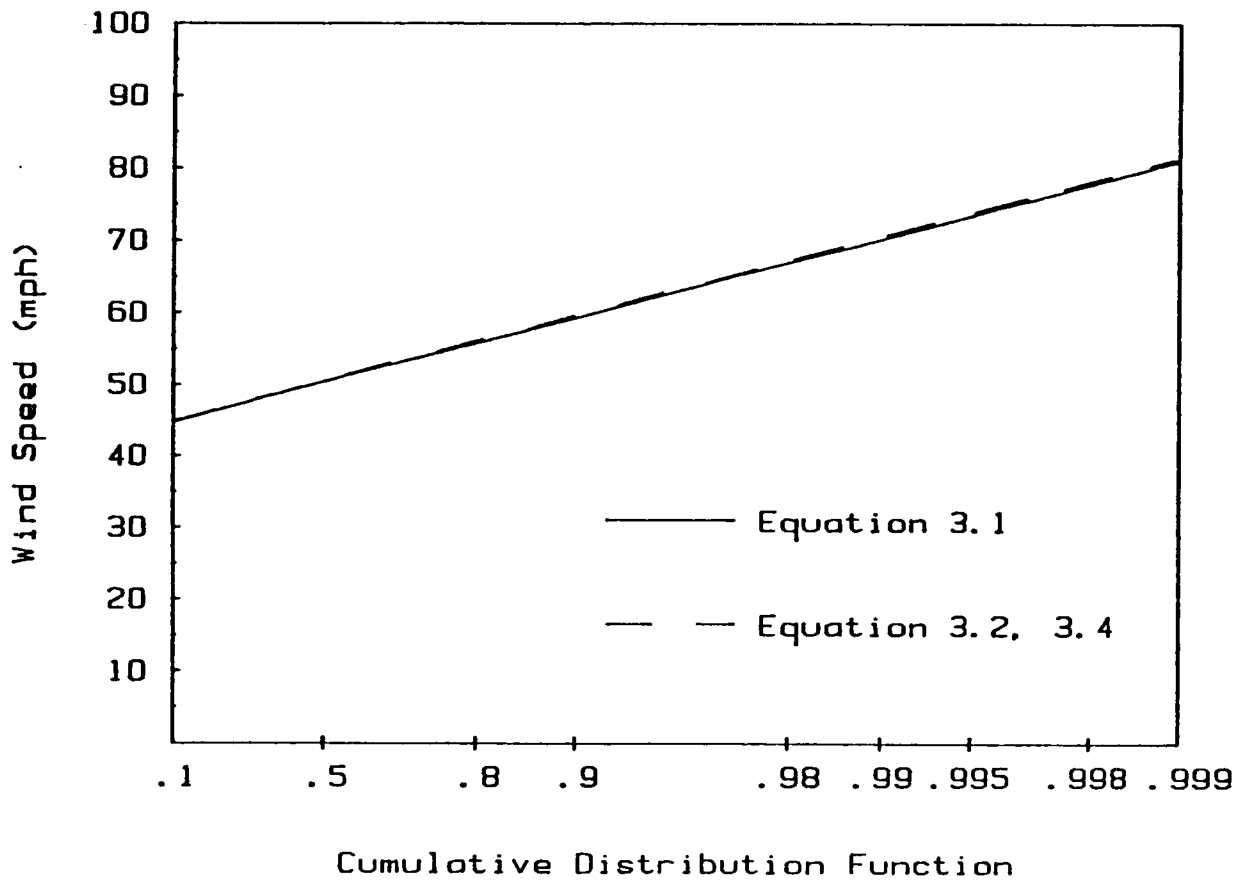


FIGURE 10. Comparison of Yearly Maximum Winds with Extreme Type I Approximation

An Extreme Type I distribution has the form:

$$F_X(x_m) = exp\left[-e^{-\alpha(x_m - u)}\right] \tag{3.2}$$

Assuming the individual wind occurrences to be statistically independent, the parameters for the yearly extreme winds, u_N and α_N, can be obtained approximately as (Ang and Tang, 1984):

$$u_N = \frac{-ln[-ln\left(1 - \frac{1}{N}\right)]}{\alpha} + u \tag{3.3a}$$

$$\alpha_N = N\,\alpha\, exp\left(-\alpha\,(u_N - u) - e^{-\alpha(u_N - u)}\right) \tag{3.3b}$$

in which N is the mean number of individual significant winds occurring in a time interval T, and u and α are the parameters of the individual wind distribution. The mean number of significant wind occurrences is determined by multiplying the mean rate of occurrence λ_c, the average number of winds per cluster n, and the time interval T. Substituting these variables into equation 3.3a and 3.3b, the resulting equations are:

$$u_N = \frac{-ln\left[-ln\,(1 - \frac{1}{n\lambda_c T})\right]}{\alpha} + u \tag{3.4a}$$

$$\alpha_N = \lambda_c T n\,\alpha\, exp\left(-\alpha\,(u_N - u) - e^{-\alpha(u_N - u)}\right) \tag{3.4b}$$

The parameters of the individual winds, u and α, are related to the mean and standard deviation as follows:

$$u = \mu - \frac{0.577215}{\alpha} \qquad 3.5a$$

$$\alpha = \frac{\pi}{\sigma\sqrt{6}} \qquad 3.5b$$

where μ is the mean and σ is the standard deviation. The approximate Extreme Type I distribution (Equation 3.2) obtained using the parameters of Equation 3.4 is also plotted in Figure 10. As can be seen, there is negligible difference between Equation 3.1 and Equations 3.2 and 3.4.

The parameters of the Extreme Type I distribution can be compared to the results from other maximum wind speed analyses. Simiu et al. (1979) presented information on extreme wind speeds at 129 stations in the contiguous United States. This information includes extreme yearly wind speeds and predicted wind speeds corresponding to various return periods. The predicted wind speeds are based on the assumption that the Extreme Type I probability distribution is a valid description of the yearly extreme wind speeds.

Extreme one-year wind speeds for the thirty-three years from 1942 through 1974 are utilized to obtain the results for Knoxville, Tennessee. Appropriate parameters for an Extreme Type I distribution for the Knoxville, Tennessee yearly maximum wind are u = 45.74 and α = 0.1864 which correspond to a mean of 48.84 miles per hour and a standard deviation of 6.88 miles per hour. The cumulative distribution is plotted in Figure 11 along with the plot of

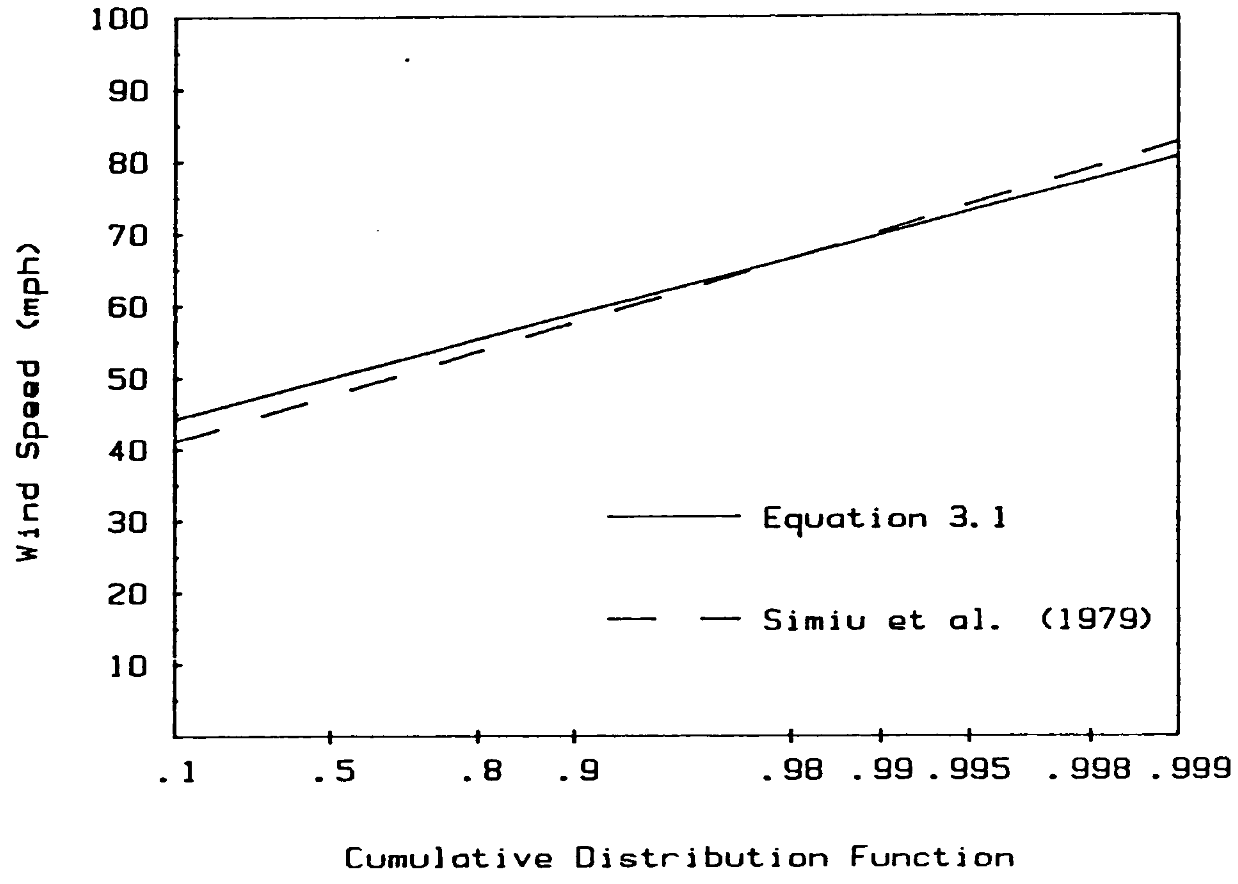


FIGURE 11. Comparison of Yearly Maximum Winds with Results from Simiu et al. (1979)

the analytical distribution of one-year wind obtained from Equation 3.1. Note the close agreement.

The extreme yearly wind speeds in Simiu et al. (1979) were obtained from the NCDC Local Climatological Data Monthly and Annual Summaries. The vast majority of this data consisted of fastest mile wind speeds. Fastest mile speeds are averaged over the time required for a volume of air with a horizontal length of one mile to pass over the anemometer (Simiu and Scanlan, 1986). The fastest mile wind speed, V_F has an averaging time $t = 3600/V_F$, where t is in seconds and V_F is in miles per hour.

The derivation of Equation 2.2 was partially based on an averaging time of ten seconds. As previously discussed, this value is related to the response time of the wind speed recording equipment. The averaging times for the fastest mile wind speeds used in Simiu et al. (1979) are significantly longer than ten seconds. This discrepancy in averaging times will cause a difference of up to 45 percent in the determination of mean hourly wind speeds. However, as the fastest mile wind speed increases and the averaging time decreases, the discrepancies in mean wind speed calculation are reduced significantly. Since the wind speeds being addressed are yearly extremes, the averaging time approaches the ten second time assumed for continuous strip chart data. Although the averaging time will doubtfully reach ten seconds, the difference in the mean hourly wind speed calculation is reduced to approximately 15-20 percent.

A comparison of one-year maximum wind statistics determined from Equations 3.2 and 3.4 is presented in Table 10 for the nine sites analyzed. The

TABLE 10

COMPARISON OF ONE-YEAR
MAXIMUM WIND STATISTICS

City	Simiu et al. (1979)			Equations 3.2 and 3.4		% Difference in Means
	Sample Size	Mean (mph)	St. Dev. (mph)	Mean (mph)	St. Dev. (mph)	
Austin	35	45.06	5.48	48.91	5.67	7.87
Baltimore	29	55.87	6.87	52.17	5.44	6.62
Billings	39	59.39	8.01	58.18	6.77	2.04
Detroit	44	48.88	6.83	56.29	6.11	13.16
Knoxville	33	48.84	6.88	53.45	6.02	8.62
Rochester	37	53.38	5.27	57.05	6.72	6.43
Sacramento	29	46.01	10.24	44.77	4.39	2.70
St. Louis	19	47.39	7.39	56.35	6.45	15.90
Tucson	30	51.38	8.61	54.00	5.60	4.85

percent difference in the mean values is well within 10 percent in every case except two. Therefore, fastest mile wind speed data and continuous strip chart data can be assumed to be comparable for extreme wind speed analysis.

The parameters of the Extreme Type I distribution for individual wind speeds can be back-calculated such that the yearly maximum wind speed distribution matches the results of Simiu et al. (1979). This is done by simultaneously solving Equations 3.4a and 3.4b for u and α. By assuming that the quantity $e^{-\alpha\,(u_N\,-\,u)}$ is small with respect to $\alpha\,(u_N - u)$, the following approximate equation can be developed:

$$\alpha \approx \frac{\dfrac{\alpha_N}{N}}{-\,ln\left(1 - \dfrac{1}{N}\right)} \qquad 3.6$$

The parameter u can then be determined from:

$$u = u_N + \frac{ln\left[-ln\,(1 - \dfrac{1}{N}\,)\right]}{\alpha} \qquad 3.7$$

Equation 3.6 is only a reasonable approximation for α. A more precise solution can be obtained by various root finding methods. Once the correct values of u and α are obtained, the mean and standard deviation for individual wind speeds can be determined from Equation 3.5. These values are listed in Table 11 along with the individual wind statistics obtained from the continuous strip charts for the nine sites analyzed. While these values do not correspond as closely as the extreme wind speed statistics presented in Table 10, they do indicate enough correlation to assume the validity of the data collected in this research. Since

TABLE 11

COMPARISON OF BACK-CALCULATED WIND STATISTICS (BASED ON NBS SAMPLE STATISTICS) AND ACTUAL WIND STATISTICS

City	Statistics From Continuous Strip Charts		Statistics Derived From Simiu et al. (1979)	
	Mean	St. Dev.	Mean	St. Dev.
Austin	28.63	5.66	21.84	5.47
Baltimore	29.20	5.43	26.86	6.86
Billings	32.70	6.76	24.21	7.99
Detroit	29.87	6.10	19.34	6.83
Knoxville	29.98	5.97	24.21	6.85
Rochester	29.91	6.70	32.10	5.25
Sacramento	27.16	4.89	9.41	10.18
St. Louis	31.09	6.43	18.46	7.36
Tucson	28.53	5.59	13.95	7.66

yearly extreme values are no longer being analyzed, it can be assumed that discrepancies between fastest mile data collection and continuous data collection become larger and contribute to the larger differences in the mean values presented in Table 11.

Simiu et al. (1979) have recorded estimates of maximum wind speeds likely to be attained over a return period of 50 years. These estimates are based on the extreme annual wind speeds which are assumed to follow an Extreme Type I distribution. The 50-year wind is defined as the velocity which has a probability of being exceeded in one year of 0.02 and can be determined by setting the extreme annual wind speed cumulative distribution function equal to 0.98. Using the statistics obtained from continuous strip chart analysis, estimates for 50-year maximum wind speeds have been calculated. These values are compared with the Simiu et al. (1979) 50-year estimates for seven individual years of data from Knoxville, Tennessee in Table 12. Although in an individual year, there can be a fairly large discrepancy in the predicted 50-year wind, the combined 7-year statistics predict the 50-year wind quite accurately.

Estimates obtained from combined Knoxville data as well as data from the eight other sites analyzed are presented in Table 13. The difference between estimates from Simiu et al. (1979) and estimates based on continuous strip chart analysis is roughly within 10 percent for all sites except one. This serves to further verify the Poisson modelling process and the selection of the Extreme Type I distribution for maximum wind speed data. It is expected that if additional data were analyzed for each site, the 50-year estimates would improve.

TABLE 12

COMPARISON OF 50-YEAR
WINDS - KNOXVILLE, TN

Year	Calculated 50-Year Wind (mph)	50-Year Wind (Simiu et al., 1979) (mph)	% Difference
1981	62.83	68	7.61
1982	74.16	68	9.10
1983	72.34	68	6.40
1984	68.52	68	0.76
1985	72.71	68	6.90
1986	49.69	68	26.90
1987	60.27	68	11.40
Combined	67.10	68	1.32

TABLE 13

COMPARISON OF 50-YEAR
WINDS - VARIOUS SITES

CITY	Calculated 50-Year Wind (mph)	50-Year Wind (Simiu et al., 1979) (mph)	% Difference
Austin	67.34	60	12.20
Baltimore	66.27	75	11.60
Billings	79.97	81	1.27
Detroit	72.13	67	7.65
Knoxville	67.10	68	1.32
Rochester	74.42	68	9.44
Sacramento	57.41	74	22.40
St. Louis	73.06	68	7.45
Tucson	68.49	75	8.68

IV. COMPOSITE UNITED STATES STATISTICS

One of the foremost goals of this research is to obtain average parameters which are applicable to the contiguous United States. These parameters may be used in load combination analyses to verify and modify current load combination design methods.

The nine sites analyzed in this report were selected in part to provide a uniform geographical representation of the continental United States (Figure 4). Therefore, data from these locations have been averaged to provide statistics for the contiguous United States. These values were determined by weighting the site statistics with the number of sample points obtained. Pertinent average statistics are listed below:

$$\text{mean rate of occurrence, } \lambda_c = 0.01589/\text{hour}$$

$$\text{mean cluster duration, } \mu_D = 4.99 \text{ hours}$$

$$\text{mean individual duration, } \mu_{D_i} = 3.46 \text{ hours}$$

$$\text{mean number of winds per cluster, } n = 1.362$$

These statistics were also calculated without weighting the individual site statistics. There were no significant differences in the values determined by either averaging method.

Based on these average temporal statistics, appropriate parameters for the individual wind intensity may be determined from yearly extreme wind speeds. Equation 3.6 becomes:

$$a = \frac{\dfrac{a_N}{(0.01589)\,(1.362)\,(8760)}}{-\ln\left[1 - \dfrac{1}{(0.01589)\,(1.362)\,(8760)}\right]}$$

$$a_N = 1.0026\ a \qquad 4.1$$

From Equation 3.5b:

$$a_N = \frac{\pi}{\sigma_N \sqrt{6}} \qquad 4.2$$

Combining Equations 4.1 and 4.2:

$$a = \frac{1.2792}{\sigma_N} \qquad 4.3$$

The standard deviation of the yearly maximum winds, σ_N, can be related to the mean, μ_N, by the coefficient of variation δ. Values of δ for yearly maximum winds are given for 129 sites across the United States in Simiu et al. (1979). The average of these values weighted by sample size is 0.1508. The standard deviation of δ is 0.0397 and the coefficient of variation of δ is 0.2632. Assuming the coefficient of variation to be a constant equal to the mean value causes Equation 4.3 to become:

$$a = \frac{8.484}{\mu_N} \qquad 4.4$$

Equation 3.7 may be used to determine the parameter u for the Extreme Type I distribution of individual winds. Using the appropriate United States statistics, Equation 3.7 may be written as follows:

$$u = u_N - \frac{5.2422}{\alpha} \qquad 4.5$$

From Equation 3.5a:

$$u_N = \mu_N - \frac{0.577215}{\alpha_N} \qquad 4.6$$

Combining Equations 4.1, 4.4, 4.5, and 4.6:

$$u = 0.3142\ \mu_N \qquad 4.7$$

The Extreme Type I cumulative distribution function of the yearly maximum wind can be approximated as:

$$F_X(V) = exp\left[-e^{\frac{-8.5063}{\mu_N}(V - 0.9321\ \mu_N)}\right] \qquad 4.8$$

where V is the design wind speed. The design wind speed can be determined for various return periods. Assuming that the design wind speed corresponds to the 50-year wind, the cumulative distribution function, $F_X(V)$ in Equation 4.8 can be set at 1 - 1/50 or 0.98. The 50-year design wind speed is then:

$$V = 1.3908\ \mu_N \qquad 4.9$$

By combining Equation 4.9 with Equations 4.4 and 4.7, the parameters of individual wind can be solved in terms of the 50-year design wind:

$$u = 0.2259\ V \qquad 4.10$$

$$\alpha = \frac{11.80}{V} \qquad 4.11$$

The 50-year wind speed at a site is generally readily available. For example, ANSI A58.1-1982 contains a map as well as a tabular listing in an Appendix of 50-year wind speeds. Equations 4.10 and 4.11 allow the engineer to directly use the 50-year wind speed to solve for the parameters u and α and calculate the mean and standard deviation of the individual wind speeds to be expected at that location.

The structural engineer has a greater interest in wind pressures than in wind speeds. Temporal statistics for wind pressures, such as mean rate of occurrence and mean duration, will be identical to those for wind speeds. Wind pressure intensities can be derived using statistical data on wind velocities, pressure coefficients, exposure data, and gust factors. While the wind speed can clearly be modelled with an Extreme Type I distribution, it is not obvious what probability distribution the wind pressure will follow. The wind pressure is proportional to the wind velocity squared. The square of an Extreme Type I variable will not necessarily follow an Extreme Type I distribution. The other factors involved in the calculation of wind pressures (pressure coefficients, exposure factors, and gust factors) are also random variables and contribute to the difficulty in determination of a proper distribution.

Ellingwood et al. (1980) determined through Monte Carlo simulation and numerical integration that wind pressure data can be fitted very well by an Extreme Type I distribution. This study was based on the assumption that the pressure coefficient, exposure coefficient, and gust factor follow a normal distribution and the wind velocity follows an Extreme Type I distribution. A factor of 0.85 is also included to account for the reduced probability that the direction of the maximum wind speed will cause the worst response of the structure. Composite statistical estimates were determined from the data for seven sites in the continental United States. These sites (Austin, Baltimore, Detroit, Rochester, Sacramento, St. Louis, and Tucson) correspond to seven of the nine sites analyzed in this research.

Assuming the Extreme Type I distribution to be proper for wind pressures, equations similar to 4.10 and 4.11 can be derived in terms of annual and 50-year maximum pressures. The coefficient of variation of the annual maximum wind pressure is given in Ellingwood et al. (1980) as 0.59. Utilizing this value, Equation 4.4 becomes:

$$\alpha = \frac{2.168}{\mu_N} \tag{4.12}$$

where μ_N is the mean annual wind pressure. Combining Equations 4.1, 4.6, and 4.12 results in:

$$u_N = 0.7344\,\mu_N \tag{4.13}$$

Combining Equations 4.12 and 4.13 with Equation 3.7:

$$u = -1.6842\,\mu_N \tag{4.14}$$

Ellingwood et al. (1980) obtained the ratio of the mean annual wind pressure to the nominal wind pressure as 0.33. Incorporating this value into Equations 4.12 and 4.14, the Extreme Type I parameters for individual wind pressures can be defined in terms of the nominal design wind pressure as follows:

$$u = -0.556\,p \qquad 4.15$$

$$\alpha = \frac{6.57}{p} \qquad 4.16$$

where p is the nominal design wind pressure obtained using ANSI A58.1-1982. The negative value of u in Equation 4.15 results from the high coefficient of variation in the annual maximum wind pressure.

Using the parameters in Equations 4.15 and 4.16 implies a mean 50-year wind pressure of 0.838p and a coefficient of variation of 0.211. These results are inconsistent with the results from Ellingwood et al. (1980), in which a mean 50-year wind pressure of 0.78p and a coefficient of variation of 0.37 was obtained. Using the 50-year statistics of Ellingwood et al. (1980) results in parameters of the Extreme Type I distribution for the individual wind pressure of:

$$u = -1.410\,p \qquad 4.17$$

$$\alpha = \frac{4.44}{p} \qquad 4.18$$

It is unclear whether Equations 4.15, 16 (yearly maximum wind pressure) or Equations 4.17, 18 (50-year maximum wind pressure) are more appropriate. One possibility is to obtain parameters u and α of the individual wind pressures such that the annual maximum and 50-year maximum mean values correspond to the respective mean values in Ellingwood et al. (1980). The resulting parameters of the Extreme Type I distribution for individual wind pressures are:

$$u = -0.339\, p \qquad 4.19$$

$$\alpha = \frac{8.70}{p} \qquad 4.20$$

These parameters result in a coefficient of variation of 0.448 for the annual maximum wind pressure and a coefficient of variation of 0.189 for the 50-year wind pressure. For both the annual and 50-year maximum wind pressure, Equations 4.19 and 4.20 significantly lowers the coefficients of variation from that given in Ellingwood et al. (1980).

Clearly, additional research is needed in the area of extreme wind pressures. Once the appropriate parameters are defined, they can be used to completely define the wind pressures to be expected at a site. This information can be utilized in various load combination design methods.

V. CONCLUSIONS

Wind speed data for nine sites in the contiguous United States have been modelled as a poisson process by analyzing continuous strip gust records. The modelling was limited to structurally significant wind storms by considering velocities of less than 20 knots as zero. Occurrence dependence was accounted for through a clustered process.

At least two years of continuous records were analyzed for all of the sites examined. A more intense study of one site, Knoxville, was conducted to examine stationarity between years. It was concluded that two years of data are sufficient to accurately model the wind at a site.

Since the nine sites analyzed in this report provide a good geographic representation of the continental United States, pertintent statistics of the wind process were averaged to provide statistics for the contiguous United States. Results were a mean rate of occurrence of 0.01589/hour, a mean cluster duration of 4.99 hours, a mean individual duration of 3.46 hours, and a mean number of winds per cluster of 1.362.

Based on an Extreme Type I distribution for wind intensity, appropriate parameters for individual winds have been developed as a function of the 50-year design wind speed. These parameters are:

$$u = 0.2259\ V$$
$$\alpha = \frac{11.80}{V} \qquad 5.1$$

where V is the 50-year design wind speed.

Assuming that the Extreme Type I distribution is also proper for maximum wind pressures, the parameters for the individual pressures are:

$$\begin{aligned} u &= -0.339p \\ \alpha &= \frac{8.70}{p} \end{aligned} \qquad 5.2$$

where p is the nominal design wind pressure.

More accurate approximations for these parameters and average composite United States statistics could be obtained from the analysis of a larger set of data. Utilizing site-specific statistics, when available, would provide more accurate results than Equations 5.1 and 5.2. The use of proper statistics of the wind process in state-of-the-art load combination analyses would significantly improve the efficiency and safety of probability-based structural design.

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