2 Prediction of design wind speeds and structural safety

2.1 Introduction and historical background

The establishment of appropriate design wind speeds is a critical first step towards the calculation of design wind loads for structures. It is also usually the most uncertain part of the design process for wind loads, and requires the statistical analysis of historical data on recorded wind speeds.

In the 1930s, the use of the symmetrical bell-shaped *Gaussian* distribution (Appendix C3.1) to represent extreme wind speeds, for the prediction of long-term design wind speeds was proposed. However this failed to take note of the earlier theoretical work of Fisher and Tippett (1928), establishing the limiting forms of the distribution of the largest (or smallest) value in a fixed sample, depending on the form of the tail of the parent distribution. The identification of the three types of *extreme value distribution* was of prime significance to the development of probabilistic approaches in engineering in general.

The use of extreme value analysis for design wind speeds lagged behind the application to flood analysis. Gumbel (1954) strongly promoted the use of the simpler Type I extreme value distribution for such analyses. However, Jenkinson (1955) showed that the three asymptotic distributions of Fisher and Tippett could be represented as a single *Generalized Extreme Value Distribution* – this is discussed in detail in a following section. In the 1950s and the early 1960s, several countries had applied extreme value analyses to predict design wind speeds. In the main, the Type I (by now also known as the 'Gumbel Distribution'), was used for these analyses. The concept of *return period* also arose at this time.

The use of probability and statistics as the basis for the modern approach to wind loads was, to a large extent, a result of the work of A. G. Davenport in the 1960s, recorded in several papers (e.g. Davenport, 1961).

In the 1970s and 1980s, the enthusiasm for the then standard 'Gumbel analysis' was tempered by events such as Cyclone 'Tracy' in Darwin, Australia (1974) and severe gales in Europe (1987), when the previous design wind speeds determined by a Gumbel fitting procedure, were exceeded considerably. This highlighted the importance of:

- sampling errors inherent in the recorded data base, usually less than fifty years, and
- the separation of data originating from different storm types.

The need to separate the recorded data by storm type was recognized in the 1970s by Gomes and Vickery (1977a).

The development of probabilistic methods in structural design generally, developed in parallel with their use in wind engineering, followed pioneering work by Freudenthal (1947, 1956) and Pugsley (1966). This area of research and development is known as 'structural reliability' theory. Limit states design, which is based on probabilistic concepts, was steadily introduced into design practice from the 1970s onwards.

This chapter discusses modern approaches to the use of extreme value analysis for prediction of extreme wind speeds for the design of structures. Related aspects of structural design and safety are discussed in Section 2.6.

2.2 Principles of extreme value analysis

The theory of extreme value analysis of wind speeds, or other geophysical variables, such as flood heights, or earthquake accelerations, is based on the application of one or more of the three asymptotic extreme value distributions identified by Fisher and Tippett (1928), and discussed in the following section. They are asymptotic in the sense that they are the correct distributions for the largest of an *infinite* population of independent random variables of known probability distribution. In practice, of course, there will be a finite number in a population, but in order to make predictions, the asymptotic extreme value distributions are still used as empirical fits to the extreme data. Which one of the three is the theoretically 'correct', depends on the form of the tail of the underlying parent distribution. However unfortunately this form is not usually known with certainty due to lack of data. Physical reasoning has sometimes been used to justify the use of one or other of the asymptotic Extreme Value distributions.

Gumbel (1954, 1958) has covered the theory of extremes in detail. A useful review of the various methodologies available for the prediction of extreme wind speeds, including those discussed in this chapter, has been given by Palutikof *et al.* (1999).

2.2.1 The Generalized Extreme Value Distribution

The *Generalized Extreme Value Distribution* (G.E.V.) introduced by Jenkinson (1955) combines the three Extreme Value distributions into a single mathematical form:

$$F_U(U) = \exp\{-[1 - k(U-u)/a]^{1/k}\} \tag{2.1}$$

where $F_U(U)$ is the cumulative probability distribution function (see Appendix C) of the maximum wind speed in a defined period (e.g. one year).

In equation (2.1), k is a shape factor and a is a scale factor. When $k < 0$, the G.E.V. is known as the *Type II Extreme Value* (or *Frechet*) Distribution; when $k > 0$, it becomes a *Type III Extreme Value Distribution* (a form of the *Weibull* Distribution). As k tends to 0, equation (2.1) becomes equation (2.2) in the limit. Equation (2.2) is the *Type I Extreme Value Distribution*, or *Gumbel* Distribution.

$$F_U(U) = \exp\{-\exp[-(U-u)/a]\} \tag{2.2}$$

The G.E.V. with k equal to –0.2, 0 and 0.2 are plotted in Figure 2.1, in a form that the Type I appears as a straight line. As can be seen the Type III ($k = +0.2$) curves in a way to approach a limiting value – it is therefore appropriate for variables that are 'bounded' on the high side. It should be noted that the Type I and Type II predict unlimited values – they are therefore suitable distributions for variables that are 'unbounded'. Since we would expect that there is an upper limit to the wind speed that the atmosphere can produce, the Type III Distribution may be more appropriate for wind speed.

A method of fitting the Generalized Extreme Value Distribution to wind data is dis-

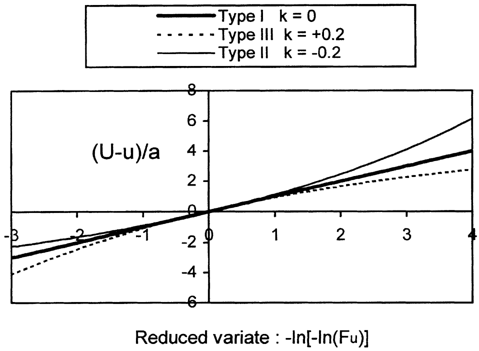

Figure 2.1 The Generalized Extreme Value Distribution (k = –0.2, 0. +0.2).

cussed in Section 2.4. An alternative method is the method of probability-weighted moments described by Hosking *et al.* (1985).

2.2.2 Return period

At this point it is appropriate to introduce the term *Return Period, R*. It is simply the inverse of the complementary cumulative distribution of the extremes.

$$\text{i.e. Return period, } R = \frac{1}{\text{Probability of exceedence}} = \frac{1}{1 - F_U(U)}$$

Thus, if the annual maximum is being considered, then the return period is measured in years. Thus a 50–year return period wind speed has a probability of exceedence of 0.02 (1/50) in any one year. It should not be interpreted as recurring regularly every 50 years. The probability of a wind speed, of given return period, being exceeded in a lifetime of a structure is discussed in Section 2.6.3.

2.2.3 Separation by storm type

In Chapter 1, the various types of windstorm that are capable of generating winds strong enough to be important for structural design, were discussed. These different event types will have different probability distributions, and therefore should be statistically analysed separately; however, this is usually quite a difficult task as weather bureaux or meteorological offices do not normally record the necessary information. If anemograph records such as those shown in Figures 1.5 and 1.7 are available, these can be used for identification purposes – although this is a time-consuming and painstaking task!

The relationship between the combined return period, R_c for a given extreme wind speed due to winds of either type, and for those calculated separately for storm types 1 and 2, (R_1 and R_2) is:

$$\left(1 - \frac{1}{R_c}\right) = \left(1 - \frac{1}{R_1}\right)\left(1 - \frac{1}{R_2}\right) \tag{2.3}$$

Equation (2.3) relies on the assumption that exceedence of wind speeds from the two different storm types are independent events.

2.2.4 Simulation methods for tropical cyclone wind speeds

The winds produced by severe tropical cyclones, also known as 'hurricanes' and 'typhoons' are the most severe on earth (apart from those produced by tornadoes which affect very small areas). However, their infrequent occurrence at particular locations, often makes the historical record of recorded wind speeds an unreliable predictor for design wind speeds. An alternative approach, which gained popularity in the 1970s and early 1980s, was the simulation or 'Monte-Carlo' approach, introduced originally for offshore engineering by Russell (1971). In this procedure, satellite and other information on storm size, intensity and tracks are made use of to enable a computer-based simulation of wind speed (and in some cases direction) at particular sites. Usually, established probability distributions are used for parameters such as: central pressure and radius to maximum winds. A recent use of these models is for damage prediction for insurance companies. The disadvantage of this approach is the subjective aspect resulting from the complexity of the problem. Significantly varying predictions could be obtained by adopting different assumptions. Clearly whatever recorded data that is available, should be used to calibrate these models.

2.2.5 Compositing data from several stations

No matter what type of probability distribution is used to fit historical extreme wind series, or what fitting method is used, extrapolations to high return periods for ultimate limit states design (either explicitly, or implicitly through the application of a wind load factor), are usually subject to significant sampling errors. This results from the limited record lengths usually available to the analyst. In attempts to reduce the sampling errors, a recent practice has been to combine records from several stations with perceived similar wind climates to increase the available records for extreme value analysis. Thus 'superstations' with long records can be generated in this way.

For example, in Australia, stations in a huge region in the southern part of the country have been judged to have similar statistical behaviour, at least as far as the all-direction extreme wind speeds are concerned. A single set of design wind speeds has been specified for this region (Standards Australia, 1989). A similar approach has been adopted in the United States (ASCE, 1998; Peterka and Shahid, 1998).

2.2.6 Incorporation of wind direction effects

Increased knowledge of the aerodynamics of buildings and other structures, through wind-tunnel and full-scale studies, has revealed the variation of structural response as a function of wind direction as well as speed. The approaches to probabilistic assessment of wind loads including direction, can be divided into those based on the parent distribution of wind speed, and those based on extreme wind speeds. In many countries, the extreme winds are produced by rare severe storms such as thunderstorms and tropical cyclones, and there is no direct relationship between the parent population of regular everyday winds, and the extreme winds. For such locations, (which would include most tropical and sub-tropical countries), the latter approach is more appropriate. Where a separate analysis of extreme wind speeds by direction sector has been carried out, the relationship between the return period, R_a, for exceedence of a specified wind speed from *all* direction sectors, and the return periods for the same wind speed from direction sectors θ_1, θ_2 etc, is given in equation (2.4).

$$\left(1 - \frac{1}{R_a}\right) = \prod_{i=1}^{N}\left(1 - \frac{1}{R_{\theta_i}}\right) \tag{2.4}$$

Equation (2.4) follows from the assumption that wind speeds from each direction sector are statistically independent of each other, and is a statement of the following:

Probability that a wind speed U is *not* exceeded for all wind directions =
(probability that U is not exceeded from direction 1)
× (probability that U is not exceeded from direction 2)
× (probability that U is not exceeded from direction 3)
.....etc

Equation (2.4) is a similar relationship to (2.3) for combining extreme wind speeds from different types of storms.

2.3 The Gumbel approach to extreme wind estimation

Gumbel (1954) gave an easily usable methodology for fitting recorded annual maxima to the Type I Extreme Value distribution. This distribution is a special case of the Generalized Extreme Value distribution discussed in Section 2.2.1. The Type I distribution takes the form of equation (2.2) for the cumulative distribution $F_U(U)$:

$$F_U(U) = \exp\{-\exp[-(U-u)/a]\} \tag{2.2}$$

where u is the mode of the distribution, and a is a scale factor.

The return period, R, is directly related to the cumulative probability distribution, $F_U(U)$, of the annual maximum wind speed at a site as follows:

$$R = \frac{1}{1 - F_U(U)} \tag{2.5}$$

Substituting for $F_U(U)$ from equation (2.5) in (2.2), we obtain:

$$U_R = u + a\left\{-\log_e\left[-\log_e\left(1 - \frac{1}{R}\right)\right]\right\} \tag{2.6}$$

For large values of return period, R, equation (2.6) can be written:

$$U_R \cong u + a\log_e R \tag{2.7}$$

In Gumbel's original extreme value analysis method (applied to flood prediction as well as extreme wind speeds), the following procedure is adopted:

- the largest wind speed in each calendar year of the record is extracted
- the series is ranked in order of smallest to largest: 1,2,…m…. to N
- each value is assigned a probability of non-exceedence, p, according to
  $p \approx m/(N + 1)$ (2.8)
- a reduced variate, y, is formed from:
  $y = -\log_e(-\log_e p)$ (2.9)
  y is an estimate of the term in { } brackets in equation (2.6)
- the wind speed, U, is plotted against y, and a line of 'best fit' is drawn, usually by means of linear regression.

The above procedure has been used many times to analyse extreme wind speeds for many parts of the world. There are two disadvantages to the above approach, however:

- Assuming that the Type I Extreme Value Distribution is in fact the correct one, the fitting method is biased, that is equation (2.8) gives distorted values for the probability of non-exceedence, especially for high values of p near 1. Several alternative fitting methods have been devised which attempt to remove this bias. However most of these are more difficult to apply, especially if N is large, and some involve the use of computer programs to implement. A simple modification to the Gumbel procedure, which gives nearly unbiased estimates for this probability distribution, is due to *Gringorten* (1963). Equation (2.8) is replaced by the following modified formula:

  $p \approx (m - 0.44)/(N + 1 - 0.88) = (m - 0.44)/(N + 0.12)$ (2.10)

  An alternative procedure is the 'best linear unbiased estimators' proposed by Lieblein (1974), in which the annual maxima are ordered, and the parameters of the distribution are obtained by weighted sums of the extreme values.
- As may be seen from equation (2.7) and Figure 2.1, the Type I, or Gumbel Distribution will predict unlimited values of U_R, as the return period, R, increases. That is as R becomes larger, U_R as predicted by equation (2.6) or (2.7) will also increase without limit. As discussed in Section 2.2.1, this can be criticized on physical grounds, as there must be upper limits to the wind speeds that can be generated in the atmosphere in different types of storms. This behaviour, although unrealistic, may be acceptable for codes and standards.

2.3.1 Example of the use of Gumbel's method

Wind gust data has been obtained from a military airfield at East Sale, Victoria, Australia continuously since late 1951. The anemometer position has been constant throughout that period, and the height of the anemometer head has always been the standard meteorological value of 10 m. Thus in this case no corrections for height and terrain are required. Also the largest gusts have almost entirely been produced by gales from large synoptic depressions (Section 1.3.1). However, the few gusts that were produced by thunderstorm downbursts were eliminated from the list, in order to produce a statistically consistent population (see Section 2.2.3).

The annual maxima for the 47 calendar years 1952 to 1998 are listed in Table 2.1 following. The values in Table 2.1 are sorted in order of increasing magnitude (Table 2.2) and assigned a probability, p, according to (i) the Gumbel formula (equation (2.8)), and (ii) the Gringorten formula (equation (2.10)). The reduced variate, $-\log_e(-\log_e p)$, according to

Table 2.1 Annual maximum gust speeds from East Sale, Australia 1952–1998 (synoptic winds)

Year	*Maximum gust speed (m/s)*
1952	31.4
1953	33.4
1954	29.8
1955	30.3
1956	27.8
1957	30.3
1958	29.3
1959	36.5
1960	29.3
1961	27.3
1962	31.9
1963	28.8
1964	25.2
1965	27.3
1966	23.7
1967	27.8
1968	32.4
1969	27.8
1970	26.2
1971	30.9
1972	31.9
1973	27.3
1974	25.7
1975	32.9
1976	28.3
1977	27.3
1978	28.3
1979	28.3
1980	29.3
1981	27.8
1982	27.8
1983	30.9
1984	26.7
1985	30.3
1986	28.3
1987	30.3
1988	34.0
1989	28.8
1990	30.3
1991	27.3
1992	27.8
1993	28.8
1994	30.9
1995	26.2
1996	25.7
1997	24.7
1998	42.2

Table 2.2 Processing of East Sale data

Rank	*Gust speed (m/s)*	*Reduced variate (Gumbel)*	*Reduced variate (Gringorten)*
1	23.7	–1.354	–1.489
2	24.7	–1.156	–1.226
3	25.2	–1.020	–1.069
4	25.7	–0.910	–0.949
5	25.7	–0.816	–0.848
6	26.2	–0.732	–0.759
7	26.2	–0.655	–0.679
8	26.7	–0.583	–0.604
9	27.3	–0.515	–0.534
10	27.3	–0.450	–0.467
11	27.3	–0.388	–0.403
12	27.3	–0.327	–0.340
13	27.3	–0.267	–0.279
14	27.8	–0.209	–0.220
15	27.8	–0.151	–0.161
16	27.8	–0.094	–0.103
17	27.8	–0.037	–0.045
18	27.8	0.019	0.013
19	27.8	0.076	0.071
20	28.3	0.133	0.129
21	28.3	0.190	0.187
22	28.3	0.248	0.246
23	28.3	0.307	0.306
24	28.8	0.367	0.367
25	28.8	0.427	0.428
26	28.8	0.489	0.492
27	29.3	0.553	0.556
28	29.3	0.618	0.623
29	29.3	0.685	0.692
30	29.8	0.755	0.763
31	30.3	0.827	0.837
32	30.3	0.903	0.914
33	30.3	0.982	0.995
34	30.3	1.065	1.081
35	30.3	1.152	1.171
36	30.9	1.246	1.268
37	30.9	1.346	1.371
38	30.9	1.454	1.484
39	31.4	1.572	1.607
40	31.9	1.702	1.744
41	31.9	1.848	1.898
42	32.4	2.013	2.075
43	32.9	2.207	2.285
44	33.4	2.442	2.544
45	34.0	2.740	2.885
46	36.5	3.157	3.391
47	42.2	3.861	4.427

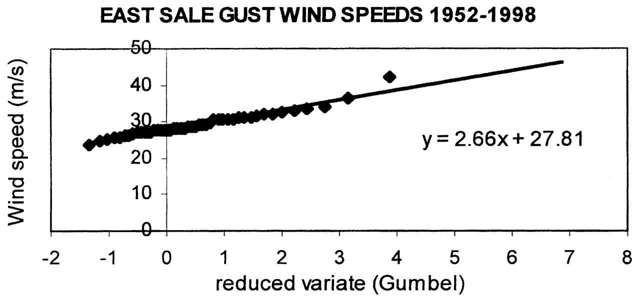

Figure 2.2 Analysis of annual maximum wind gusts from East Sale, using the Gumbel method.

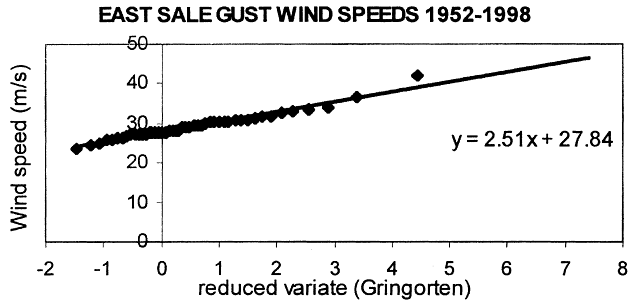

Figure 2.3 Analysis of annual maximum wind gusts from East Sale, using the Gringorten fitting method.

equation (2.9), is formed for both cases. These are tabulated in Table 2.2. The wind speed is plotted against reduced variate, and a straight line is fitted. The results of this are shown in Figures 2.2 and 2.3, for the Gumbel and Gringorten methods, respectively. The intercept and slope of these lines give the mode, u, and slope, a, of the fitted Type I Extreme Value Distribution, according to equation (2.1).

Predictions of extreme wind speeds for various return periods can then readily be obtained by application of either equation (2.6) or (2.7). In this case, Table 2.3 lists these

Table 2.3 Prediction of extreme wind speeds for East Sale (synoptic winds)

Return period (years)	*Predicted gust speed (m/s) (Gumbel)*	*Predicted gust speed (m/s) (Gringorten)*
10	33.8	33.5
20	35.7	35.3
50	38.2	37.7
100	40.1	39.4
200	41.9	41.2
500	44.3	43.5
1000	46.2	45.2

predictions based on the two fitting methods. For return periods up to 50 years, the predicted values by the two methods are within 0.5 m/s of each other; gradual divergence occurs for higher return periods. However these small differences are swamped by sampling errors, i.e. the errors inherent in trying to make predictions for return periods of 100 years or more from less than fifty years of data. This problem is illustrated by the following exercise. The problem of high sampling errors can often be circumvented by compositing data, as discussed in Section 2.2.5.

2.3.1.1 Exercise

Re-analyse the annual maximum gust wind speeds for East Sale for the years 1952 to 1997, i.e. ignore the high value recorded in 1998. Compare the resulting predictions of design wind speeds for (a) 50 years return period, and (b) 1000 years return period, and comment.

2.3.2 Use of dynamic pressure

Cook (1982) has proposed the use of extreme dynamic pressure (i.e. velocity squared) instead of velocity, in extreme value analyses using the Gumbel Distribution. This has the effect of introducing curvature into the velocity versus return period graph – that is, it has a similar shape to the Type III Extreme Value Distribution (Figure 2.1 with positive shape factor). A similar result can be obtained by fitting the Generalized Extreme Value Distribution, and allowing the data to 'find' its own shape factor. One approach for doing this is discussed in the following section.

2.4 The excesses over threshold approach

The approach of extracting a single maximum value of wind speed from each year of historical data, obviously has limitations in that there may be many storms during any year, and only one value from all these storms is being used. A shorter reference period than a year could, of course, be used to increase the amount of data. However, it is important for extreme value analysis that the data values be statistically independent – this will not be the case if a period as short as one day is used. An alternative approach which makes use only of the data of relevance to extreme wind prediction is the *excesses over threshold* approach (e.g. Davison and Smith, 1990; Lechner *et al.* 1992; Holmes and Moriarty, 1999).

A brief description of the method is given here. This is a method which makes use of all wind speeds from independent storms above a particular minimum threshold wind speed, u_0 (say 20 m/s). There may be several of these events, or none, during a particular year. The basic procedure is as follows:

- several threshold levels of wind speed are set: u_0, u_1, u_2, etc. (e.g. 20, 21, 22 …m/s)
- the exceedences of the lowest level u_0 by the maximum storm wind are identified, and the number of crossings of this level per year, λ, is calculated
- the differences $(U - u_0)$ between each storm wind and the threshold level u_0 are calculated and averaged (only positive excesses are counted)
- the previous step is repeated for each level, u_1, u_2 etc, in turn
- the mean excess is plotted against the threshold level

- a scale factor, σ, and a shape factor, k, are determined from the following equations (Davison and Smith, 1990)

$$\text{slope} = \frac{-k}{(1+k)}; \quad \text{intercept} = \frac{\sigma}{(1+k)} \tag{2.11}$$

Prediction of the R-year return period wind speed, U_R, can then be calculated from:

$$U_R = u_0 + \sigma[1 - (\lambda R)^{-k}]/k \tag{2.12}$$

In equation (2.12), the shape factor, k, is normally found to be positive (usually around 0.1). As R increases to very large values, the upper limit to U_R of $u_0 + (\sigma/k)$ is gradually approached.

When k is zero, it can be shown mathematically that equation (2.12) reduces to equation (2.13)

$$U_R = u_0 + \sigma\log_e(\lambda R) \tag{2.13}$$

The similarity between equations (2.7) and (2.13) should be noted.

The highest threshold level, u_n, should be set so that it is exceeded by at least ten wind speeds. An example of this method is given in the following section.

2.4.1 Example of the use of the 'excesses over threshold' method

Daily wind gusts at several stations in the Melbourne, Australia, area since 1940 have been recorded. Those at the four airport locations of Essendon, Moorabbin, Melbourne Airport (Tullamarine), and Laverton, are the most useful since the anemometers are located at positions most closely matching the ideal open country conditions, and away from the direct influence of buildings. Table 2.4 summarizes the data available from these four stations.

The two most common types of event producing extreme wind in the Melbourne area are gales produced by the passage of large low pressure or frontal systems ('synoptic' winds – see Section 1.3.1), and severe thunderstorm 'downbursts' (Section 1.3.3). Downbursts are usually accompanied by thunder, but the occurrence of thunder does not necessarily mean that an extreme gust has been generated by a downburst. The occurrences of downbursts in the data from the four stations were identified by inspection of the charts

Table 2.4 Summary of data for Melbourne stations

Station	*Station number*	*Years*	*Maximum recorded gust (m/s)*	*Rate/year (synoptic gusts >21 m/s)*	*Rate/year (downburst gusts >21 m/s)*
Essendon	86038	1940–71	40.6	34.6	1.1
Moorabbin	86077	1972–92	41.2	19.3	0.7
Tullamarine	86282	1970–97	38.6	30.1	1.3
Laverton	87031	1946–95[a]	42.7	28.4	0.8

[a] *Note:* 1953, 1954 and 1956 are missing from Laverton data.

stored by the Australian Bureau of Meteorology or the National Archives. Table 2.4 shows that the rate of occurrence of downbursts greater than 21 m/s is quite low (around one per year at each station); however, as will be seen they are significant contributors to the largest gusts.

The largest recorded gusts in the Melbourne area are listed in Table 2.5. Approximately half of these were generated by downbursts.

Extreme value analysis of the data was carried out in the following stages:

- Daily gusts over 21 m/s were retained for analysis
- Gusts generated by downbursts were identified by inspection of anemometer charts, and separated from the synoptic gusts
- The data from the four stations were composited into single data sets, for both downburst gusts and synoptic gusts
- The synoptic data were corrected to a uniform height (10 m), and approach terrain (open country), using correction factors according to direction derived from wind-tunnel tests for each station
- For both data sets, the 'excesses over threshold' analysis was used to derive relationships between wind speed and return period

The last stage enabled a scale factor, σ, and a shape factor, k, to be determined in the relationship:

$$U_R = u_0 + \sigma[1 - (\lambda R)^{-k}]/k \qquad (2.12)$$

u_0 is the lowest threshold – in this case 21 m/s, and λ is average annual rate of exceedence of u_0, for the combined data sets. For the current analysis, λ was 23.4 for the synoptic data, and 0.97 for the downburst data.

The results of the two analyses were expressed in the following forms for the Melbourne data:

For synoptic winds:

Table 2.5 Largest recorded gusts in the Melbourne area 1940–97

Date	*Station*	*Gust speed (knots)*	*Gust speed (m/s)*	*Type*
14/1/1985	Laverton	83	42.7	Synoptic
25/12/1978	Moorabbin	80	41.2	Downburst
6/9/1948	Essendon	79	40.6	Synoptic
15/11/1982	Tullamarine	75	38.6	Downburst
3/1/1981	Tullamarine	74	38.1	Downburst
26/10/1978	Laverton	71	36.5	Downburst
4/8/1947	Essendon	70	36.0	Synoptic
27/2/1973	Laverton	70	36.0	Downburst
8/11/1988	Tullamarine	70	36.0	Synoptic
1/7/1942	Essendon	67	34.5	Downburst
5/8/1959	Laverton	67	34.5	Synoptic
24/1/1982	Laverton	67	34.5	Downburst
10/8/1992	Tullamarine	67	34.5	Synoptic

$$U_R = 68.3 - 39.3R_1^{-0.059} \tag{2.14}$$

For downburst winds:

$$U_R = 69.0 - 48.1R_2^{-0.108} \tag{2.15}$$

The combined probability of exceedence of a given gust speed from either type of wind is obtained by substituting in equation (2.3):

$$\frac{1}{R_C} = 1 - \left[1 - \left(\frac{68.3 - U_R}{39.3}\right)^{\frac{1}{0.059}}\right]\left[1 - \left(\frac{69.0 - U_R}{48.1}\right)^{\frac{1}{0.108}}\right] \tag{2.16}$$

Equations (2.14), (2.15) and (2.16) are plotted in Figure 2.4. The lines corresponding to equations (2.14) and (2.15) cross at a return period of 30 years. It can also be seen that the combined wind-speed return period relationship is asymptotic to the synoptic line at low return periods, and to the downburst line at high return periods.

2.5 Parent wind distributions

For some design applications it is necessary to have information on the distribution of the complete population of wind speeds at a site. An example is the estimation of fatigue damage for which account must be taken of damage accumulation over a range of wind storms (see Section 5.6). The population of wind speeds produced by synoptic wind storms at a site is usually fitted with a distribution of the *Weibull* type:

$$f_U(\bar{U}) = \frac{k\bar{U}^{k-1}}{c^k}\exp\left[-\left(\frac{\bar{U}}{c}\right)^k\right] \tag{2.17}$$

Equation (2.17) represents the probability density function for mean wind speeds produced by synoptic events. There are two parameters: a *scale factor*, c, which has units of

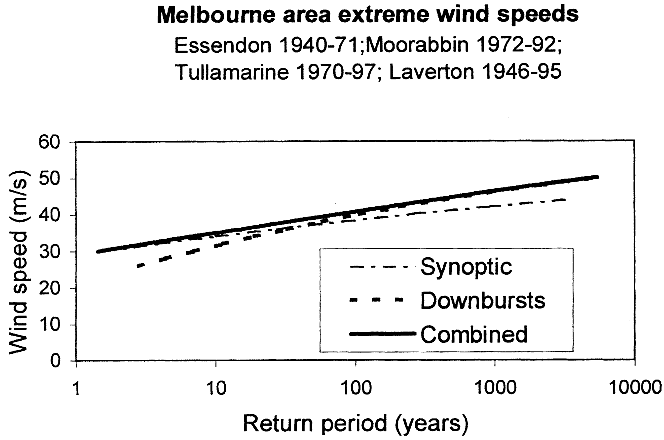

Figure 2.4 Wind speed versus return period for the Melbourne area.

wind speed, and a *shape factor*, k, which is dimensionless (see also Appendix C3.4). The probability of *exceedence* of any given wind speed is given by equation (2.18)

$$1 - F(\bar{U}) = \exp\left[-\left(\frac{\bar{U}}{c}\right)^k\right] \tag{2.18}$$

Typical values of c are 3 to 10 m/s, and k usually falls in the range 1.3 to 2.0. An example of a Weibull fit to recorded meteorological data is shown in Figure 2.5.

Several attempts have been made to predict extreme winds from knowledge of the parent distribution of wind speeds, and thus make predictions from quite short records of wind speed at a site (e.g. Gomes and Vickery, 1977b). The 'asymptotic' extreme value distribution for a Weibull parent distribution is the Type I, or Gumbel, distribution. However such approaches are unlikely to be as accurate as direct analysis of extreme values as discussed in Sections 2.2 to 2.4. They also will give misleading results in mixed wind climates, as short duration wind storms, such as thunderstorm downbursts will not be fully represented in the parent distribution, which is usually derived from data recorded a few times per day at most.

2.6 Wind loads and structural safety

The development of structural reliability concepts – i.e. the application of probabilistic methods to the structural design process, has accelerated the adoption of probabilistic methods into wind engineering since the 1970s. The assessment of wind loads is only one part of the total structural design process, which also includes the determination of other loads and the resistance of structural materials. The structural engineer must proportion the structure so that collapse or overturning has a very low risk of occurring, and defined serviceability limits on deflection, acceleration, etc., are not exceeded very often.

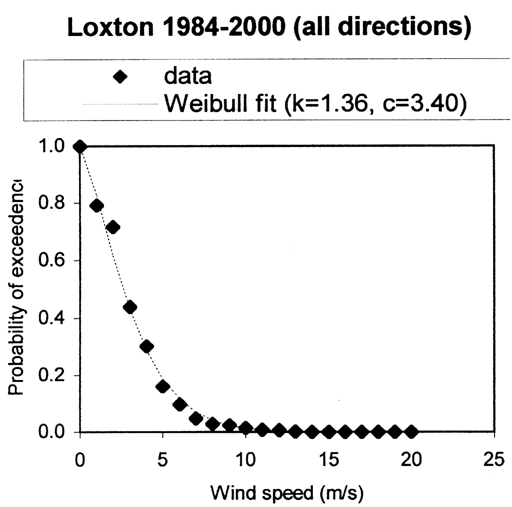

Figure 2.5 Example of a Weibull distribution fit to parent population of synoptic winds.

2.6.1 Limit states design

Limit states design is a rational approach to the design of structures, which has gradually become accepted around the world. As well as explicitly defining the ultimate and serviceability limit states for design, the method takes a more rational approach to structural safety by defining 'partial' load factors ('gamma' factors) for each type of loading, and a separate resistance factor ('phi' factor) for the resistance. The application of the limit states design method is not, in itself, a probabilistic process, but probability is usually used to derive the load and resistance factors.

A typical ultimate limit states design relationship involving wind loads, is as follows.

$$\varphi R \geq \gamma_D D + \gamma_W W \tag{2.19}$$

where φ is a resistance factor, R is the nominal structural resistance, γ_D is the dead load factor, D is the nominal dead load, γ_W is the wind load factor and W is the nominal wind load.

In this relationship, the partial factors, φ, γ_D, and γ_W are adjusted separately to take account of the variability and uncertainty in the resistance, dead load and wind load. The values used also depend on what particular nominal values have been selected. Often a final calibration of a proposed design formula is carried out by evaluating the safety, or reliability, index as discussed in the following section, for a range of design situations, e.g. various combinations of nominal dead and wind loads.

2.6.2 Probability of failure and the safety index

A quantitative measure of the safety of structures, known as the *safety index*, or *reliability index*, is used in many countries as a method of calibration of existing and future design methods for structures. As will be explained in this section, there is a one-to-one relationship between this index and a probability of failure, based on the exceedence of a design resistance by an applied load (but not including failures by human errors and other accidental causes).

The design process is shown in its simplest form in Figure 2.6. The design process consists of comparing a structural load effect, S, with the corresponding resistance, R. In the case of limit states associated with structural strength or collapse, the load effect could be an axial force in a member or a bending moment, or the corresponding stresses. In the case of serviceability limit states, S and R may be deflections, accelerations or crack widths.

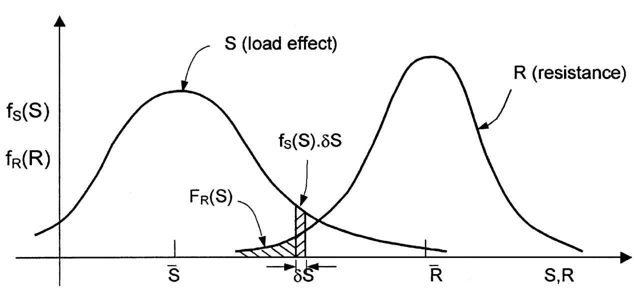

Figure 2.6 Probability densities for load effects and resistance.

The probability density functions $f_s(S)$ and $f_R(R)$ for a load effect, S, and the corresponding structural resistance, R are shown in Figure 2.6. (Probability density is defined in Section C2.1 in Appendix C.) Clearly, S and R must have the same units. The dispersion or 'width' of the two distributions represents the uncertainty in S and R.

Failure (or unserviceability) occurs when the resistance of the structure is less than the load effect. The probability of failure will now be determined assuming S and R are statistically independent.

The probability of failure occurring at a load effect between S and $S + \delta S$

=[probability of load effect lying between S and $S + \delta S$] $\times$
[probability of resistance, R, being less than S]

$$=[f_S(S)\delta S] \times F_R(S) \tag{2.20}$$

where $F_R(R)$ is the cumulative probability distribution of R, and,

$$F_R(S) = \int_{-\infty}^{S} f_R(R)dR \tag{2.21}$$

The terms in the product in equation (2.20) are the areas shown in Figure 2.6.

The total probability of failure is obtained by summing, or integrating, equation (2.20) over all possible values of S (between $-\infty$ and $+\infty$):

$$p_f = \int_{-\infty}^{\infty} f_S(S).F_R(S)dS \tag{2.22}$$

Substituting for $F_R(S)$ from equation (2.21) into (2.22),

$$p_f = \int_{-\infty}^{\infty}\int_{-\infty}^{S} f_S(S).f_R(R).dR.dS = \int_{-\infty}^{\infty}\int_{-\infty}^{S} f(S,R).dR.dS \tag{2.23}$$

where $f(S,R)$ is the *joint* probability density of S,R.

The values of the probability of failure computed from equation (2.23) are normally very small numbers, typically 1×10^{-2} to 1×10^{-5}.

The safety, or reliability, index is defined according to equation (2.24), and normally takes values in the range 2 to 5.

$$\beta = \Phi^{-1}(p_f) \tag{2.24}$$

where $\Phi^{-1}()$ is the inverse cumulative probability distribution of a unit normal (Gaussian) variate, i.e. a normal variate with a mean of zero and a standard deviation of one.

The relationship between the safety index, β, and the probability of failure, p_f, according to equation (2.24) is shown plotted in Figure 2.7.

Equations (2.23) and (2.24) can be evaluated exactly when S and R are assumed to have Gaussian (normal) or lognormal (Appendix C3.2) probability distributions. However, in other cases (which include those involving wind loading), numerical methods must be

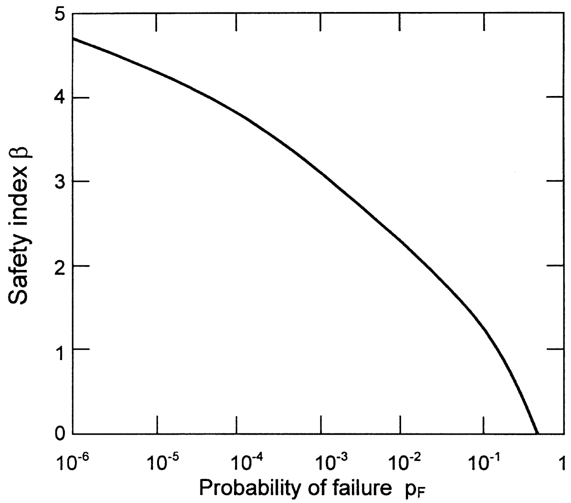

Figure 2.7 Relationship between safety index and probability of failure.

used. Numerical methods must also be used when, as is usually the case, the load effect, S, and resistance, R, are treated as combinations (sums and products) of separate random variables, with separate probabilistic characteristics.

Details of structural reliability theory and practice can be found in a number of texts on the subject (e.g. Blockley (1980), Melchers (1987), Ang and Tang (1990)).

2.6.3 Nominal return period for design wind speeds

The return periods (or annual risk of exceedence) for the nominal design wind speeds in various wind loading codes and standards are discussed in Chapter 15. The most common choice is fifty years. There should be no confusion between return period, R, and expected lifetime of a structure, L. The return period is just an alternative statement of annual risk of exceedence, e.g. a wind speed with a fifty-year return period is one with an expected risk of exceedence of 0.02 (1/50) in any one year. However the risk, r, of exceedence of a wind speed *over the lifetime,* can be determined by assuming that all years are statistically independent of each other.

Then,

$$r = 1 - \left[1 - \left(\frac{1}{R}\right)\right]^L \tag{2.25}$$

Equation (2.25) is very similar to equation (2.4) in which the combined probability of exceedence of a wind speed occurring over a range of wind directions was determined.

Setting both R and L as fifty years in equation (2.25), we arrive at a value of r of 0.636. There is thus a nearly 64% chance that the fifty-year return period wind speed will be exceeded at least once during a fifty-year lifetime – i.e. a better than even chance that it *will occur*. Wind loads derived from wind speeds with this level of risk must be factored up when used for ultimate limit states design. Typical values of wind load factor, γ_W, are in the range of 1.4 to 1.6. Different values may be required for regions with different wind speed/return period relationships.

The use of a return period for the nominal design wind speed substantially higher than the traditional 50 years, avoids the need to have different wind load factors in different regions. This was an important consideration in the revision of the Australian Standard for Wind Loads in 1989 (Standards Australia, 1989), which in previous editions, required the use of a special ‘Cyclone Factor’ in the regions of northern coastline affected by tropical cyclones. The reason for this factor was the greater rate of change of wind speed with return period in the cyclone regions. A similar ‘hurricane importance factor’ appeared in some editions of the American National Standard (ASCE 1993), but was later incorporated into the specified basic wind speed (ASCE 1998).

In subsequent editions of AS1170.2, the wind speeds for ultimate limits states design had a nominal probability of exceedence of 5% in a lifetime of fifty years (a return period of 1000 years, approximately).

However a load factor of 1.0 is applied to the wind loads derived in this way – and this factor is the same in both cyclonic and non-cyclonic regions.

2.6.4 Uncertainties in wind load specifications

A reliability study of structural design involving wind loads requires an estimation of all the uncertainties involved in the specification of wind loads – wind speeds, multipliers for terrain, height, topography, etc., pressure coefficients, local and area averaging effects, etc. Some examples of this type of study for buildings and communication towers are given by Pham *et al.* (1983, 1992).

2.7 Summary

In Chapter 2, the application of extreme value analysis to the prediction of design wind speeds has been discussed. In particular, the Gumbel, and ‘excesses over threshold’ approaches were described in detail. The need to separate wind speeds caused by windstorms of different types was emphasized, and wind direction effects were considered.

The main principles of the application of probability to structural design and safety are also introduced.

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