

Rational wind-load design and wind-load factors for locations affected by tropical cyclones, hurricanes and typhoons

J.D. Holmes¹

¹ JDH Consulting, P.O. Box 269, Mentone, Victoria, 3194, Australia; PH +61-3-9585-3815; e-mail: john.holmes@jdhconsult.com

ABSTRACT

This paper discusses rational approaches to the design of structures for wind loads, at locations where tropical cyclones, including hurricanes and typhoons, are the dominant windstorm event. The characteristics of the windstorm event itself, including current knowledge of mean velocity and gust profiles are discussed. Structural safety and reliability principles are discussed, in relation to design for wind loads in the extreme wind environments affected by tropical cyclones. It is shown that the greatest uncertainty is the wind speed itself. Typical variations of the basic wind speed with return period (annual risk of exceedence) and appropriate return periods and wind load factors for design are proposed.

INTRODUCTION

With the exception of tornados, tropical cyclones, known as ‘hurricanes’ and ‘typhoons’ in the North Atlantic and North West Pacific respectively, generate the strongest wind speeds of any storm type on Earth. Tornados can generate higher wind speeds, but affect much smaller areas and are generally ignored in structural design, except for high-risk structures such as nuclear facilities.

The purpose of this paper is to review design approaches for wind loads in areas affected by tropical cyclones that have developed in various parts of the world. Some emphasis is given to methods developed in Australia and the United States, based on the author’s own knowledge and experience.

THE NATURE OF TROPICAL CYCLONES

Tropical cyclones are intense cyclonic storms which occur over the tropical oceans, mainly in late summer and autumn. They are driven by the latent heat of the oceans, and require a minimum sea temperature of about 26 degrees Celsius to sustain them; they rapidly degenerate when they move over land, or into cooler waters. The conditions for a tropical storm with high convection to form into a tropical cyclone are complex: some initiating vorticity, assisted by the Coriolis force due to the Earth’s rotation, and low vertical wind shear, are required.

Tropical cyclones generally will not form within about five degrees of the Equator (due to the low Coriolis forces), and do not reach full strength until they reach at least ten degrees latitude. They are usually at full strength when they are located between 20 and 30 degrees latitude, but can travel to higher latitudes if there are warm ocean currents to sustain them.

A developed tropical cyclone has a three-dimensional vortex structure. The horizontal dimensions of these storms are less than those of extra-tropical cyclones, or depressions, but their effects can extend for several hundred kilometres. The circulation flows with a radial component towards the 'eye', outside of which is a region of intense thermal convection, with air currents spiralling upwards. Inside the eye is a region of relative calm with slowly sinking air; the diameter of the eye can range between 8 and 80 kilometres. Often clear skies have been observed in this region. The strongest winds occur just outside the eye wall.

The tracks of tropical cyclones are usually quite random and difficult to predict. However, there is a general tendency for them to move away from the Equator and towards the west; when they reach higher latitudes there is a trend to 'recurve' towards the east. The occurrence of tropical cyclones has only been monitored fully since about 1980, with the introduction of geostationary satellites.

The full effects of tropical cyclones is primarily felt within about 50 kilometres from the coastline, with rapidly reducing effects further inland as the storm weakens. However, occasionally significant structural damage may be sustained further inland, as occurred in parts of Orlando during Hurricane 'Charley' in 2003.

VERTICAL PROFILES OF WIND SPEEDS

Tower Measurements. Although there have been quite a few measurements of vertical wind profiles (mean and peak gust) from towers in hurricanes and tropical cyclones, in many cases they have been from sites away from the main path of the storm track, or have been obtained during weak events. Of course, for structural design at ultimate limit states, we would like to have data at the peak strength of substantial tropical cyclones.

Tower measurements of vertical wind profiles in tropical cyclones were made by Wilson (1979a, 1979b) at the North-West Cape near Exmouth in Western Australia. Observations were made from anemometers mounted on guyed tower at heights of 60m, 191m, 279m and 390m (with anemometers operating at all heights only during one cyclone). Another anemometer was mounted at 9m height on a pole about 350m away from the main tower. The fetch was open water with only a short land fetch of less than 5 kilometres, for a large range of wind directions from NW through N to S.

Several gust wind 'profiles' were recorded during the passage of four tropical cyclones at the North-West Cape in the late 1970s; this data was collected by Wilson (1979a, 1979b). These data are reproduced in Figure 1 and compared with the gust profile (i.e. square root of K_z) derived from ASCE 7-05 (Table 6-3) for Exposure C. The latter is generally conservative with respect to the average of these measurements, above a height of 100 metres (330 feet).

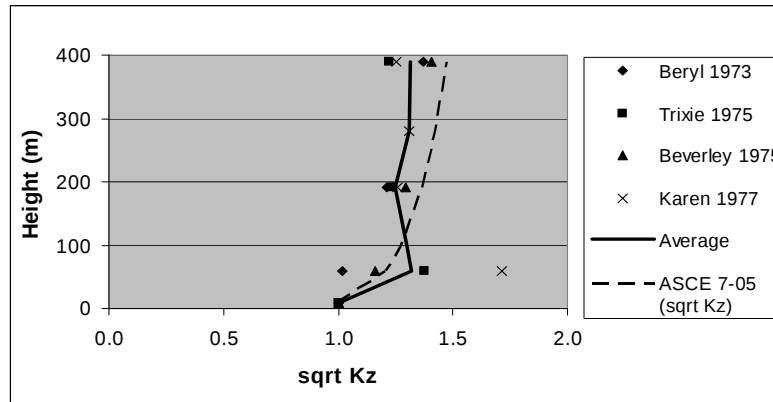


Figure 1. Comparison of measured maximum gust profiles for four tropical cyclones at North-West Cape, Western Australia, and K_z from ASCE 7-05

Dropwindsonde Profiles. Since 1997 the National Oceanic and Atmospheric Administration (NOAA) and the U.S. Air Force have been deploying GPS-based dropwindsondes into hurricanes in the Atlantic and eastern North Pacific oceans. The dropwindsonde is a probe containing sensors and a GPS satellite receiver that enables profiles of various atmospheric variables, including wind speed, to be monitored as it falls after being dropped from an aircraft. These have generated vertical profiles of wind and thermodynamic parameters from flight level (typically about 3000 metres) down to sea level. It should be noted that dropwindsondes are not normally deployed over land.

Detailed analyses of wind profiles from 1997-1999 have been described by Franklin *et al.* (2003); the main purpose was to obtain a ratio between surface (10 metres) and flight level wind speeds for forecasting purposes. Wind profiles were averaged separately for the eyewalls of hurricanes, and for the outer vortex regions. Powell *et al.* (2003) analysed similar data and fitted logarithmic profiles for various speed ranges.

From an engineering design point of view, the analyses of Powell *et al.* and Franklin *et al.* have produced the following significant results:

- A logarithmic law for the mean wind speed appears to hold in the eyewall region for heights above the surface up to about 300 metres. Above that height, the rate of increase is much smaller with a maximum mean velocity at about 500 metres. Above 500 metres, the velocity falls significantly in the eyewall region.
- For mean velocities at 10 metres height above water surface of about 33 m/s, the aerodynamic roughness length (an indicator of the surface roughness, and of the ratio of wind speed at upper levels to that near the surface) reaches a maximum value of about 3 mm, falling to 1 mm for velocities of 50 m/s (Powell *et al.* 2003).

Franklin *et al.* (2003) gives velocity ratios, $\bar{U}_z / \bar{U}_{10}$ for the eyewall region. These values are reproduced in Table 1, and compared with a logarithmic law with roughness length, z_0 , equal to 0.0001 m (0.1 mm). This value of roughness length is significantly lower than those currently used in most codes and standards for design of buildings (or the equivalent power law exponent), for off-water winds from hurricanes or tropical cyclones.

Since dropwindsondes are only deployed over the ocean, there are no profiles available for wind over land from hurricanes, as required for the design of most structures. It also should be noted that the dropwindsonde profiles do not give any useful information on turbulence intensities, and hence on peak gust envelope profiles as used in many design codes or standards, such as ASCE 7. However, it would not be expected that the latter would differ significantly from the mean velocity profiles, (and may have an even lower slope.)

TABLE 1
MEAN VELOCITY PROFILE IN THE HURRICANE EYEWALL FROM FRANKLIN *et al* (2003)
(COMPARED WITH A LOGARITHMIC LAW WITH $Z_0 = 0.1$ MM)

Height (m)	$\bar{U}_z / \bar{U}_{10}$	$\log_e(z/0.0001) / \log_e(10/0.0001)$
10	1.000	1.000
15	1.027	1.035
20	1.048	1.060
30	1.081	1.095
50	1.128	1.140
75	1.169	1.175
100	1.198	1.200
150	1.229	1.235
200	1.261	1.260
250	1.288	1.280
300	1.305	1.295

Profiles in Wind Codes and Standards. In ASCE 7-05, (American Society of Civil Engineers, 2006), the same wind profiles, based on a power law, are used for the hurricane-affected regions of the Caribbean and Atlantic coastlines, as are used for other parts of the country. Up to now the results of the dropwindsonde measurements have not been incorporated into ASCE 7.

STRUCTURAL SAFETY AND RELIABILITY

The assessment of wind loads is one part of the total structural design process, which also includes the determination of other loads and the resistance of structural materials. From the nineteen-seventies a coherent probabilistic framework for structural safety and reliability has developed, leading to major changes in the formats of design codes and standards in many countries.

Limit States Design. The probabilistic approach is linked closely to the concepts and formats of 'limit states design'. As well as explicitly defining the ultimate and serviceability limit states for design, limit states design takes a more rational approach to structural safety by defining 'partial' load factors for each type of loading, and a separate resistance factor for the resistance.

A typical ultimate limit states design relationship involving wind loads and dead (i.e. permanent gravity) loads, is as follows.

$$\phi R \geq \gamma_D D + \gamma_W W \quad (1)$$

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
 W is the nominal wind load

The partial factors, ϕ , γ_D , and γ_W are determined using probabilistic methods 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.

Safety Index. A quantitative measure of the safety of structures, known as the *safety index*, or *reliability index*, is generally used as a method of calibration of existing and future design methods for structures. There is a one-to-one relationship between this index and a notional probability of failure, based on the exceedence of a design resistance by an applied load (but not including the effects of human errors and ‘accidental loads’).

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

$$\beta = -\Phi^{-1}(p_f) \quad (2)$$

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 probability of failure and the safety index is shown in Figure 2.

Ellingwood *et al.* (1980) introduced probability based design criteria based on the above concepts for the American structural standards and codes. Pham *et al.* (1983) discussed the application of the structural reliability approach to wind loading in Australia – including locations at which extreme winds from tropical cyclones were dominant. Estimates of the mean to nominal values and coefficients of variation of the various parameters involved in wind loading estimation were made, and appropriate probability distributions were assumed. These are summarized (for tropical cyclone regions) in Table 2.

It can be seen from Table 2 that the greatest contributor to the variability and uncertainty in wind load estimation is the wind speed itself - particularly as it is raised to a power of two (or greater, when dynamic effects are important) when wind loads and effects are calculated. A secondary contributor is the uncertainty in the exposure parameter in Table 2, which is also squared, and includes uncertainties in the vertical profile of mean and gust speeds as discussed in earlier sections of this paper.

Based on the assumptions in Table 2, Pham *et al.* calculated the notional probability of failure, and hence from Equation (1) the safety index, β , for designs based on the then-current Australian Standards, and for proposed new limit states versions. It was shown that an acceptable safety index, with a value of β of around 3, was obtained (for quasi-static structures)

by the use of a high return period nominal design wind speed, together with a wind load factor, γ_w , of 1.0 in Equation (1) in both cyclonic and non-cyclonic regions. The magnitude of safety index was consistent with earlier 'working stress' code formats based on a 50-year return period wind speeds.

For structures, such as tall buildings, wind loads and effects vary with wind speed raised to a power somewhat greater than 2. Holmes and Pham (1993) considered the effect of the varying exponent on the safety index (see Equation (2)) and showed that use of the traditional approach of 'working stress' wind loads based on a 50-year return period wind speed, with a fixed wind load factor of 1.5, resulted in a significant reduction in safety as the exponent increased. However, a near-uniform safety (insensitive to the effects of dynamic response) was achieved by the use of a high-return period nominal design wind speed together with a wind load factor of 1.0.

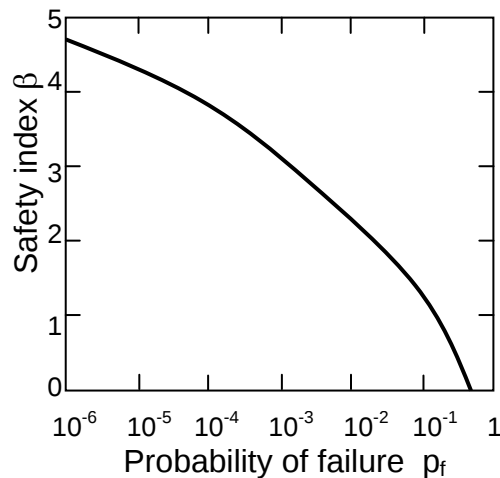


Figure 2. Relationship between safety index and probability of failure

TABLE 2.
STATISTICAL VARIABILITY OF WIND LOADING PARAMETERS FOR TROPICAL CYCLONE REGIONS
(from Pham *et al* (1983))

PARAMETER	MEAN/NOMINAL	COEFFT. OF VARIATION	ASSUMED DISTRIBUTION
Wind speed (50 year maximum)	1.12	0.28	Gumbel
Directionality	0.9	0.05	Lognormal
Exposure	0.8	0.15	Lognormal
Pressure coefficient	0.8	0.15	Lognormal
Local & area reduction effects	0.85	0.10	Lognormal

WIND SPEED PROBABILITY DISTRIBUTIONS AND LOAD FACTORS

Extreme Value Probability Distributions. The determination of design wind speed at some standard reference point (such as 10 metres in flat open terrain) is a crucial step in determination of design wind loads, and for tropical cyclones the greatest source of uncertainty. The intermittent occurrence of tropical cyclones at many locations can make prediction of extreme wind speeds based on recorded historical values from anemometers statistically unreliable. However, at other locations, tropical cyclones occur relatively frequently, allowing acceptable predictions from recorded data to be made. The latter group includes locations on the North-west coast of Australia (such as Port Hedland) and Hong Kong.

An alternative approach involves using historical satellite data on tropical cyclone tracks, forward speed, strength (e.g. central pressure) and size to simulate many more storms of similar characteristics. Together with a suitable wind field model, reasonable predictions of local wind speeds at a site can be made. The most developed approach of this type has been used for hazard modelling in the United States, and is described by Vickery *et al.* (1995, 2000).

A hybrid approach that has been used in Australia, involves ‘filling in’ missing historical data for tropical cyclones during which anemometers that have failed to record the maximum gusts.

Whether fitted to recorded or simulated data, extreme wind speeds in tropical cyclones are usually fitted with one of the family of Generalized Extreme Value Distributions (GEV). The general form of this distribution is as follows (e.g. Holmes, 2007).

$$F_U(U) = \exp \{ -[1 - k(U-u)/a]^{1/k} \} \quad (3)$$

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

In Equation (3), k is a shape factor, a is a scale factor, and u is a location parameter. 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 (3) becomes Equation (4) in the limit. Equation (4) defines the *Type I Extreme Value*, or *Gumbel, Distribution*.

$$F_U(U) = \exp \{ -\exp [-(U-u)/a] \} \quad (4)$$

The Type II E.V. Distribution gives values of wind speed which increase with increasing return period with increasing slope. A version of this distribution was used in earlier predictions of wind speeds (e.g. Thom, 1967), but this may have arisen from failing to separate extreme winds caused by hurricanes from those produced by other wind types.

The Type III E.V. Distribution, with a positive value of shape factor, k , has an upper limit of the predicted wind speed (equal to $u + a/k$). Physically, this might be related to the thermodynamic upper limits on the strength of tropical cyclones suggested by a number of authors (e.g. Holland, 1997; Emanuel, 1999). The Type III E.V. distribution is currently used

in Australia for extrapolation of extreme wind speeds in both cyclonic and non-cyclonic regions. It has also been proposed for use in the United States to give a better estimate of risk in hurricane regions (Heckert *et al.*, 1998).

The Type I, or Gumbel, Distribution also gives unlimited wind speeds, but has been widely used for extreme value prediction of wind speeds.

Wind Load Factors. Given that the risk of exceedence of a wind speed with 50-year or 100-year return period is quite high in a typical building lifetime, the main purpose of wind load factors in Equation (1) is to reduce the risk of the factored wind load being exceeded in the building lifetime.

The following equation gives the risk, r , of exceedence of the R -year wind speed being exceeded in a lifetime of L years.

$$r = 1 - \left[1 - \left(\frac{1}{R} \right) \right]^L \quad (5)$$

For example, substituting $R=L=50$ into Equation (5) shows that a 50-year wind has about a 64% risk of exceedence in a 50-year building life. This risk is unacceptable, and consequently most codes and standards, that use the 50-year return period wind speed (denoted by V_{50}) as the characteristic wind speed for design, also apply a wind load factor of 1.4 to 1.6 to the nominal wind loads calculated from it.

However, wind load factors in this range have been derived on the basis of two assumptions:

- a) Wind loads vary as the square of wind speeds. This is valid for small structures of high frequency with high natural frequencies. However this assumption is not valid for tall buildings for which the effective wind loads vary to a higher power of wind speed, due to the effect of resonant dynamic response. This power is up to 2.5 for along-wind response, and up to 3 for cross-wind response.
- b) The wind load factors have been derived for non-tropical-cyclone wind loads, i.e. climates for which the rate of change of wind speed with return period R is relatively low, compared with those in regions affected by tropical cyclones, typhoons or hurricanes.

The last point leads to a requirement for a higher wind load factor for building design in locations affected by tropical cyclones than for non-cyclonic locations. For example, in Australia between 1971 and 1989, this problem was solved by the requirement that a 'Cyclone Factor' of 1.15 was applied to the nominal 50-year return period wind speed. For non-dynamic structures, this was equivalent to an extra load factor of 1.32 (i.e. 1.15^2). A similar 'Hurricane Importance Factor', with a value of 1.05, appeared in some editions of the American Loading Standard, ASCE 7 (up to 1995), but was later incorporated into the specified basic wind speed (ASCE, 2006).

The need for higher load factors in locations affected by tropical cyclones is illustrated by Table 3. This shows the equivalent return period of wind speeds when a wind load factor of 1.4 is applied to the 50-year return period value for several different locations. For two locations in the United Kingdom, the application of the 1.4 factor is equivalent to calculating wind loads from wind speeds with return periods considerably greater than 1000 years. However, in Penang, Malaysia, where design wind speeds are governed by winds from relatively infrequent severe thunderstorms, the factor of 1.4 is equivalent to using a wind speed of about 500 years return period.

At Port Hedland (Western Australia), and Hong Kong, where tropical cyclones and typhoons are dominant, and extreme wind events are even rarer, the factored wind load based on V_{50} is equivalent to applying wind loads of only 150 and 220 years return period, respectively. In Australia, early editions of AS1170.2 adjusted the effective return period by use of the 'Cyclone Factor' as discussed above. However, in AS/NZS1170:2002, a nominal design wind speed with 500 years return period (80 m/s basic gust wind speed at Port Hedland) with a wind load factor of 1.0, is used for design of most structures for ultimate limit states. An 'uncertainty' factor of 1.10 is also applied to the wind speeds in that part of Australia, further increasing the effective return period based on recorded data.

A similar approach to AS/NZS1170.2 will be adopted in the 2010 edition of ASCE 7, in which design wind speeds with return periods of 300, 700 or 1700 years will be used (depending on the importance of the structure), together with a wind load factor of 1.0 (Irwin, 2009).

TABLE 3.
RETURN PERIOD OF FACTORED WIND LOADS WITH A WIND-LOAD FACTOR OF 1.4

LOCATION	WIND TYPE	V_{50} (m/s)	$\sqrt{1.4 V_{50}^2}$ (m/s)	RETURN PERIOD OF FACTORED WIND SPEED (years)
Cardington (U.K.)	Atlantic gales	42.9	50.7	1130
Jersey (U.K.)	Atlantic gales	45.6	53.9	1470
Penang (Malaysia)	Thunderstorms	27.6	32.7	510
Port Hedland (W.A.)	Tropical cyclones	60.5	71.6	150
Hong Kong	Typhoons	61.5	72.8	220

Notes: i) Values of wind speed shown are gust speeds at 10 metres height over flat open terrain, except for Hong Kong for which the gust speed is adjusted to 50m height above the ocean.

ii) Probability distributions for Cardington and Jersey obtained from Cook (1985).

iii) The predicted wind speeds for Hong Kong were obtained by the author from typhoon data recorded at Waglan Island from 1953 to 2006, using a 'peaks over threshold approach' and a Type III Extreme Value Distribution.

GLOBAL WARMING AND CLIMATE CHANGE

Finally some comments on possible effects of global warming on the numbers and strength of tropical cyclones worldwide are appropriate. There have been a number of studies of this in recent years (e.g. Webster *et al.* 2005; Emanuel, 2005; Klotzbach, 2006; Kossin *et al.*, 2007).

Since it is well established that a sea surface temperature of 26°C is required for tropical cyclone formation in the current climate, it might be expected that there would be an increase in the number of tropical cyclones worldwide with increasing average sea temperatures. In fact, Webster *et al.* (2005) found there was no significant trend in global cyclones of all strengths. However they did show a statistically significant increasing trend in Category 4 to 5 storms from the 1970s to the decade 1995 to 2004 (see Figure 3). These mainly seem to have occurred in the North Atlantic basin.

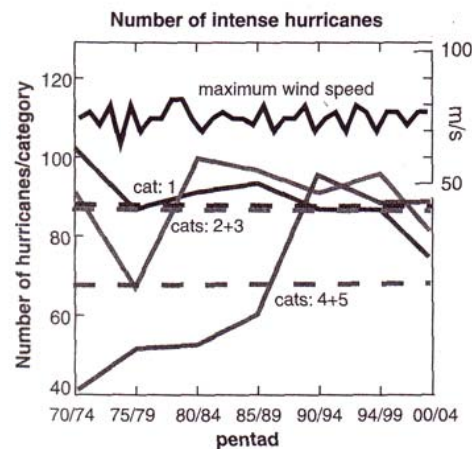


Figure 3. Apparent increase in number of Category 4 and 5 hurricanes world-wide (from Webster *et al.* (2005))

Klotzbach (2006) extended the analysis to all basins with tropical cyclone activity, and excluded data before 1986 on the basis that, before the mid 1980s, only *visible* satellite information was available and hence night-time observations were excluded; also the quality and resolution of satellite imagery had improved greatly by the later period. Klotzbach's analysis, using the more recent (and more reliable) data, found only a small increase in Category 4-5 hurricanes in the North Atlantic and Northwest Pacific during the 20-year study period.

However, the last three decades have seen few cyclones make landfall on the Queensland coast of Australia compared to earlier years. This is in contrast to the North Atlantic and Caribbean where the 2004 to 2008 period saw a large number of hurricane occurrences; rightly or wrongly, these have often been attributed to the effects of global warming.

The following summary statement in the U.N. International Panel on Climate Change Report of 2007 is relevant:

'There is observational evidence for an increase in intense tropical cyclone activity in the North Atlantic since about 1970, correlated with increases in sea surface temperatures. There are also suggestions of increased intense tropical cyclone activity in some other regions where concerns

over data quality are greater. ... There is no clear trend in the annual numbers of tropical cyclones.'

These studies and predicted trends are currently under review by building codes and standards bodies in Australia and the United States. Clearly there is a great deal of uncertainty in the predictions, and regulators may be reluctant to impose additional economic costs of higher design wind loads on the community without more confidence in the trends. A rational approach to design in these circumstances might suggest a small increase in load factor to cover these uncertainties.

CONCLUSIONS

The following conclusions have resulted from this review of design for wind loads in regions affected by tropical cyclones, hurricanes and typhoons.

- i) There is still uncertainty in the profiles of mean and gust wind speeds in the high-wind eye-wall region of tropical cyclones (the region of interest for structural design). Some valuable data has recently been obtained by dropwindsondes in the North Atlantic, but so far the profiles obtained are limited to ensemble averages over the ocean, with no information on turbulence or simultaneous gust profiles over land.
- ii) Studies of all the uncertainties in the structural design process indicate that the major uncertainty in the design process, in regions affected by tropical cyclones or typhoons, is the wind speed itself (at some defined reference height). Hence, the level of safety and reliability of structures whose design is dominated by wind loads is largely determined by the nominal return period of the design wind speed and the wind load factor that is applied in the limit states design equation (e.g. Equation (1)).
- iii) The intermittent nature of impact of severe tropical cyclones and hurricanes results in a rate of change between wind speed and return period (or annual probability of exceedence) which is significantly greater than in higher-latitude temperate climates (such as those occurring in Canada or Western Europe).
- iv) Structures with significant resonant response to wind, such as tall buildings, experience a greater increase in response with increasing wind speed, than do 'static' structures such as low-rise buildings, which have structural responses which vary as wind speed squared.
- v) (iii) and (iv) above lead to the conclusion that use of wind load factors derived for static structures in non-tropical cyclone climates with low-return period nominal wind speeds, will result in lower safety for tall structures in tropical cyclone or hurricane situations. Therefore, significantly higher wind load factors should be considered in these circumstances. An alternative approach is to adopt a high return period nominal wind speed with a lower wind load factor (such as 1.0). In the latter case the safety level is insensitive both to the windstorm type and to the occurrence of dynamic response.

vi) The effects of global warming on the number and strength of tropical cyclones due to global warming are quite uncertain, with some contradictory trends appearing. A rational approach to design in these circumstances would suggest a small increase in wind load factor to cover these uncertainties.

ACKNOWLEDGEMENTS

Useful discussions of design wind speeds and wind load factors with David P. Thompson of KTA Structural Engineers of Alberta, Canada, are acknowledged by the author.

REFERENCES

American Society of Civil Engineers (2006). *Minimum design loads for buildings and other structures*, ASCE Standard ASCE/SEI 7-05.

Ellingwood, B., Galambos, T.V., McGregor, J.G. and Cornell, C.A. (1980). Development of a probability based load criterion for American National Standard A58. National Bureau of Standards Special Publication 577.

Emanuel, K.A., (1999). Thermodynamic control of hurricane intensity. *Nature*, 401, 665-669.

Emanuel, K.A. (2005). Increasing destructiveness of tropical cyclones over the past thirty years. *Nature*, 436, 686-688.

Cook, N.J. (1985). *The Designer's Guide to Wind Loading of Building Structures*. BRE-Butterworths, U.K.

Franklin, J.L., Black M.L. and Valde K. (2003). GPS dropwindsonde wind profiles in hurricanes and their operational implications. *Weather and Forecasting*, 18, 32-34.

Heckert, N., Simiu, E. and Whalen, T. (1998). Estimates of hurricane wind speeds by peaks over threshold method. *J. Struct.Engg. (ASCE)*, 124, 445-449.

Holland, G.J., (1997). The maximum potential intensity of tropical cyclones. *Journal of Atmospheric Sciences*, 54, 2519-2541.

Holmes, J.D. (2007). *Wind loading of structures*. 2nd Edition. Taylor and Francis London, U.K.

Holmes, J.D. and Pham, L. (1993). Wind-induced dynamic response and the safety index. *6th International Conference on Structural Safety and Reliability*, Innsbruck, August 9-13, Proceedings, pp1707-1709, A.A. Balkema, Publishers.

Irwin, P.A. (2009). Wind engineering research needs, building codes and project-specific studies. *11th Americas Conference on Wind Engineering*, San Juan, Puerto Rico, June 22-26, 2009.

Klotzbach, P.J. (2006). Trends in global tropical cyclone activity in the last twenty years (1986-2005). *Geophysical Research Letters*, 33, L10805

Kossin, J.P., Knapp, K.R., Vimont, D.J., Murnane, R.J. and Harper B.A. (2007). A globally consistent reanalysis of hurricane variability and trends. *Geophysical Research Letters*, 34, L04815.

Pham, L., Holmes, J.D. and Leicester, R.H. (1983). Safety indices for wind loading in Australia. *Journal of Wind Engineering and Industrial Aerodynamics*, 14, 3-14.

Powell, M.D., Vickery P.J. and Reinhold T.A. (2003). Reduced drag coefficient for high wind speeds in tropical cyclones. *Nature*, 422, 279-283.

Standards Australia (2002). *Structural design actions. Part 2: Wind actions*. Australian/New Zealand Standard AS/NZS1170.2:2002.

Thom, H.C.S. (1967). Towards a universal climatological extreme wind distribution. *International Research Seminar on Wind effects on Buildings and Structures*, Ottawa, Canada, September 11-15, 1967, Proceedings pp 669-683, University of Toronto Press, 1968.

Vickery, P.J. and Twisdale, L.A. (1995). Prediction of hurricane wind speeds in the United States. *J. Struct.Engg. (ASCE)*, 121, 1691-1699.

Vickery, P.J., Skerlj, P.F. and Twisdale L.A. (2000). Simulation of hurricane risk in the U.S. using empirical track model. *J. Struct.Engg. (ASCE)*, 126, 1222-1227.

Webster, P.J., Holland, G.J., Curry J.A. and Chang H.R. (2005). Changes in tropical cyclone number, duration and intensity in a warming environment. *Science*, 309, 1844-1846.

Wilson, K.J. (1979a). Wind observations from an instrumented tower during Tropical Cyclone Karen, 1977. *12th Technical Conference on Hurricanes and Tropical Meteorology*, New Orleans, Louisiana, April 1979.

Wilson, K.J. (1979b). Characteristics of the subcloud layer wind structure in tropical cyclones. *International Conference on Tropical Cyclones*, Perth, Western Australia, November 1979.