DEVELOPMENTS IN CODIFICATION OF WIND LOADS IN THE ASIA PACIFIC

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ABSTRACT

In this paper, an overview of codified wind loads in the Asia Pacific region is given. A description of the various codes and standards in the region is given including the historical origins. Recent APEC-WW meetings and the calculations for three example buildings are mentioned. The importance of wind load factors is discussed. It is clear that there is currently little consistency between neighbouring economies with many places having codes and standards based on documents from elsewhere in the world. Concepts for performance-based wind engineering are suggested. A strategy for improving alignment of codes and standards in the future is outlined.

KEYWORDS: ASIA-PACIFIC, CODES, STANDARDS, WIND LOADS

Introduction

For the purposes of this paper, I have defined the ‘Asia-Pacific’ as that part of the world bounded by the 75° East and 180° East lines of longitude. This essentially incorporates countries bordering the western side of the Pacific Ocean and the eastern parts of the Indian Ocean, and is the generally accepted ‘catchment’ of the Asia-Pacific Conference on Wind Engineering. However, it should be noted that the APEC (Asia-Pacific Economic Cooperation) group includes, in addition, countries bordering the eastern rim of the Pacific Ocean.

The region incorporates a wide range of meteorological influences with some areas affected by tropical cyclones (typhoons) with very high design wind speeds. At the other extreme, most equatorial locations suffer only from relatively benign monsoons and local thunderstorm downdrafts and have low design wind speeds. Previous papers and reports (Holmes and Melbourne, 1997; Holmes and Weller, 2002) have attempted to zone the Asia-Pacific region with respect to extreme wind speeds for design. These will not be discussed in detail in this paper.

Regional wind codes and standards

Table 1 summarizes the major wind loading codes and standards in the Asia-Pacific region as at the time of writing. It is clear that there is a mixture of basic wind averaging times and return periods – i.e. similar variability as exists across the world as a whole. For various historical colonial and/or cultural connections, many of the documents are derived from other documents used elsewhere in the world – not only from the Asia-Pacific, but also from Europe and North America.
The averaging time for the design wind speed is either 3-seconds, 10-minutes or 1-hour. Although there are good reasons for using one or the other, such as the form of available meteorological data, and the type of extreme wind event(s) (i.e. typhoon, thunderstorm downdraft, extra-tropical gales) that are dominant for the particular economy, in some cases the choice of averaging period in Table 1 is not appropriate. In the author’s opinion the translation of gust data into hourly means in tropical climates using ‘standard’ gust factors from temperate climate models, with little or no relation to actual hourly values in the country in question, are misleading and confusing, especially for those charged with producing more rational codes and standards into the future. (Note: However, this practice may be more justifiable for wind-tunnel operators who require a stable long-duration mean wind speed.)

Table 1: Characteristics of regional codes and standards

<table>
<thead>
<tr>
<th>Economy</th>
<th>Designation</th>
<th>Averaging time</th>
<th>Return Period</th>
<th>Complexity</th>
<th>Origin</th>
</tr>
</thead>
<tbody>
<tr>
<td>Australia/New Zealand</td>
<td>AS/NZS1170.2: 2002</td>
<td>3-second</td>
<td>500-1000 years</td>
<td>high</td>
<td>AS (1971-date)</td>
</tr>
<tr>
<td>China</td>
<td>GB50009-2001</td>
<td>10-minutes</td>
<td>50 years</td>
<td>high</td>
<td>China/ISO</td>
</tr>
<tr>
<td>Hong Kong</td>
<td>CP-2004</td>
<td>3-second/1-hour</td>
<td>50 years (150 years)*</td>
<td>medium</td>
<td>HK</td>
</tr>
<tr>
<td>India</td>
<td>IS875 (Part 3)-1987</td>
<td>3-second</td>
<td>50 years</td>
<td>high</td>
<td>AS/BS</td>
</tr>
<tr>
<td>Indonesia</td>
<td>SNI-03-1727</td>
<td>3-second</td>
<td>not defined</td>
<td>low</td>
<td>?</td>
</tr>
<tr>
<td>Japan</td>
<td>AIJ-RLB-2004</td>
<td>10-minutes</td>
<td>100 years</td>
<td>high</td>
<td>AIJ (1994-)</td>
</tr>
<tr>
<td>Korea</td>
<td>KBC (2005)</td>
<td>10-minutes</td>
<td>100 years</td>
<td>high</td>
<td>AIK/AIJ</td>
</tr>
<tr>
<td>Malaysia</td>
<td>MS1553-2002</td>
<td>3-second</td>
<td>50 years</td>
<td>high</td>
<td>AS/NZS</td>
</tr>
<tr>
<td>Philippines</td>
<td>NSCP-2001</td>
<td>3-second</td>
<td>50 years</td>
<td>medium</td>
<td>ASCE-7-95</td>
</tr>
<tr>
<td>Taiwan</td>
<td>TBC (2005)</td>
<td>10-minutes</td>
<td>50 years</td>
<td>high</td>
<td>ASCE-7-02/AIJ</td>
</tr>
<tr>
<td>Thailand</td>
<td>EIT-1018-46 (2003)</td>
<td>1-hour</td>
<td>10-100 years</td>
<td>medium</td>
<td>NBCC (Canada)</td>
</tr>
<tr>
<td>Vietnam</td>
<td>TCVN2737-2005</td>
<td>10-minutes</td>
<td>50 years</td>
<td>high</td>
<td>VN (Russia ?)</td>
</tr>
</tbody>
</table>

* Based on recent analyses, the actual return period of the basic wind speeds used in Hong Kong may be closer to 150 years (e.g. Holmes et al, 2009)

APEC-WW meetings and example buildings

With the support of the Centre of Excellence in Wind Engineering at Tokyo Polytechnic University, four Workshops on Regional Harmonization of Wind Loading and Wind Environmental Specifications (APEC-WW) have been held between 2004 and 2007. These are summarized in Table 2.

At the Hong Kong Workshop (APEC-WW2), three example buildings were defined representing typical low-rise, medium rise and high-rise buildings. At that, and the subsequent two Workshops, wind load calculations, using their own codes and standards were requested from representatives of all economies participating. The results were summarized by Holmes, Tamura and Krishna (2008).
Some of the coefficients of variation for various loads or responses are summarized in Table 3. It may be surprising that the coefficients of variation of the responses of the high-rise building, which are more complex to calculate, are generally lower than those for the other buildings. However, this probably resulted from the origin of the methods of calculation from only three or four original sources.

### Table 3: APEC-WW Buildings - Coefficients of variation

<table>
<thead>
<tr>
<th>Building</th>
<th>parameter</th>
<th>Coefft. Of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low rise</td>
<td>Door and window loads</td>
<td>13-26%</td>
</tr>
<tr>
<td>Medium rise</td>
<td>Base shear &amp; bending moment</td>
<td>22%</td>
</tr>
<tr>
<td>Medium rise</td>
<td>Cladding load</td>
<td>22-23%</td>
</tr>
<tr>
<td>High rise</td>
<td>Base shear &amp; bending moment</td>
<td>14-17%</td>
</tr>
<tr>
<td>High rise</td>
<td>Top accelerations</td>
<td>17-18%</td>
</tr>
</tbody>
</table>

It should be noted that the variations shown in Table 3 are primarily those associated with shape factors, pressure and force coefficients, dynamic response factors etc. This is because the wind speeds and turbulence intensities at the top of each building were pre-specified in the comparisons. The variability in wind models for various terrains would introduce further variabilities in the calculated wind loads, as would the effects of topography, if examples were used in which those effects were important.

**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.

\[ \phi R \geq \gamma_D D + \gamma_W W \]  

where

\( \phi \) is a resistance factor  
\( R \) is the nominal structural resistance  
\( \gamma_D \) is the dead load factor  
\( D \) is the nominal dead load  
\( \gamma_W \) is the wind load factor  
\( W \) is the nominal wind load

The partial factors, \( \phi \), \( \gamma_D \), and \( \gamma_W \) 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.

\section*{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 Eq. (2), and normally takes values in the range 2 to 5.

\[ \beta = - \Phi^{-1} (p_t) \]  

where \( \Phi^{-1}(\cdot) \) 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 1.

![Figure 1: Relationship between safety index and probability of failure](image_url)
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 4.

It can be seen from Table 4 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 4, 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 4, Pham et al. calculated the notional probability of failure, and hence from Eq. (1) the safety index, \( \beta \), 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 \( \beta \) 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, \( \gamma_W \), of 1.0 in Eq. (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.

Table 4: Statistical variability of wind loading parameters (from Pham et al., 1983)

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>MEAN/NOMINAL</th>
<th>COEFFT. OF VARIATION</th>
<th>ASSUMED DISTRIBUTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind speed (50 year maximum)</td>
<td>1.12</td>
<td>0.28</td>
<td>Extreme value</td>
</tr>
<tr>
<td>(cyclonic)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wind speed (50 year maximum)</td>
<td>1.05</td>
<td>0.12</td>
<td>Extreme value</td>
</tr>
<tr>
<td>(non-cyclonic)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Directionality</td>
<td>0.9</td>
<td>0.05</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Exposure</td>
<td>0.8</td>
<td>0.15</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Pressure coefficient</td>
<td>0.8</td>
<td>0.15</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Local &amp; area reduction effects</td>
<td>0.85</td>
<td>0.10</td>
<td>Lognormal</td>
</tr>
</tbody>
</table>

Although the values in Table 4 were specified for Australian conditions, the values may well have some relevance for other countries in the Asia-Pacific.

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 Eq. (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.

Nominal design values and wind load factors are discussed further in the following sections.

**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 Eq. (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.

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

(3)

For example, substituting $R=L=50$ into Eq. (3) 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 5. 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.

Table 5: Return period of factored wind loads with a wind load factor of 1.4

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

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).

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, previous 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.

Future editions of the American Standard ASCE 7 will use a basic wind speed with a 700-year return period coupled with a wind load factor of 1.0 for all parts of that country (Irwin, 2009) – thus adopting a similar method to that used in Australia and New Zealand since 1989 and 1992, respectively.

Performance-based design in wind engineering

Performance-based design is a concept that has been used in earthquake engineering for at least twenty years. However, the concept has yet to be applied in wind engineering – at least explicitly. A tentative list of design states for design of tall buildings against wind loads is shown in Table 6. Progressing down the Table, wind loads of decreasing frequency and increasing magnitude are specified. Each risk /frequency level is accompanied by a
required performance level for a specified parameter. In the case of tall buildings, with significant dynamic response, building dynamic properties such as damping may also change as the building state changes. Representative, but not definitive, values of damping are listed.

The first three states in Table 6 are currently considered in design - primarily in project-specific wind-tunnel tests but also when design to advanced wind codes is carried out. The fourth state is a tentative one in which the specified wind loads are very high and unlikely, but the building is only required to remain standing and not collapse under these loads. Such states are an established feature of performance-based earthquake engineering.

Table 6: Tentative design states in performance-based design

<table>
<thead>
<tr>
<th>Risk</th>
<th>Return period of wind speed (years)</th>
<th>Criteria/parameters</th>
<th>Performance</th>
<th>Damping (% of critical)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2 to 1 per year</td>
<td>1-5</td>
<td>Acceleration (serviceability)</td>
<td>15-20 mg (depends on frequency)</td>
<td>1</td>
</tr>
<tr>
<td>0.04 to 0.05 per year</td>
<td>20-25</td>
<td>Deflection (serviceability)</td>
<td>1:500</td>
<td>1-1.5</td>
</tr>
<tr>
<td>0.05 to 0.1 per 50 years</td>
<td>500 -1000</td>
<td>Strength (ultimate)</td>
<td>No structural damage</td>
<td>3</td>
</tr>
<tr>
<td>0.01 per 50 years</td>
<td>5000 ?</td>
<td>Strength (ultimate)</td>
<td>No collapse</td>
<td>5</td>
</tr>
</tbody>
</table>

Discussion and Conclusions

Current wind codes and standards in the Asia-Pacific region show a variety of formats, averaging times for basic wind speeds, and nominal return periods for design. In some cases, there is little attempt to tailor the format to the type of wind event that is dominant for the country or region. There is a considerable scope for progress to be made in improving consistency across the region.

In APEC-WW Workshops held between 2004 and 2007, three example buildings were defined representing typical low-rise, medium-rise and high-rise buildings. Using these examples, wind load calculations using their own codes and standards, were requested from representatives of all the Asia-Pacific economies participating. These examples indicated coefficients of variation between 13 and 26% for the calculated wind loads. Surprisingly, the coefficients of variation were generally smaller for the high-rise building. Most users of these wind codes and standards would consider that this variability as too large, and efforts should be made in resolving the differences, perhaps starting with the ‘outliers’, and promoting more harmonization in wind codes and standards in the region.

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. Eq. (1)).

The intermittent nature of impact of severe tropical cyclones and typhoons 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.

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.

The comments 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 typhoon 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.

Some ideas for future alignments of wind codes and standards in the Asia-Pacific are:

* Simplification of terrain categories for surface roughness exposures into six or seven standardized ones (with accompanying velocity profiles and turbulence levels). From these the wind codes or standards would select appropriate ones for their own use (a working group under Professor Edmund Choi is currently reviewing this task).
* The differences arising from the example buildings (as summarized in Table 3) need to be resolved and the main causes of the differences, particularly the outliers, identified.
* A simplified regional model code/standard for small buildings could be developed (suggested by Professor Prem Krishna). This may be useful for countries where seismic forces are dominant, or for small countries (such as islands in the South Pacific) without the expertise or resources to develop their own documents. This could be based on the draft ISO Standard, or the Australian Standard on Wind Loads for Housing (AS4055).
* Development of an agreed regional map of 3-second, 10-minute and 1-hour extremes with 50- and 500-year return periods. This would resolve some existing step changes and boundaries.
* Consideration and incorporation of recommendations from the IAWE Codification working groups summarized in a recent Special Issue on Codification of Wind Loads in the journal *Wind and Structures* (Holmes *et al.*, 2005).

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