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Case Study: Ultimate Limit State Wind Loads for a Dynamically Sensitive Super Tall Tower

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Abstract

RWDI was engaged as the wind engineering consultant for a super-tall tower to be built in the equatorial region of SE Asia. This confidential project presented unique challenges arising from the structure's slenderness and the local meteorological records. Utilizing wind tunnel test data for the project a comparison is made between ultimate limit state design wind loads derived using "first-order second moment" reliability theory and those generated from a probabilistic study utilizing numerical statistical techniques to account for uncertainties associated with the ultimate wind speed estimate as well as the structure's natural frequencies and damping ratio. The results are compared with the predicted structural responses referencing deterministic estimates of ultimate limit state wind speed and dynamic properties.

Introduction

High rise building designs are becoming increasingly tall and slender. In recent years a significant number of supertall and megatall buildings, towers with height in excess of 300 m, have been built or are under construction. These structures present unique challenges due to their pronounced dynamic sensitivity to wind effects – especially cross-wind excitation arising from vortex shedding. RWDI recently was engaged as the wind engineering consultant for a super-tall tower to be built in the equatorial region of SE Asia. This confidential project presents unique challenges arising from the structure's slenderness and the quality of local meteorological records.

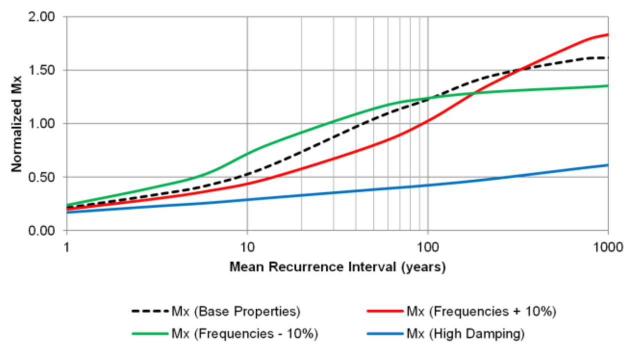

Figure 1a. The wind speed-base moment response curve for Mx is shown for variations of the tower's estimated natural frequencies (best estimate of fundamental Y sway = 0.089Hz): as well as a case assuming high damping (ζ = 10%). To ensure confidentiality of the project, base moments are presented normalized to their estimated values based on the analytical methodology of AS/NZS 1170.2.2011.

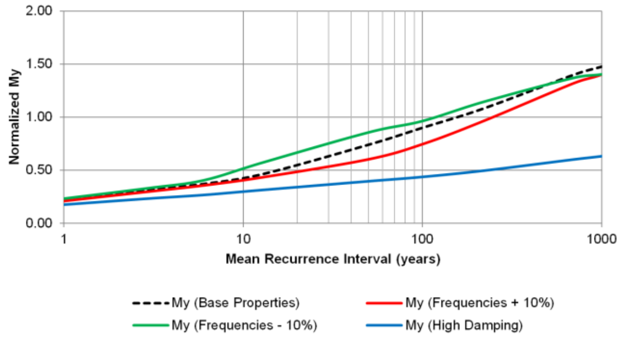

Figure 1b. The wind speed-base moment response curve for My. Fundamental X Sway = 0.077Hz

It is evident from Figures 1a and 1b that the tower is particularly dynamically sensitive – although the trends differ between Mx and My at the high mean recurrence intervals (MRI) associated with the ultimate limit state (ULS). Mx can be seen to have amplified sensitivity to changes in the dynamic properties. Furthermore stiffening the tower and increasing frequencies amplifies the loading in this direction.

The high dynamic sensitivity and differences in behaviour between Mx and My make this tower a particularly interesting case study to explore different approaches to derive reliable wind loads suitable for ULS design. This paper presents a comparison between an approach that was used to derive design loads for the project based on "first-order second moment" (FOSM) reliability theory, and a probabilistic study using numerical statistical techniques to account for the dominant sources of uncertainties.

Wind Load Factors for Dynamically Sensitive Structures

In the case of tall and slender buildings, wind tunnel testing is necessary as the analytical code-based approaches to predict extreme structural responses are not applicable. Typically responses are derived from analysis of wind tunnel data that has been scaled with reference to wind speeds from a wind climate analysis of local wind speed records or codified values. Wind load and importance factors are then applied by the structural engineer to achieve the building's intended reliability at the ULS.

Wind load factors in traditional building codes increase the loads based on a nominal lifetime of a building to ensure an appropriate level of safety against failure. These factors are based on the assumption that loading is proportional to the square of the wind velocity with limited dynamic amplification. They account for uncertainty associated with a number of other sources of random variability including wind speed (wind climate, gust and exposure factors) and pressure coefficients (Ellingwood and Tekie, 1999; Bartlett et al, 2003). However in the case of super-tall towers, the response often increases much more rapidly than the square of the wind velocity and the parameters of frequency and damping play a significant role. In light of the uncertainties associated with these parameters, and that the response is typically not proportional to the square of the wind velocity, increased wind load factors may be warranted for such structures.

The AS/NZS 1170.2.2011 and ASCE 7-10 are examples of modern building codes that define ULS winds loads based on a design wind speed associated with a high MRI and a load factor of unity. Such codes have the advantage of accounting for the wind speed-response relationship of dynamically sensitive structures. In the judgement of Allsop (2011) "reasonable safety" is provided when adopting this approach and utilizing best-estimate values of other design parameters; a perspective that is consistent with the view of many contemporary design practitioners. Chen and Huang (2009) developed a refined full-order method to estimate the extreme wind load effect considering uncertainty as a function two parameters: the annual maximum wind speed and the extreme load effect. They showed through a comprehensive parametric analysis that adopting a suitably high target recurrence interval wind speed (500 years) and the 78% fractile (that is, 78% of the data exists below this value) extreme load effect can serve as a characteristic ULS loading for dynamically sensitive structures.

Adopting a high MRI and a load factor of unity does not explicitly consider the significance of uncertainty associated with the extreme wind speed estimate or other wind loading parameters. Studies that have propagated uncertainties associated with a comprehensive range of parameters have found load factors for dynamically sensitive structures that are in some cases significantly larger than contemporary design practice (Gabbai et al, 2008; Bashor and Kareem, 2009; Kwon et al, 2015). Amongst these studies there are significant differences in the magnitudes of the load factors derived depending on the analysis approach, parametric uncertainties considered and the definition of load factor. Comparison between studies is therefore difficult, particularly since later research was unable to replicate load factors recommended by Gabbai et al, (2008) which were as high as 2.3 for rigid buildings and 3.5 for flexible buildings. For the purposes of providing a basis of reference for this paper Bashor and Kareem (2009) recommended the load factor for a dynamically sensitive building is around 1.9 for the conversion between the serviceability limit state (SLS) loads to the ULS as compared to 1.6 for a rigid structure. This factor corresponds with a higher load than simply using a higher wind velocity as it also accounts for frequency and damping uncertainties. This factor is reasonably comparable to the same case considered by Kwon et al (2015). Their study examined a more extensive range of parameters and found that uncertainties associated with wind speed, frequency and damping contribute most to the uncertainty in the response of a dynamically sensitive structure.

The increased load factors for dynamically sensitive structures identified by these parametric studies (Gabbai et al, 2008; Bashor and Kareem, 2009; Kwon et al, 2015) were based on the assumption that the structure's dynamic properties are constant between SLS and ULS loading. However full-scale data indicates that as response increases, frequencies tend to decrease and structural damping increases. It is a common design assumption that damping will likely exceed nominal design values in the extreme responses associated with the ULS event as inelastic behaviour of the structure is to be expected (Bashor and Kareem, 2009; Allsop, 2011). Furthermore at the ULS aerodynamic damping is likely to play a more significant role as it generally increases proportional to wind speed.

As the above discussion highlights, the selection of reliable ULS wind loads for a super-tall tower may warrant a project specific reliability analysis that is not required for more typical structures. The following sections discuss two approaches using the tower described in introduction.

Reliability Analysis using the traditional "First-Order Second Moment" (FOSM) method

The governing code for the tower adopts the traditional approach of converting from SLS to ULS loads using a standard load factor. In recognition of the limitations of this approach as highlighted in the discussion above, project specific load factors were derived for ULS design using the FOSM reliability framework.

The wind tunnel study employed the high-frequency force balance technique with the analysis referencing deterministic "best-estimates" of the tower's dynamic properties provided by the structural engineer. The estimates of the extreme wind speed that was referenced for the analysis was based on the studies described below.

<u>Wind Climate Uncertainties</u>

The wind speed data set from the local airport varied in quality due to location of the anemometer and due to doubtful or inconsistent readings. Using a Fisher-Tippett Type I fit to derive a statistical model of the wind climate a detailed analysis was conducted to derive annual maxima wind speeds. This was based on the historical data set, considering the entire 36 year record, a more recent portion of the record considered most reliable and filtered with and without doubtful readings. An analysis was also undertaken in which records for days of thunderstorms were removed from the dataset.

The resulting predictions of the 50-year return period, 3-second gust speed at 10 m height in standard open terrain was found to range from a low of 28 m/s to as high as 35 m/s, depending on the various assumptions used. Omitting thunderstorms gave the lowest value of 28 m/s. Analyses of wind speed records from the two regional stations provided corresponding 50-year speeds of 30.5 m/s and 32.4 m/s. Ultimately based on judgement the statistical wind climate that was considered to be the "most probable" 50-year 3-second gust speed in standard open terrain is about 31.5 m/s.

<u>Wind Load Factors</u>

The theory behind load factors, in particular first-order second-moment theory, and its application to wind loads in particular was further illuminated by Davenport (1983). From those references the Load Factor λ_W for wind in combination with dead load can be expressed in the form:

$$\lambda_W = \frac{1}{K_W} e^{\alpha^2 \beta V_w} \tag{1}$$

Where K_W is a bias factor, $\alpha$is a load combination factor taken as 0.75 based on Davenport (1983), β is the reliability index, which is taken here as 3.0 based on Ellingwood and Tekie (1999), and V_W the coefficient of variation (COV) of the wind load. The COV of the wind load is the standard deviation of the "error" in load due to various sources of uncertainty divided by the mean value of the maximum load occurring during the service life of the structure (which was taken as 100 years for the project, as compared to the local code design speed which corresponds to the 50 year return period).

For the project, efforts were made to avoid any bias in the load estimates by using wind tunnel simulations employing best practices, implying that the bias factor is unity i.e. K_W = 1.0

which agrees with Bartlett et al (2003). Also in wind tunnel studies efforts are made to minimize V_W by minimizing all the possible sources of error in predicting the wind loads. However, there are limits to how much they can be reduced. For a building that is dynamically sensitive the wind loads for a particular wind direction can, for certain ranges of speed, be related to wind speed by an expression of the form

$$W = CU^2\left(\frac{U}{f}\right)^{n-2}\frac{1}{\zeta^{n_{damp}}} = C\frac{U^n}{f^{n-2}\zeta^{n_{damp}}} \qquad (2)$$

where C is an empirical constant, U the wind speed, f the natural sway frequency and ζ the damping ratio. Since the variables C, U, f and ζ are independent of each other we deduce from Equation 2 that the COV for W is given by

$$V_W \approx \sqrt{V_C^2 + (nV_U)^2 + ((n-2)V_f)^2 + (n_{damp}V_\zeta)^2} \qquad (3)$$

where V_C, V_U, V_f, and V_ζ are the COVs of each of the variables C, U, f and ζ, respectively. These were estimated to be V_C = 0.10, V_U = 0.10, V_f = 0.05 and V_ζ = 0.32.. Note that V_C has contributions from a number of sources including: selection of wind profile and turbulence to be simulated in the wind tunnel; accuracy of wind simulation; instrumentation and calibration; Reynolds number effects, etc. V_U is primarily due to the inherent variability of the serviceability level wind speed and was estimated from the curve of wind speed versus return period. V_f was based on RWDI's experience in comparing full scale measurements of building's frequencies with computations by knowledgeable structural designers and agrees well with Kwon et al, 2015. The value of V_ζ was taken from Davenport (1983). The values of the exponents n and n_{damp} were determined for the building's dynamic properties, referencing the analysis of the wind tunnel data for the wind directions creating the highest loading and the appropriate wind speed range. During the design process different sets of dynamic properties were analysed and the values of the exponent n were found to range from about 2 to approximately 4 depending on the dynamic property set, and n_{damp} ranged from about 0.3 to 0.5. As an example, if we consider n = 4 and n_{damp} = 0.5

$$V_W \approx \sqrt{0.1^2 + (4\times 0.1)^2 + (2\times 0.05)^2 + (0.5\times 0.32)^2} = 0.45$$

Using this in Equation 1, with α = 0.75, reliability index β = 3.0 and K_W = 1.0, leads to a structural reliability-based load factor of

$$\lambda_W = \exp(0.75^2 \times 3.0 \times 0.45) = 2.1$$

Using this approach, a load factor consistent with a structural reliability index β = 3.0 was determined for each dynamic property set. The predicted 100-year return period wind loads based on the "most probable" wind climate model were then multiplied by the appropriate structural reliability-based load factor to obtain the ULS loads for design.

Reliability Analysis using Numerical Statistical Techniques

This project highlights the significance of Kwon et al's findings (2015) that the uncertainties associated with wind speed, frequency and damping contribute most to the uncertainty in the response of a dynamically sensitive structure. In the case of this project, the estimate of the 50-year return period wind speeds varied by approximately ±10% depending on the various judgements and assumptions used to interpret the imperfect wind speed data set. Considering that that the exponent n ranged as high as 4, such variations in the wind speed estimate clearly have a significant impact on predictions of the response. Furthermore the dynamic sensitivity of the tower highlights the significance of the additional uncertainties associated with frequency and damping when determining reliable ULS design loads.

Wind Climate Bootstrapping Analysis

This section of the paper describes a probabilistic study to derive reliable ULS wind loads for the project utilizing numerical statistical techniques to directly account for the dominant sources of uncertainty – those associated with wind speed, frequency and damping.

An assessment of the extreme wind climate for the project site was performed by conducting a Superstation Extreme Value Analysis on annual maxima wind speeds, based on the methodology described by Peterka and Shadid (1998). A Fisher-Tippett Type I fit was used. A superstation is a group of relatively closely located meteorological stations whose yearly maxima are produced by different storms, and can therefore be combined to produce effective record lengths longer than associated a single station. For this study, eleven stations within equatorial South East Asia were combined corresponding to a total effective record of approximately 260 years with a geographic distribution generally similar to the eleven stations that were referenced to derive the design speeds in HB 212-2002. In order to assess the uncertainty in the resulting extreme value fit, the bootstrapping approach described by Cook (2004) was employed.

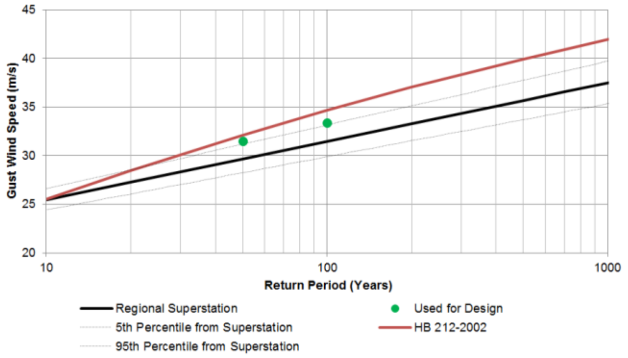

Figure 2. 3-Second Gust Wind Speeds at 10 meters height in Open Terrain

The results of the Superstation analysis in terms of the mean estimate, 5% and 95% confidence intervals have been plotted in Figure 2. Also plotted are the design wind speeds for this region from HB 212-2002, and the wind speeds used for design of the project that were derived based on the studies described in Section 3.1. A comparison indicates the design wind speeds for the project selected by RWDI and those provided in HB 212-2002 are conservative as compared to the bootstrapping analysis. It is also notable that the difference between the upper and lower bound estimates identified in Section 3.1 is almost double that between the 5% and 95% confidence limits, highlighting the poor quality of the local wind speed data set.

Monte Carlo Simulations of Ultimate Limit State Wind Loads

For the ULS wind loads of the tower the dominant sources of uncertainty are those associated with wind speed, frequency and damping. The following algorithm was developed to derive probabilistic estimates of ULS wind loads at a MRI of interest:

Step 1) Distribution of Wind Speed at target MRI was generated based on bootstrapping analysis described in Section 4.1.

Step 2) Distributions of frequency and damping were generated referencing the deterministic "best-estimates" as mean values and their distribution based on Kwon et al (2015). In the case of frequency a lognormal distribution was assumed with a COV of 0.05. For damping a lognormal distribution was assumed with a COV of 0.40. It was assumed there was no correlation between variations of these parameters.

Step 3) Monte Carlo simulations conducted to generate response estimates at the target MRI. For each simulation experiment the high-frequency force balance data was analysed (consistent with study in Section 3) referencing randomly sampled estimates of the wind speed at the MRI and randomly generated sets the tower's dynamic properties based on the distributions derived in Steps 1 and 2.

The results of Monte Carlo simulations entail sampling errors. These errors reduce with increasing sampling size, but to achieve higher precision imposes increasing computational processing. It was found samples of 10,000 simulations provided stable 3 significant-figure estimates of the mean wind-induced response and 2 significant-figure estimates for the 5% and 95% fractile.

Results and Discussion

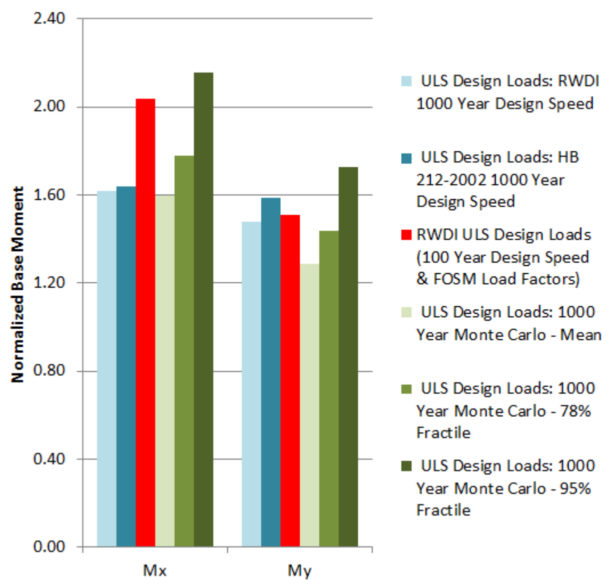

Figure 3. Estimates of ULS Base Moments based on different methodologies.

The results of the studies described above are summarized in Figure 3. Results are also provided based for two comparison cases where the analysis of the wind tunnel data referenced deterministic estimates of the 1000 year return period wind speed (based on RWDI "most probable" wind climate model and HB 212-2002) and "best-estimate" values for the tower's structural properties.

Comparison of the results indicate RWDI's ULS Design loads fall somewhere between the 78% and 95% fractile loads based on the Monte Carlo simulations for both My and Mx. It is not possible to generalize based on a single case study, but these results suggests both approaches are capable of generating risk consistent ULS wind loads for dynamically sensitive structures such as this project.

For the two cases (dark and light blue) utilizing deterministic estimates of the 1000 year return period wind speed and "best-estimate" values for the tower's structural properties they are generally comparable to each other. They fall between the 78% and 95% fractile loads for My, but for Mx these cases are significantly lower than RWDI's ULS Design loads and are approximately equal to the mean estimate from the Monte Carlo simulations. This difference in apparent reliability between Mx and My can be attributed to the increased uncertainty associated with the tower's dynamic properties not being captured. This comparison highlights that a structure which has high dynamic sensitivity may warrant a project specific reliability analysis that is not required for more typical structures.

The numerical analysis approach demonstrated with this case study – coupling bootstrapping analysis of the extreme wind climate and Monte Carlo simulations of the response – has the advantage of allowing realistic project specific uncertainties in input parameters to be studied and their impact on its outputs by means of a realistic model of the wind-induced response. In light of these advantages, this numerical approach shows promise of being potentially capable of deriving "realistic" estimates of probability of failure within a performance based design framework as compared to the nominal probabilities associated with the β values of the traditional FOSM approach.

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