Monitoring of wind-induced motion of tall buildings

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Abstract

This paper describes the results of monitoring the wind-induced building motion of five tall buildings in New Zealand between 2009 and 2012: four in Wellington and one in Auckland. The measured accelerations were compared with acceptability criteria from ISO Standard 10137:2007. The accelerations were within the acceptability criteria for four of the buildings, and exceeded the criteria by about 20% for the fifth building. The measured wind-induced accelerations are approximately proportional to the cube of the wind speed. Various aspects of the design of tall buildings with the potential to cause high accelerations have been discussed, and a simple predictive equation is shown to produce a reasonable estimate of building motion.

1 Introduction

This paper describes the results of monitoring the wind-induced motion of tall buildings in New Zealand between 2009 and 2012: four in Wellington and one in Auckland. The monitoring has been undertaken as part of a research programme to develop an improved methodology for the design of buildings, to ensure that wind-induced motion of new tall buildings remains within acceptable limits.

2 Acceptability criteria for building motion

ISO Standard 10137:2007 [1] provides guidance for human response to wind-induced motions in buildings. It indicates that peak accelerations should not exceed the basic evaluation curve for the respective occupancy. There are separate curves for offices and for residences, with the limits for residences being 2/3 of those for offices. The ISO figure is reproduced in Figure 1.

The ISO limits in the frequency range 1 to 2 Hz are approximately:

1. Offices 6.1 milli-g
2. Residences 4.1 milli-g

Kwok [2] has discussed the ISO criteria in comparison with other published criteria, and indicated that they are at the low end of the range.

The ISO standard specifies that both translation and torsion should be considered in applying the criteria. However it is not clearly specified in the standard whether these should be considered separately or combined. If the effects of translation and torsion are combined, then the analysis is essentially concerned with the accelerations at the corners of the top floor of the building. If
translation and torsion are considered separately, then the analysis is concerned with the accelerations on the whole of the top floor.

![Graph showing evaluation curves for wind-induced vibrations in buildings in a horizontal direction for a one-year return period.](image)

**Key**
- $A$: peak acceleration, m/s$^2$
- $f_0$: first natural frequency in a structural direction of a building and in torsion, Hz

1. Offices  
2. Residences

*Figure 1: ISO 10137:2007 Evaluation curves for wind-induced vibrations in buildings in a horizontal direction for a one-year return period.*

Note that in Figure 1, accelerations are measured in m/s$^2$. However, in the remainder of this paper, the more commonly used units milli-g have been preferred.

In view of Kwok’s observation that the ISO criteria are the lower end of the range of acceptability criteria, we suggest that translation and torsion should be considered separately in this analysis of building motion comparison with the criteria, rather than considered together. The practical effect of this is that translation accelerations alone are on average about 30% less than the combined accelerations, and therefore the building is less likely to exceed the criteria when the accelerations are considered separately. This suggestion is also based in the knowledge that none of the measured buildings were reported to us as experiencing excessive wind-induced motion.

### 3 Selection of the buildings

Two of the buildings which have been analysed are part of the New Zealand GeoNet project. The instrumentation was not specifically installed for the wind motion research. It was fortunate that data from these buildings became available for inclusion in this study.

Three of the buildings were specifically chosen by Opus and University of Auckland for wind motion research. The factors which were considered in selecting these buildings were as follows:
Two buildings in Wellington were chosen, primarily due to the high proportion of windy conditions in Wellington.

One building in Auckland was also selected, to provide a wider geographical spread of buildings.

Some New Zealand buildings have been reported as having high or uncomfortable motion in strong winds. We made a decision not to focus on these buildings, preferring to study representative tall buildings which could have wind-induced motion at around the acceptability criteria limits. Consequently, none of the buildings that we studied had been previously reported to us as having detectable wind-induced motion.

The equipment was located on the building roofs, where it would not typically be noticeable to building users. Buildings with sloping roofs, or with roofs where access was unsafe were therefore unsuitable.

Relatively modern buildings were chosen, less than 20 years old.

Some building owners decided that there was a risk of adverse publicity if it became known that their buildings were being investigated for wind-induced motion, and therefore declined our request for access. Consequently, we made it standard practice in our approach to building owners, that it was stated that the buildings would not be named in publications arising from the research.

4 Description of the buildings

The five buildings which have been analysed are referred to as Buildings A, B, C, D and E, which are listed in the order that monitoring commenced.

- Building A is in Wellington. It is 10 storeys high, with a rectangular planform, and a steel frame structure. Monitoring as part of the New Zealand GeoNet project has been on-going since early 2009.
- Building B is in Wellington. It is 25 storeys high, with an approximately square planform, and has a structure consisting of concrete perimeter columns with a central core. It was monitored by Opus during the period from 21 August 2009 to 15 October 2009.
- Building C is in Wellington. It is 17 storeys high, with an approximately square planform, and has a concrete structure including a wall on one side, and an offset core adjacent to the concrete wall. It was monitored by Opus during the period from 21 October 2009 to 22 February 2010.
- Building D is in Auckland. It is 25 storeys high, with a rectangular planform, and has a concrete structure. It was monitored by Opus during the period from 20 October 2010 to 27 May 2011.
- Building E is in Wellington. It is 28 storeys high, and has a concrete structure. Monitoring as part of the New Zealand GeoNet project has been on-going since early 2012.

Multiple modes of vibration have been identified for each building through analysis of the motion time histories. The measured X, Y and Torsion frequencies for each building are listed in Table 1. A notable feature of these measured frequencies is the low torsion frequency of Building C. The frequencies in all three directions are similar, and the frequencies in the Y direction and in torsion are the same, indicating that the building oscillates with coupled mode response.
Table 1: Measured X, Y and Torsion frequencies for the five buildings

<table>
<thead>
<tr>
<th>Building</th>
<th>X direction</th>
<th>Y direction</th>
<th>Torsion</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.56 Hz (~EW)</td>
<td>1.42 Hz (~NS)</td>
<td>2.10 Hz</td>
</tr>
<tr>
<td>B</td>
<td>0.55 Hz (~NS)</td>
<td>0.54 Hz (~EW)</td>
<td>0.84 Hz</td>
</tr>
<tr>
<td>C</td>
<td>0.63 Hz (~NS)</td>
<td>0.65 Hz (~EW)</td>
<td>0.65 Hz</td>
</tr>
<tr>
<td>D</td>
<td>1.09 Hz (~NS)</td>
<td>0.79 Hz (~EW)</td>
<td>1.41 Hz</td>
</tr>
<tr>
<td>E</td>
<td>0.44 Hz (~NS)</td>
<td>0.46 Hz (~EW)</td>
<td>0.68 Hz</td>
</tr>
</tbody>
</table>

5 Instrumentation

Buildings A and E have been instrumented as a part of the Geonet Building Instrumentation Programme funded by the New Zealand Earthquake Commission (EQC). This is a long term programme that aims to install earthquake strong motion instruments in up to 30 structures across New Zealand. The equipment is set up for earthquake monitoring, recording at a rate of 200 Hz, with limitations on its ability to record continuously. The consequence of this is that much less statistical data has been available for the analysis of Buildings A and E, compared to Buildings B, C, D.

The instrumentation in Buildings B, C and D was installed by Opus. The recording was continuous, typically at a rate of 25 Hz. The two accelerometers were mounted at diagonally opposite corners of the roof, which enabled the X, Y and torsion modes of vibration of the building to be measured.

The anemometer was located at a height of 2.5 m above the roof at the windward corner of each building for the most common wind directions. This was the NW corner for buildings B and C, and the SW corner for Building D. These locations meant that the anemometer was sheltered by the building for the other less common wind directions. Only data for northerly winds has been included in the analysis for buildings B and C in this paper, and only data for westerly winds has been included in the analysis for building D.

The height of the anemometer was less than ideal for measurement of the reference wind speed, but provided an adequate measure of the variation in wind speed for the most common wind directions. The reasons that a taller anemometer pole was not used included:

- It was necessary to comply with the city planning rules concerning the heights of structures on the building roofs.
- A short pole was simple and safe to install.
- We wanted to avoid drawing any attention to the monitoring being done on the building, or causing any difficulties for the building owners.

6 Data Analysis

Table 2 lists a summary of accelerations measured during the single biggest building-motion event for each building. The most extensive data was obtained for Buildings C and D.
Table 2: Summary of accelerations measured during the single biggest building-motion event for each building.

<table>
<thead>
<tr>
<th>Measured</th>
<th>Building</th>
</tr>
</thead>
<tbody>
<tr>
<td>X direction at centre of building (milli-g)</td>
<td>A  2.1 B 1.1 C 1.8 D 0.6 E 3.2</td>
</tr>
<tr>
<td>Y direction at centre of building (milli-g)</td>
<td>3.2 2.9 3.0 1.3 4.3</td>
</tr>
<tr>
<td>Combined XY at centre of building (milli-g)</td>
<td>3.3 2.9 3.6 1.3 4.4</td>
</tr>
<tr>
<td>Acceleration at the corners due to torsion (milli-g)</td>
<td>2.3 0.7 3.6 0.5 2.9</td>
</tr>
<tr>
<td>Corner (max acceleration at either corner) (milli-g)</td>
<td>3.6 3.2 5.8 1.7 6.2</td>
</tr>
<tr>
<td>Amplitude at centre of building (mm)</td>
<td>5.4 3.8 1.2 10.5</td>
</tr>
<tr>
<td>Airport mean wind speed (m/s)</td>
<td>23 16 14 14 18</td>
</tr>
<tr>
<td>Airport wind direction</td>
<td>210 300 340 240 340</td>
</tr>
</tbody>
</table>

Note:

1. Accelerations in torsion are measured at the corners of the building, relative to the centre of the building.
2. Wellington reference wind speed was measured at Wellington Airport. Auckland reference wind speed was measured at Whenuapai Airport.

7 Relationship between wind speed and acceleration

The relationship between wind speed measured on the roof of the building, and the acceleration at the centre of the roof of the building, has been analysed for Buildings B, C and D. The best correlation was obtained from the so-called “effective wind speed” measure of wind speed, which is the average of the 1-hour mean wind speed and the maximum gust speed in the hour. Some measures of the wind speeds closer to the building motion event were also included in the analysis, including the 100 s mean wind speed, the 10 s mean wind speed, and the maximum wind speed during the 100s. The correlation with the measured accelerations was examined for all these wind speed measures. It was notable that the two wind speed measures for the whole hour (the mean and the maximum) produced substantially better correlations than the other measures, and the best correlation of all was achieved using the average of the 1-hour mean and the 1-hour maximum. For Building C the data has been analysed for all hours when the 1-hour mean wind speed at the building for northerly winds exceeded 7 m/s; there were 535 hours of data in this category for Building C. The resulting plot is shown in Figure 2. The data has been further analysed by averaging the measured accelerations into bands, as shown in Figure 3.

There was a very good power-law fit to the band-averaged data for all three buildings. The exponent of the power-law fit calculated for the band-averaged data is as follows:

- Building B: 2.89
- Building C: 3.10
- Building D: 3.18
For the three buildings combined, the average exponent of the power-law fit is 3.06. The measurements consequently confirm our expectation that the exponent would be close to 3 for these buildings, as proposed by Cenek et al [3]. This data has been further described by Carpenter et al [4].

8 Predicted annual maximum building motion

A statistical analysis of the largest building motion events has been applied to predict the building motions with a 1-year return period. These are listed in Table 3. Also listed here are the ISO 10137 limits for each building, and the predicted accelerations calculated using equations (1) and (2). Building C exceeds the ISO limit by about 20% at the centre of the building. The building has a coupled mode response which contributes to the higher measured accelerations. The other four buildings are within the limits.

Figure 4 plots the relationship between the estimated annual maximum combined XY accelerations from the measured data compared to the accelerations predicted using equation (1). The best fit line through the data indicates that the prediction, on average, under predicts the measured data by 12%. It is reassuring to find that one of the tools that have been applied for many years provides results which are so close to the measured accelerations of the real buildings.

\[
a = \frac{0.46\bar{V}(h)^3}{f m_0}
\]  

(1)

where

- \( \bar{V}(h) \) = mean hourly wind speed at the top of the building (m/s)
- \( f \) = fundamental frequency (Hz)
- \( m_0 = \rho A \) = mass per unit length over the top one third of the structure (kg/m)
- \( a \) = peak resultant acceleration (m/s²)
\( \rho_b \) = building density (kg/m\(^3\))  
\( A \) = building plan area (m\(^2\))

The predictive equation (1) calculated in 1989 uses 1-hour mean wind speed in the analysis, which was consistent with the dynamic analysis procedures in the wind loading standard at that time. In 2002 the standard [2] was changed to use gust wind speeds. Also it seems prudent to revise the equation for the small 12% average under prediction that has been measured. The revised equation becomes:

\[
a = \frac{0.113V_{\text{des,1-year}}^3}{fm_0} \tag{2}
\]

where \( V_{\text{des,1-year}} \) may be calculated using the loading standard AS/NZS 1170.2 [6].

9 Conclusions

This paper describes the results of monitoring the wind-induced building motion of five tall buildings between 2009 and 2012: four in Wellington and one in Auckland. The buildings were selected to be fairly representative of tall buildings in New Zealand; buildings which are known to experience high accelerations were not selected for the study. The measured accelerations were compared with acceptability criteria from ISO Standard 10137:2007. The accelerations were within the acceptability criteria for four of the buildings, and exceeded the criteria by about 20% for the fifth building. The relationship between wind speed and acceleration has been examined for three of the buildings. The measured wind-induced accelerations are approximately proportional to the cube of the wind speed. This demonstrates that accurate estimation of the wind speed is critical for accurate design predictions of wind-induced building motion. A simple predictive equation has been found to give reasonable estimates of the expected annual maximum building motion.

Acknowledgements

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References