Full-scale performance evaluation of tall buildings under wind
Rachel Bashor, Sarah Bobby*, Tracy Kijewski-Correa, Ahsan Kareem
University of Notre Dame, Notre Dame, IN, USA

1. Introduction

The existing design procedures for tall buildings rely exclusively on computational and scaled experimental models tested in wind tunnels. Although these models have been extensively refined, there is still considerable uncertainty associated with the actual performance of these structures under dynamic excitation. As structures become increasingly taller and more complex, it is imperative that engineers be able to accurately predict their performance in order to create cost-effective designs satisfying survivability, serviceability, and habitability requirements. However, in the case of tall buildings, habitability requirements — accelerations adversely affecting the comfort of the building occupants — often become the governing limit state for design. As such there is considerable interest in better quantifying not only the levels of acceleration that cause discomfort to occupants, but also in enhancing the reliability of predicted accelerations in the design process (Bentz and Kijewski-Correa 2009). The reliability of these design predictions rests squarely on the procedures to predict accelerations through wind tunnel testing and the dynamic properties supplied by designers as part of this process.

Full-scale monitoring provides the most faithful means to validate both of these aspects. Unfortunately, access to buildings for full-scale validation of design predicted accelerations has been quite limited (e.g., Littler, 1991; Li et al., 2004), and while there have been greater efforts to catalog in-situ dynamic properties (e.g., Lagomarsino, 1993; Satake et al., 2003), data from buildings of significant height is generally lacking. As a result, there has been growing attention in full-scale monitoring of tall buildings, with the addition of several tall buildings in China, e.g., Di Wang Tower and the Bank of China (Li et al., 2003a, 2003b, 2005; Xu et al., 2003). Unfortunately, most of the full-scale monitoring efforts that have been published in the literature have not been sustained long enough to observe responses under a wide spectrum of wind events necessary to fully validate design practices. In response to this need, nearly a decade ago, the University of Notre Dame, The Boundary Layer Wind Tunnel Laboratory (BLWTL) at the University of Western Ontario (UWO), and Skidmore, Owings, & Merrill LLP (SOM) in Chicago established the Chicago Full-Scale Monitoring Program (CF SMP) to monitor three tall buildings in Chicago with the goal of correlating the measured responses and dynamic properties of the buildings with their predicted behavior under a range of wind environments. CF SMP is a unique effort as it has accumulated a decade of data for three signature tall buildings whose structural systems are representative of those common to high-rise design. While estimates of the dynamic properties of these buildings...
under select wind events have been reported in previous publications (e.g., Kijewski-Correa et al., 2006; Kijewski-Correa and Pirnia, 2007; Pirnia et al., 2007; Bentz and Kijewski-Correa, 2009) and select events have been used to validate wind tunnel predicted accelerations (e.g., Kijewski-Correa et al., 2006; Kijewski-Correa and Kochly, 2007). This paper explores a larger cross section of recorded datasets to explore the variability and potential amplitude-dependence in in-situ dynamic properties for the fundamental sway modes of these buildings using established system identification techniques and then, using these in-situ dynamic properties, compares predicted wind tunnel RMS accelerations to the values observed in full-scale for these events.

2. Description of instrumented buildings

To ensure that this program continues to collect valuable data, at the request of the building ownership, the structures’ identities must remain anonymous so they are referred to herein generically as Buildings 1–3. These buildings are all located in downtown Chicago and are representative of structural systems common to high-rise design, i.e., buildings standing 60 stories or higher. As discussed in Kijewski-Correa et al. (2006), each building utilizes straight shaft reinforced concrete caissons extending to bedrock. The primary lateral load-resisting system of Building 1 is comprised of a steel tube with exterior columns, spandrel ties, and additional stiffening elements. This allows a near uniform load distribution on the columns across the flange face with very little shear lag so that the building is dominated by cantilever action. Building 2 is a reinforced concrete building with shear walls located near the core of the building to provide lateral load resistance. At two levels the core is tied to the perimeter columns via reinforced concrete outrigger walls to control the wind drift and reduce the overturning moment in the core shear walls. Finally, Building 3 consists of a steel, moment-connected, framed tube comprised of closely spaced wide columns and deep spandrel beams along multiple frame lines. A summary of the design-specified fundamental frequencies ($f_x$, $f_y$) and critical damping ratios ($\zeta_x$, $\zeta_y$) for the x and y axes of the buildings is found in Table 1. Note that the each building’s x and y axes are assumed to positively align with cardinal directions of East and North, respectively. A more complete description of the finite element models that generated these frequencies was provided previously in Kijewski-Correa et al. (2006), while damping values were assumed by the designers based on general rules of thumb.

3. Description of instrumentation

3.1. Instrumentation system for building response

As discussed in Kijewski-Correa et al. (2006), the primary instrumentation systems were installed in Buildings 1, 2, and 3 on June 14, 2002, June 15, 2002, and April 30, 2003, respectively. Each building is instrumented with four Columbia SA–107 LN high-sensitivity force-balance accelerometers that are capable of accurately measuring accelerations down to 0 Hz with a 15 V/g sensitivity. The accelerometers were installed in orthogonal pairs at opposite corners of the ceiling at the highest possible floor of each building, noted schematically in Fig. 1. The data outputs of the accelerometers are sampled every 0.12 s by Campbell CR23X data loggers to yield an overall system resolution of approximately 0.001 milli-g. The data logger is programmed to capture continuous 10 min statistics from the accelerometers, and when motions exceed a user-defined threshold, the data logger automatically begins to capture continuous hour-long time-histories of data for as long as the threshold level is exceeded. Estimates of the sway and torsional responses are obtained via algebraic manipulation of the accelerometer outputs. While Building 1 also employs a high-precision global positioning system to monitor its displacements, this data will not be investigated in the present study, though additional details can be found in Kijewski-Correa and Kochly (2007).

3.2. Instrumentation system for wind field characteristics

Wind speeds referenced in this paper are taken from one of the National Oceanic and Atmospheric Administration Great Lakes Environmental Research Laboratory (NOAA GLERL) Young 5103 V, propeller-type sensors. This sensor is located approximately 4.83 km offshore of Chicago on the Harrison–Dever crib and samples wind velocity at 1 Hz and records 5 min averaged statistics (NOAA GLERL, 2006). Gradient wind speeds referenced in this study are estimated from these surface wind speeds using a procedure described in Kijewski-Correa et al. (2006), which assumes gradient height to be 300 m over open water and uses methods to account for the influence of terrain roughness and fetch (ESDU, 2001). A minor correction to the wind azimuth is also applied to account for the rotation of the velocity vector with height (Davenport, 1987).

4. System identification techniques

In order to maintain consistency with previous published analyses of data from the CFMSP, as described in Kijewski-Correa et al. (2006), frequency domain estimates will be executed on low-bias (less than −2%) power spectra using the Half Power

<table>
<thead>
<tr>
<th>Property</th>
<th>Building 1</th>
<th>Building 2</th>
<th>Building 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_x$ (Hz)</td>
<td>0.201</td>
<td>0.148</td>
<td>0.131</td>
</tr>
<tr>
<td>$f_y$ (Hz)</td>
<td>0.142</td>
<td>0.156</td>
<td>0.131</td>
</tr>
<tr>
<td>$\zeta_x$, $\zeta_y$</td>
<td>1.0%</td>
<td>1.0%</td>
<td>1.0%</td>
</tr>
</tbody>
</table>

* 1.0% damping used for serviceability and 1.5% used for strength design.
Bashor Transforms applied to locally averaged Random Decrement Signatures (RDS) (Kijewski and Kareem, 2003). This implementation of the Random Decrement Technique (RDT) begins with pre-processing by Butterworth bandpass filters to isolate each mode of interest and a positive point trigger value is enforced to select the segments of the response averaged to generate the RDS (Bashor et al., 2005). As RDT is inherently sensitive to the trigger conditions, which directly influence the number of segments captured and thereby the quality of the RDS, repeated triggering is implemented, as proposed by Kijewski-Correa (2003) and previously implemented in the context of the CSFMP by Bashor et al. (2005) and Kijewski-Correa and Pirnia (2007). This is accomplished by generating a suite of RDSs associated with a range of positive point triggers that are within a few percent of the desired trigger $X_p$. The resulting RDSs are then processed using the Hilbert Transform and the natural frequency and the critical damping ratio are determined from the phase and amplitude of the analytic signal, respectively. The resulting vector of frequency and damping estimates are then averaged to yield mean estimate and corresponding coefficient of variation (CoV). The reliability of these frequency and time domain approaches for system identification from wind-induced vibration data has been previously evaluated by Kijewski and Kareem (2002).

Both of the aforementioned approaches assume stationarity of the data, to varying extents. Although in practice wind is often viewed as a stationary random process, transient or nonstationary features are generally present in most field data. Therefore, before data is processed by any of the aforementioned techniques, its stationarity is established using the Run and Reverse Arrangement Tests (Bendat and Pierson, 2000). In addition to these two tests, additional verifications are made using a method proposed by Montpellier (1996).

### 5. In-depth study of two wind events

Before exploring the trends in dynamic properties over multiple wind events, two wind events are selected for in-depth discussion to evaluate the performance of the system identification techniques being employed. It should be noted that confirmation of stationarity by at least two of the three tests discussed in the previous section was executed to qualify the two wind events featured here as stationary; the same criteria will be used for all data presented in later sections of the paper. The identified natural frequencies and critical damping ratios are respectively presented in Tables 2 and 3 for Wind Events 1 and 2 with respective mean hourly gradient winds and average wind respectively presented in Tables 2 and 3 for Wind Events 1 and 2. For Building 2, while RDT results are quite consistent (within 12% between events), HPBW results deviate by as much as 48%. This again can be credited to the fact that the two steel buildings are characterized by considerably more narrowband spectra and are thereby more susceptible to variance errors in the presence of limited amounts of data. Interestingly, while RDT proves to be the more consistent damping estimator, particularly for the steel buildings, particularly Building 3, which is the building with the strongest degree of coupling between modes. Interestingly, the time and frequency domain system identification approaches are most consistent in their damping estimate for the concrete structure (within 14%) and show the most significant deviation for the steel structures whose power spectra are more narrowband and generated with fewer spectral averages for a fixed duration wind event. Furthermore, given the low bias requirement placed on the estimation of the power spectra, it is not surprising that the damping estimates in the frequency domain are not consistently larger than the unbiased time domain estimates, affirming that the residual error source is indeed random. When comparing damping values between the two events, Building 2 yields the most consistent damping values, regardless of the method employed, with HPBW results being within 22% and RDT results being within 18%. For Building 1, while RDT results are quite consistent (within 12% between events), HPBW results deviate by as much as 50%. The case is similar for Building 3, where RDT damping results are within 19% of one another, while HPBW results deviate by as much as 48%. As mentioned above, the RDT CoVs are less than 1%. When comparing values between the wind events, the natural frequencies diminish slightly in the y-sway response of Building 2 in the second event. Note that for this event, the y-axis experiences acrosswind action and comparatively larger responses; therefore, the reduction in frequency is consistent with the amplitude dependence noted in previous studies (Kijewski-Correa and Pirnia 2007).

### 5.1. Performance of system identification techniques

For both wind events the natural frequency estimates are completely consistent between the time and frequency domain techniques, with RDT CoVs less than 1%. When comparing values between the wind events, the natural frequencies diminish slightly in the y-sway response of Building 2 in the second event. Note that for this event, the y-axis experiences acrosswind action and comparatively larger responses; therefore, the reduction in frequency is consistent with the amplitude dependence noted in previous studies (Kijewski-Correa and Pirnia 2007). On the other hand, critical damping ratios estimated by RDT have CoVs that are one to two orders of magnitude higher than those associated with natural frequency estimates. In fact, the CoVs are larger for the steel buildings, particularly Building 3, which is the building with the strongest degree of coupling between modes. Interestingly, the time and frequency domain system identification approaches are most consistent in their damping estimate for the concrete structure (within 14%) and show the most significant deviation for the steel structures whose power spectra are more narrowband and generated with fewer spectral averages for a fixed duration wind event. Furthermore, given the low bias requirement placed on the estimation of the power spectra, it is not surprising that the damping estimates in the frequency domain are not consistently larger than the unbiased time domain estimates, affirming that the residual error source is indeed random. When comparing damping values between the two events, Building 2 yields the most consistent damping values, regardless of the method employed, with HPBW results being within 22% and RDT results being within 18%. For Building 1, while RDT results are quite consistent (within 12% between events), HPBW results deviate by as much as 50%. The case is similar for Building 3, where RDT damping results are within 19% of one another, while HPBW results deviate by as much as 48%. This again can be credited to the fact that the two steel buildings are characterized by considerably more narrowband spectra and are thereby more susceptible to variance errors in the presence of limited amounts of data. Interestingly, while RDT proves to be the more consistent damping estimator, particularly for the two steel buildings, when comparing results between the two events, the consistency is generally an order of magnitude better in the x-axis than the y-axis, which again experiences higher amplitude acrosswind response in Event 2. As a result the lack of “consistency” may not be the result of errors inherent to the method but potentially due to the amplitude-dependence previously observed in damping values in these buildings (Kijewski-Correa and Pirnia, 2007). In particular, in non-symmetric systems, the axes of the buildings typified by greater frame action tend to manifest more amplitude dependence in their dynamic properties: Building 1’s x-axis as a result of potential shear lag along the elongated floor plate and Building 2’s y-axis where primary lateral resistance is derived from slab and frame elements. The potential effects of amplitude

### Table 2

Estimated dynamic properties for wind event 1.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Test</th>
<th>Building 1</th>
<th>Building 2</th>
<th>Building 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>fn (Hz)</td>
<td>$\zeta$ (%)</td>
<td>fn (Hz)</td>
<td>$\zeta$ (%)</td>
</tr>
<tr>
<td>x-Sway</td>
<td>HPBW</td>
<td>0.204</td>
<td>0.65</td>
<td>0.178</td>
</tr>
<tr>
<td></td>
<td>RDT</td>
<td>0.204</td>
<td>0.87</td>
<td>0.178</td>
</tr>
<tr>
<td></td>
<td>(CoV, %)</td>
<td>(0.10)</td>
<td>(23.88)</td>
<td>(0.22)</td>
</tr>
<tr>
<td>y-Sway</td>
<td>HPBW</td>
<td>0.141</td>
<td>1.14</td>
<td>0.177</td>
</tr>
<tr>
<td></td>
<td>RDT</td>
<td>0.141</td>
<td>0.88</td>
<td>0.177</td>
</tr>
<tr>
<td></td>
<td>(CoV, %)</td>
<td>(0.10)</td>
<td>(8.82)</td>
<td>(0.63)</td>
</tr>
</tbody>
</table>

### Table 3

Estimated dynamic properties for wind event 2.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Test</th>
<th>Building 1</th>
<th>Building 2</th>
<th>Building 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>fn (Hz)</td>
<td>$\zeta$ (%)</td>
<td>fn (Hz)</td>
<td>$\zeta$ (%)</td>
</tr>
<tr>
<td>x-Sway</td>
<td>HPBW</td>
<td>0.204</td>
<td>1.37</td>
<td>0.178</td>
</tr>
<tr>
<td></td>
<td>RDT</td>
<td>0.204</td>
<td>0.89</td>
<td>0.178</td>
</tr>
<tr>
<td></td>
<td>(CoV, %)</td>
<td>(0.13)</td>
<td>(12.61)</td>
<td>(0.31)</td>
</tr>
<tr>
<td>y-Sway</td>
<td>HPBW</td>
<td>0.141</td>
<td>0.88</td>
<td>0.176</td>
</tr>
<tr>
<td></td>
<td>RDT</td>
<td>0.141</td>
<td>1.00</td>
<td>0.176</td>
</tr>
<tr>
<td></td>
<td>(CoV, %)</td>
<td>(0.13)</td>
<td>(6.43)</td>
<td>(0.96)</td>
</tr>
</tbody>
</table>
dependence may be evidenced by the fact that RDT damping values in the y-axis of all three buildings are larger in Event 2 than Event 1.

In previous analysis of the data from these buildings, damping values similar to the RDT results reported here were observed for Building 1, which tends to manifest comparable levels of damping on both axes. In the case of Building 2, y-sway damping values were previously observed to be larger than x-sway (Kijewski-Correa and Pirnia, 2007), and this was affirmed by the RDT results in this study. While Building 2 would be generally expected to have greater damping than Building 1, by virtue of its use of concrete, what is more interesting is the stark difference in damping values between the two axes of this concrete building. Reasons for this variation in damping along the two axes of this building were explored in Kijewski-Correa and Pirnia (2007) for this building and for other buildings in Erwin et al. (2007) and Bentz and Kijewski-Correa (2008). These studies have demonstrated that structural systems with greater degrees of frame action tend to dissipate more energy than systems that are dominated by cantilever action (member level axial deformations). In Building 2, shear walls and outriggers engage the exterior columns to achieve global cantilever action in the x-direction, whereas the y-direction is dominated by comparatively more frame action as the beams and slabs are the primary mechanisms to engage the lateral resistance of the building. Therefore the in-situ observations reported herein are consistent with the hypothesis that frame-dominated systems yield higher levels of damping. Even when comparing the RDT damping levels in the two steel buildings, one may hypothesize that Building 1, which has been observed to be dominated by cantilever action as an essentially pure tube (Bentz and Kijewski-Correa, 2008) would have less damping than Building 3, whose panel zone shear deformations have been extensively studied (Bentz et al., 2010).

In total these observations help to provide a more rational basis for the levels of damping to be assumed in design, as opposed to the crude respective assumptions of 1% or 2% critical for steel and concrete.

5.2. Comparison of results with finite element models

One of the primary objectives of the CFSMP is to validate assumptions made in the development of finite element models used in design by comparing their predictions and in-situ values. Comparisons of the results in Tables 2 and 3 to the design predictions in Table 1 affirms that Building 1’s in-situ fundamental sway frequencies show excellent agreement with the design predictions, which may be explained by the fact that, as mentioned previously, the structure’s elements are engaged primarily axially as a structural tube and may thereby be less susceptible to uncertainties in modeling material, section or connection details. Building 2 is 20% stiffer in-situ in the x-direction and 13% stiffer in-situ in the y-direction than originally predicted by the finite element model, while the converse is true for Building 3 (11% softer in-situ), reaffirming observations in previous studies (Kijewski-Correa et al., 2006). Moreover, the in-situ values for Building 2 suggest a greater consistency between the fundamental mode frequencies in sway than predicted by finite element models. A variety of sources have been and continue to be explored to determine the causes of this discrepancy, from rigid off sets in connection modeling, to the influence of panel zones, to the assumptions regarding in-situ material properties and the degree of cracking in concrete elements (Kijewski-Correa et al., 2006; Bentz et al., 2010).

6. Analysis of overall trends in dynamic properties

In the following, several hundred wind events were identified as stationary, i.e., having 80% of triggered data in a given event pass the aforementioned tests, and were analyzed using the same system identification approaches discussed previously. The number of records available for analysis varies for each building, depending on the number of times it triggers. In this study, 500 events will be analyzed for Building 1 and 200 for each of Buildings 2 and 3—events are defined as those having at least five triggered 1 h time histories.

6.1. Overall trends in frequency estimates

The natural frequency estimates for Buildings 1, 2, and 3 from both HPBW and RDT are shown in Figs. 2, 3 and 4, respectively. For reference, the natural frequency assumed for the finite element model is indicated in the figures by a thick horizontal line except in instances where discrepancies between in-situ data and predictions are so great that they cannot be reasonably shown on the same figure. It should be noted that the amplitude of responses for each of the buildings varies widely, as does the number of triggered events displayed. While more pronounced scattering may be evident with increasing amplitude, particularly with Building 2, this cannot be fully concluded given the limited amount of data. Still, clear evidence of amplitude dependence and softening with increased response is displayed for all three buildings, consistent with initial observations from a narrower subset of CFSMP data in Kijewski-Correa and Pirnia (2007). Interestingly, in some cases, though recognizing the limited extent of the amplitude ranges available for analysis here, the frequency softening appears to plateau (see Building 1 x-axis and both axes of Building 2), whereas in the case of Building 3, there is...
clear evidence of a strongly linear and decreasing trend with no apparent plateau over the range of amplitudes considered.

As discussed in the previous section, the discrepancies between predicted and in-situ natural frequencies for Building 2 could be attributed to numerous modeling assumptions. For example, given the age of the building, it is quite likely that the degree of cracking included in finite element models intended to represent the structure at key design limit states are not yet realized in the structure at present. To observe if the structure’s frequencies have reduced with time as the result of the natural process of cracking, Building 2’s frequency estimates are presented with time in Fig. 5. While there is potentially a slight softening evident from this figure, given the comparatively limited time span of these observations in comparison with the expected life cycle of the building, a definitive conclusion on the evolution of permanent softening cannot be reached.

6.2. Overall trends in damping ratios

The resulting damping values for Buildings 1, 2 and 3 are presented in Figs. 6, 7 and 8, respectively. As expected, the damping estimates show significantly more scatter than the frequency estimates for all three buildings and, consistent with the observations in Section 5, the scatter is more pronounced for HPBW. This of course emphasizes the importance of evaluating damping over a suite of events. To facilitate discussion, the average damping values for each building are reported in Table 4. The average damping estimate for Building 1 is within approximately 10% for the two
methods, despite the comparatively higher variance in HPBW estimates, and remains slightly less than the 1% assumed in design. Moreover for this building, which has the largest amount of data available for analysis, there is some evidence of increasing damping with amplitude, particularly when viewing the RDT results in isolation, which is consistent with the findings of Kijewski-Correa and Pirnia (2007); though admittedly larger amplitude results would need to be observed to further confirm this trend.

In the case of Building 2, mean-sense agreement between the two methods is quite satisfactory: x-sway results are within 10% of one another and y-sway results are identical. While HPBW again shows the greater degree of scatter, the mean-sense agreement of the damping estimates suggests that both approaches were fairly unbiased estimators. Rationale for the comparatively larger damping value on the y-axis was previously offered in Section 5 and in Kijewski-Correa and Pirnia (2007). Again this supports the observation that systems governed by cantilever action have lower damping values and highlights the importance of determining appropriate damping estimates based on structural system as well.

**Table 4**

<table>
<thead>
<tr>
<th>Building 1</th>
<th>Building 2</th>
<th>Building 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>HPBW (%)</td>
<td>RDT (%)</td>
<td>HPBW (%)</td>
</tr>
<tr>
<td>$\zeta_x$</td>
<td>0.91</td>
<td>1.4</td>
</tr>
<tr>
<td>$\zeta_y$</td>
<td>0.87</td>
<td>2.2</td>
</tr>
</tbody>
</table>
as material type. It is also important to note that the amplitudes of response for the events analyzed for Building 2 are comparatively smaller than those of the steel buildings. As discussed in Kijewski-Correa and Pirnia (2007), at low amplitudes, RDT results can bias toward higher damping estimates due to the “noise floor” in low amplitude response data. Once the acceleration levels exceed this “noise floor,” the samples triggered in the RDT process more rapidly converge to a stable Random Decrement Signature to yield more reliable damping estimates. Therefore, the degree of scatter evidenced particularly in Building 2 may be driven by this phenomena and with the availability of more data at RMS acceleration levels of 0.5 milli-g’s or greater, a more definitive estimate of damping and any amplitude dependence can be ascertained.

Similarly in Building 3, the agreement between methods in a mean-sense is quite reasonable, with average critical damping ratios within 15% for x-sway and within 6% for the y-sway. Again, mean-sense is quite reasonable, with average critical damping levels of 0.5 milli-g’s or greater, a more definitive estimate scatter evidenced particularly in Building 2 may be driven by this rapidly converge to a stable Random Decrement Signature to toward higher damping estimates due to the “noise floor” in low Correa and Pirnia (2007), at low amplitudes, RDT results can bias smaller than those of the steel buildings. As discussed in Kijewski-Correa and Pirnia, 2007; Bentz and Kijewski-Correa, 2008). Additionally, there is substantial scatter in the low frequency range, even for RDT, which may again be tied to some of the challenges mentioned previously for low amplitude responses.

6.3. Interpretation of results and implications for the practice

The significant scatter seen in the results can be attributed to numerous factors, including temperature effects, features of the wind induced response, and errors inherent in the analysis techniques, among others. The determination of variability due to site conditions (e.g. temperature effects) is beyond the scope of this study. However, it is important to note that the established analysis techniques used in this paper admittedly have inherent errors that will cause variability in the estimated dynamic properties, particularly in the damping estimates. While it is known that high variability can be seen in the results from such analyses, the methods used in this paper are widely used in practice. Therefore, it is important to acknowledge the limitations of the techniques and the associated impact on the estimates of the dynamic properties particularly when a large number of events are analyzed utilizing bulk processing, as is the case in this paper. The levels of variability have been quantified for both the frequency and time domain approaches used in this study (Kijewski and Kareem, 2002; Pirnia, 2009). The accuracy of the spectral analysis technique used for this analysis is dependent on the smoothness of the PSD. Since HPBW is a point estimator, a jagged spectral peak is known to cause variability in the determined properties, and is particularly notable in the damping estimates. While increasing the number of segment averages does reduce the jaggedness, the bias associated with the fixed spectral resolution does not improve when more than 100 averages are used (Pirnia, 2009). RDT is also known to give higher CoV values when limited amounts of data are available. Averaging a large number of RDSs will reduce the deviation in the results, particularly in the damping estimates. However, there are still variance issues for structures with long periods and low damping such as the buildings monitored in this study (Kijewski and Kareem, 2002; Pirnia, 2009).

Typically full-scale data is presented on specific significant events in publications. This paper also focuses on the presentation of data from many wind events; thus, it was necessary to perform automated bulk processing on the data. While this provides the opportunity to present valuable data obtained over years of monitoring, the automated fashion of the analysis also indicates that the results fail to benefit from analysis by a trained professional. Thus identification of outliers and intricacies in data manipulation normally benefiting from computer analysis coupled with engineering judgment may be overlooked, and significant scatter can occur, in part due to the errors inherent in the analysis techniques, as discussed previously. The results presented must therefore be interpreted in a mean sense, and ongoing work be performed to quantify the variability and to explore additional sources of variability.

6.4. Comparison of predicted and measured response

One purpose of the Chicago Full-Scale Monitoring Program is to compare wind tunnel response predictions with the responses measured in full-scale on a continuous basis under a variety of stationary wind conditions. High-frequency base balance (HFBB) tests for the CFSP were performed at the Boundary Layer Wind Tunnel Laboratory (BLWTL) at the University of Western Ontario (UWO) with the approaches employed described more fully in Kijewski-Correa et al. (2006). These wind tunnel results were used to estimate the response of each of the buildings for the gradient wind speed and direction extrapolated from the NOAA sensor for a given event that triggered full-scale data. An uncoupled analysis was performed for Building 1, while a coupled analysis was required for both Buildings 2 and 3. In both the uncoupled and coupled analyses the Equivalent Static Wind Loads (ESWL) and corresponding building response are determined using the full-scale spectrum of the base moment (SM) and the rms value of the spectrum (σM), calculated as follows using scaling laws and wind tunnel data:

\[
\sigma_M(i) = \left( \frac{f_{ESW}}{f_M} \right) \left( \frac{\sigma_{M}}{f_M} \right) \left( \frac{U}{f_M} \right) \left( \frac{1}{f_M} \right)
\]

where

\[
M = \frac{\sigma_{CM} M}{\sqrt{f_M}}
\]

and \( f \) is the frequency, \( M \) is the reference moment, \( B \) is the building width, and \( U \) is the wind velocity. The resonant moment, resonant ESWL, and rms acceleration for the uncoupled analysis are determined using a mode shape correction factor of 0.7 and the following equations, respectively:

\[
M_R = \sigma_{CM} M \sqrt{\frac{f}{f_M}}
\]

\[
P_R(i) = \begin{cases} 
\sum_{m} m \phi_{m}^{(m)} \phi_{i}^{(m)} & \text{for sway} \\
\sum_{m} m \phi_{m}^{(m)} \phi_{i}^{(m)} & \text{for torsion} 
\end{cases}
\]

\[
\sigma_x = \begin{cases} 
\sum_{m} m \phi_{m}^{(m)} \phi_{i}^{(m)} & \text{for sway} \\
\sum_{m} m \phi_{m}^{(m)} \phi_{i}^{(m)} & \text{for torsion} 
\end{cases}
\]

where \( \sigma_{CM} \) is the rms value of the non-dimensionalized moment spectrum, \( \zeta \) is the damping ratio, \( m(i) \) is the mass at floor \( i \), \( l(i) \) is the moment of inertia at floor \( i \), and \( \phi_{i}^{(m)} \) is the \( j \)-th mode shape at floor \( i \). The coupled analysis becomes more involved as it incorporates the loading cross-spectra determined between different directions estimated from the coupled response of the building. Details of this approach can be found in Chen and Kareem (2005).
The generalized mass and stiffness for each mode are then determined and used to calculate the rms deflection for each mode. The analysis techniques employed are described in greater detail in Bashor (2011). While the in-situ frequency and damping values are used in this analysis in attempt to remove errors due to inaccurate modeling assumptions, as the previous section and previous studies (Bashor et al., 2005; Kijewski-Correa et al., 2006) demonstrated, in-situ damping values are still an uncertain quantity. As a result, a range of damping values was assumed when calculating the accelerations of the buildings from the wind tunnel data. Therefore, upper and lower bounds for wind tunnel predictions are generated to envelope the possible range of predicted accelerations, as detailed in Bashor (2011). The results of analyses for data recorded from Buildings 1, 2, and 3 are presented in Figs. 9, 10 and 11, respectively, as a function of gradient wind speed. As one would expect, there is scatter due to the wide-ranging uncertainties associated with such response predictions, particularly given that on-site wind conditions are not monitored at each of the buildings but rather estimated from the NOAA sensor. Table 5 lists, for each building, the range of gradient wind speeds for which the correlation between in-situ RMS accelerations and wind tunnel predictions is greatest. In the case of Buildings 1 and 2, the wind speeds yielding the best correlation were the same for both fundamental sway responses; however, Building 3 did show sensitivity to the mode in question. While the responses were correctly predicted approximately 25% of the time for all the buildings, in a mean sense, the wind tunnel predictions follow the full-scale observed accelerations with a greater degree of conservatism for the concrete building (Building 2). Furthermore, when reviewing Table 4, it becomes evident that the more dynamically sensitive steel buildings showed greater correlation with wind tunnel predictions at higher gradient wind speeds than the concrete building. Since all of the wind events considered in this study were well below the design gradient wind speed for Chicago (90 mph, 40.2 m/s), the extent to which HFFB measurements should be expected to accurately capture such low amplitude responses is

![Fig. 9. Comparison of full-scale measurements with wind tunnel predictions for Building 1 for y-sway (left) and x-sway (right) as a function of gradient wind speed.](image)

![Fig. 10. Comparison of full-scale measurements with wind tunnel predictions for Building 2 for x-sway (left) and y-sway (right) as a function of gradient wind speed.](image)

![Fig. 11. Comparison of full-scale measurements with wind tunnel predictions for Building 3 for x-sway (left) and y-sway (right) as a function of gradient wind speed.](image)

<table>
<thead>
<tr>
<th>Building</th>
<th>Lower wind speed (mph)</th>
<th>Higher wind speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building 1</td>
<td>40 (17.9 m/s)</td>
<td>50 (22.4 m/s)</td>
</tr>
<tr>
<td>Building 2</td>
<td>26 (11.6 m/s)</td>
<td>35 (15.6 m/s)</td>
</tr>
<tr>
<td>Building 3 (x-Sway)</td>
<td>41 (18.3 m/s)</td>
<td>45 (20.1 m/s)</td>
</tr>
<tr>
<td>Building 3 (y-Sway)</td>
<td>51 (22.8 m/s)</td>
<td>55 (24.6 m/s)</td>
</tr>
</tbody>
</table>
somewhat questionable, as noted previously in Kijewski-Correa and Kochly (2007). Ongoing monitoring will attempt to validate the hypothesis that wind tunnel predictions will show greater degrees of correlation for design level events.

7. Concluding remarks

As the design of tall buildings is often governed by the habitability limit state — accelerations that adversely affect occupant comfort — there is considerable need to validate the processes used to predict RMS accelerations in the design stage. The Chicago Full-Scale Monitoring Program has been focused on such validations by observing the in-situ behaviors of three tall buildings whose systems are common to high rise design for a decade. This paper presented a cross section of that data to first extract estimates of dynamic properties and then to use those to predict RMS acceleration levels. The in-depth analysis of two selected wind events from the database was first presented in order to evaluate the performance of the system identification techniques used in this study. While the time and frequency domain techniques produced consistent natural frequency estimates, more considerable deviations were observed in the damping values, though these were not due to bias but rather random errors. In particular, variance errors in power spectral estimates rendered the resulting critical damping ratios less reliable than those from Random Decrement Signatures, particularly for the more narrow-bandded steel buildings. It was found that structural systems with greater degrees of frame action tend to yield higher levels of damping than systems that are dominated by cantilever action, consistent with the findings of previous studies, and that such systems tend to be more difficult to accurately model in commercial finite element packages. The dynamic properties of the three buildings were then determined for several hundred storms to evaluate trends in the dynamic properties over a range of amplitudes. It was found that the dynamic properties estimated from an automated analysis suite for the bulk processing of several hundreds of events showed significant scatter, partially due to the inherent errors in the analysis techniques and the lack of identification of outliers using human judgment, and must therefore be considered in a mean sense. The discussion of additional sources of error is beyond the scope of this paper and is being explored by the authors. Clear evidence of amplitude dependence and softening with increased response was displayed for all three buildings, consistent with initial observations from a narrower subset of CFSPM data. While damping values showed expected scatter, particularly for the frequency domain identification approach, agreement in the mean sense between methods was quite reasonable for all three buildings and affirmed that the study’s purest steel tube had the lowest level of damping while the study’s concrete building had the highest damping, though less damping was observed on the axis where outriggers are employed. In fact, the systems with panel zones and comparatively greater reliance on distributed frame action tended to produce higher damping values.

When the in-situ frequency and damping values were used to predict the RMS accelerations of the buildings, considering the potential range of these uncertain parameters, responses were predicted accurately only 25% of the time for all three buildings. While there is expected scatter, particularly given that on-site wind conditions are not monitored at each of the buildings but rather estimated from the NOAA sensor, the wind speeds over which the predictions are most accurate were noted for each building and tend to be higher for the steel buildings. However, since all of the wind events considered in this study were well below the design gradient wind speed for Chicago, the extent to which HFFB measurements should be expected to accurately capture such low amplitude responses is somewhat questionable. The program’s continued monitoring of these buildings will enable the observation of even higher amplitude events to further vet the hypotheses posted here surrounding the amplitude-dependence of their dynamic properties and the ranges of wind speed over which HFFB methods prove most effective in predicting RMS accelerations to inform habitability design of tall buildings.

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