Full-Scale Performance Evaluation of Tall Buildings Under Winds

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ABSTRACT: The Chicago Full-Scale Monitoring Program has been actively monitoring the acceleration and displacement responses of several tall buildings since 2002, collecting valuable data on the performance of these common structural systems under the action of wind. The major thrust of this work has been developing capabilities for real-time monitoring of tall buildings and site wind speeds during significant events to assess performance and provide a comparison of actual response with computational/analytical/simulated predictions utilizing database-assisted schemes. This study overviews the major findings of this program, current activities, and new directions as the program expands to include additional tall buildings worldwide. Specifically the role of amplitude-dependent dynamic properties will be highlighted, as will the role of panel zone effects in finite-element modeling of wind-sensitive structures.

KEYWORDS: Dynamic Responses, Wind Tunnel Tests, Acceleration, Damping, Full-Scale Monitoring, High-Rise Design.

1 INTRODUCTION

The Chicago Full-Scale Monitoring Project, a collaborative effort between the University of Notre Dame, the design firm of Skidmore, Owings & Merrill LLP (SOM), and the Boundary Layer Wind Tunnel Laboratory (BLWTL) at the University of Western Ontario, is currently in its second phase. The main goal of its first phase was to evaluate the performance of high-rise buildings under wind loading by comparing their measured and predicted responses, generated through the use of commercially available finite element codes (FEM) and state-of-the-art wind tunnel testing. Through the use of web-based data collection/archiving (windycity.ce.nd.edu) and advanced instrumentation, including accelerometers, global positioning systems (GPS), and anemometers, the in-situ wind-induced response levels of three tall buildings in Chicago have been investigated for nearly five years. These buildings represent different common structural systems: Building #1 (B1) is a steel, stiffened tubular structure; Building #2 (B2) is reinforced concrete (RC) with a shear wall/outrigger system; and Building #3 (B3) is a steel tubular system.

As demonstrated in Kijewski-Correa *et al.* [1], the resonant response levels of the buildings have been correlated against wind tunnel predictions. While showing general consistency of trend, the observed responses manifest considerable scatter that may be in part the result of on-site wind speed uncertainty. For this reason, the establishment of rooftop anemometry and a means to relate surface winds to gradient winds within the urban zone have been undertaken, as discussed in Section 2. Still, the existing wind monitoring protocol has allowed GPS verifications of wind tunnel response predictions for not only the resonant response but also the background component [2].

While the correlation between predicted and measured response is certainly of interest, it is directly related to the accuracy with which dynamic properties are estimated in design. With respect to damping, while the values reported to date appear to be consistent and even conservative with respect to common assumptions [1], the effect of amplitude dependence was not investigated and is now addressed in Section 3. On the other hand, previous comparisons of in-situ periods to those predicted by finite element models used in design highlighted that the reinforced concrete building (B2) was notably stiffer than the model prediction, while the steel tubular system's (B3) lateral modes were approximately 10% softer than the model prediction. (Note that Building #1 was modeled with remarkable accuracy by FEM). While uncertainties surrounding the in-situ properties of concrete, including the extent of cracking, and the degree of participation of various gravity elements in the lateral resistance may help to explain the discrepancies noted in Building #2, additional parametric studies were required to suggest possible reasons for the softer in-situ periods in Building #3. These findings are discussed in Section 4 followed by an overview of the program's second phase and recent informal assessments of occupant comfort in a large population of tall buildings.

2 WIND CLIMATE

While wind speed and direction are monitored regularly at Chicago's surrounding airports, it is essential to have a reliable measure of wind speed and direction in the downtown area. Two ultrasonic anemometers, the Vaisala WAS425 and FT Technologies FT702, were installed on masts 41 m above the rooftop of Building #3 so that the reference wind speed and direction could be measured on-site. These anemometers are at a record height in an urban area, posing unique challenges including exposure to severe thunderstorms, radio frequency (RF) fields and potential rooftop aerodynamic interferences. Prior to their installation and validation, an interim wind monitoring protocol was established to permit the use of data collected from a NOAA Great Lakes Environmental Research Laboratory (GLERL) meteorological station in Lake Michigan [1]. The NOAA GLERL surface data was extrapolated to an arbitrary reference height of 500 m and compared to the extrapolations of surface winds from the Midway and O'Hare Airports using the BLWTL wind climate model for Chicago and were found to be reasonably consistent.

On February 25, 2004, the program began collecting statistics and time histories of the upper level wind data at Building #3, including several episodes of high winds. An example of the data comparison over a one week period with relatively strong winds is shown in Figure 1, where 10-minute average wind speeds and the turbulence intensity measured by the FT702 are plotted. The wind speed measurements from the FT anemometer are reasonably consistent with the NOAA GLERL meteorological station only over select intervals. This is perhaps due to RF interference, suggested by the change in the turbulence intensity. As part of the program's second phase, addi-

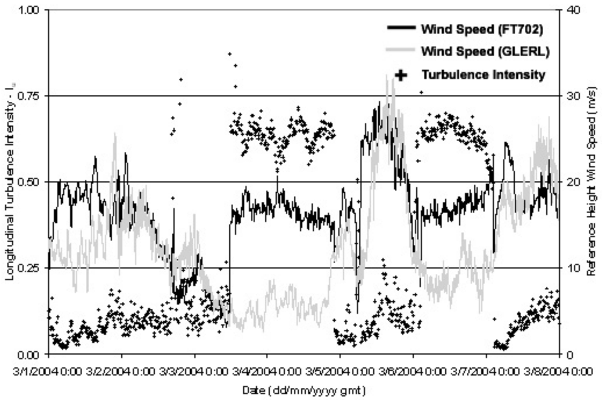

Figure 1. FT702 upper level wind speeds (and turbulence intensity) versus extrapolated NOAA GLERL data.

tional anemometers are being installed on the top of Building #3 and other buildings in the study to permit a more reliable verification of surface wind extrapolations.

3 ROLE OF AMPLITUDE-DEPENDENT DYNAMIC PROPERTIES

The Random Decrement Technique (RDT) is next employed to extract amplitude-dependent properties associated with the fundamental x-sway, y-sway and torsional responses of each of the three instrumented buildings, following the framework outlined in Pirnia *et al.* [3]. The framework involves the use of RDT with peak trigger conditions over a range of amplitudes unique to each building, a local averaging approach constrained to +/-3% of the desired trigger to reduce variability, and fitting of the RDT decay curve by an analytic signal approach over its first three cycles of oscillation to insure accurate referencing back to the trigger amplitude. Only RDT estimates resulting from a minimum of 250 averages were retained herein. Viable stationary datasets were identified for each building by targeting wind conditions known to generate sustained response in each building. These are summarized in Table 1. Acceleration responses under these conditions were subjected to the aforementioned RDT framework. The resulting amplitude-dependent frequency curves are omitted in the interest of space. Since all showed a linear decay with amplitude, they were approximated by a best-fit straight line and summarized in Table 2 for brevity. As most of the buildings are relatively stiff torsionally, the torsional response is quite modest with respect to the ambient noise floor and parameters estimated from it may have questionable reliability. Although shown for completeness, the torsional parameters are not discussed at this stage as the very low amplitudes prohibit adequate extraction of the torsional response from the ambient noise floor.

Table 1. Summary of wind events considered

	Direction		Speed (m/s)		Data Quantity
	Mean	Range	Mean	Range	(Hours)
Building #1	231° (WSW)	229°-234°	9.0 m/s	4.5-12.0	156
Building #2	270° (W)	266°-275°	10.5 m/s	7.0-12.5	72
Building #3	289° (WNW)	286°-295°	10.7 m/s	5.6-14.0	72

Note: 5-minute statistical averages (surface level) of NOAA GLERL data [1]

3.1 *Building #1*

As shown in Table 2, the two lateral responses show only a slight amplitude-dependence in frequency, with the rate of softening being less than 2% of the initial stiffness. With respect to damping, the lateral responses show a remarkable consistency over the trigger amplitudes considered, with a discernable increase in damping with amplitude approaching the 1% value commonly assumed in steel construction (Fig. 2a).

Table 2. Summary of linear representations of frequency amplitude-dependence

	X-Sway (Hz) [R^2]	Y-Sway (Hz) [R^2]	Torsion (Hz) [R^2]
Building #1	-0.0034x + 0.2078 [0.92]	-0.0019x + 0.1438 [0.94]	-0.0186x + 0.5059 [0.74]
Building #2	-0.0062x + 0.1827 [0.91]	-0.0204x + 0.1854 [0.96]	-0.3251x + 0.3110 [0.93]
Building #3	-0.0030x + 0.1200 [0.88]	-0.0029x + 0.1200 [0.71]	-0.0556x + 0.2319 [0.82]

Notes: Best-fit lines project frequency in Hertz as a function of acceleration (x) in milli-g's.
R^2 [in brackets] is a measure of error in linear approximation.

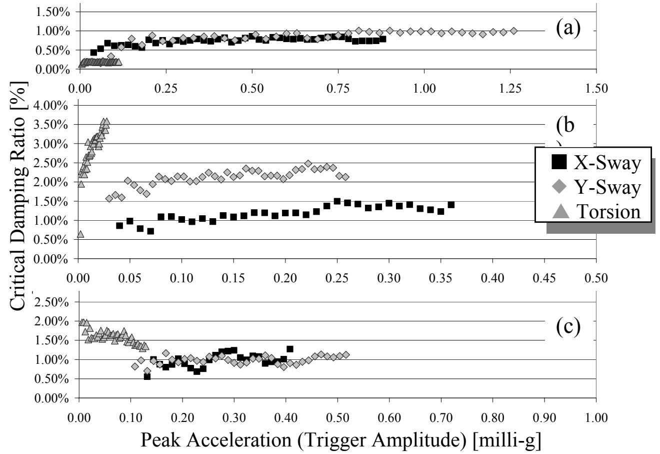

Figure 2. Amplitude-dependent critical damping ratios for Buildings (a) #1, (b) #2 and (c) #3.

3.2 *Building #2*

A markedly stronger degree of amplitude dependence is noted in this building, particularly for the y-sway response, with the rate of softening of 11% the initial stiffness (Table 2). Note that the shear wall area is greater in the x-direction, while the lateral resistance of the y-direction is derived primarily from frame action. As will be further supported by the findings for Building #3, modes characterized by a greater degree of frame action (vs. cantilever) tend to manifest greater amplitude dependence. These observations are echoed in the damping estimates shown in Figure 2b, where damping values are comparatively larger for the y-response and actually exceed the 2% level often assumed for RC structures. This is consistent with the findings in Erwin *et al.* [4], who also observed comparatively lower damping values for reinforced concrete systems governed by cantilever action, as is the case for the x-direction response in Building #2. The differing levels of damping observed between the two lateral responses underscores the importance of linking assumed damping levels to structural system characteristics and not material type alone.

3.3 *Building #3*

As Building #3 is generally characterized by “symmetry” in its lateral modes [1], it is not surprising to see both lateral responses manifesting the same softening rate of 2.5% the initial stiffness (Table 2). Interestingly, despite also being steel in nature, the amplitude dependence in sway period is nearly doubled in this building in contrast with Building #1. Furthermore, damping values show a far greater degree of fluctuation in both this building and Building #2, in comparison to Building #1. This would seem to underscore a greater degree of amplitude-sensitivity in these buildings, both of which are known to have comparably greater frame action in their overall response, as well as varying degrees of coupling [1]. (However, it should be acknowledged that the stability noted in Building #1’s damping estimate may also be the result of the significantly greater amount of data available for this building, as shown in Table 1.) It was previously hypothesized in Kijewski-Correa *et al*. [1] and reiterated in Erwin *et al.* [4], that buildings with greater proportion of frame action demonstrate comparatively larger damping values. Thus while both Building #1 and #3 are steel, the relative contributions of “cantilever” and “frame” deformation mechanisms to their overall response may indeed offer an explanation of their differences.

3.4 *Comparisons to Previously Published Values*

All three buildings had been analyzed previously for other wind events using stationary analyses by both RDT and half power bandwidth (HPBW) method [1, 5]. These are now compared to the present amplitude-dependent RDT results averaged over all amplitudes considered, the FEM predicted periods, and the assumed damping levels for serviceability design. Again, as torsional responses are modest, their damping values in particular may not be reliable and are not discussed herein. As evidenced by Table 3, the amplitude-sensitive analyses result in only slightly stiffer average periods. (Rationale for deviations between in-situ and FEM periods will be addressed in the following section). With respect to damping, it is widely known that PSD results will be somewhat larger than RDT results due to inherent spectral bias. However, as first observed in Pirnia *et al.* [3], frequency variability may generate larger damping estimates when spectral methods are applied. Variability in the observed damping values in Table 3 results not only from this, but also due to the amplitude dependence of damping itself. The most repeatable damping values have been observed for Mode 1 of Building #1, which showed the least amount of amplitude-dependence in Figure 2a. Again the dominant trend with respect to damping is that responses governed by comparatively more cantilever action (Building #1, Building #2 x-axis) have lower damping values.

4 FINITE ELEMENT MODELING: THE ROLE OF PANEL ZONE EFFECTS

For tall, slender buildings and those lightly damped, motion perception often becomes the governing design criteria. Therefore, it is critical that the engineer be able to accurately predict the full-scale behavior of the structure by means of analytical representation through FEM. As discussed previously, results show very reasonable correlation between measured and predicted building frequencies; however, some differences in dynamic properties have been noted. In particular, the discrepancies related to buildings of concrete construction are dependent upon the post-cracking stiffness of the concrete lateral system elements under service level loads. Other influencing factors include the level of interactivity between reinforced concrete floors, exterior columns, and core elements, as well as in-situ material property variation, i.e., elastic modulus. Meanwhile in the case of steel frame

buildings, the roles of panel zone stiffness and beam/column frame connectivity have been investigated as potential sources for increased system flexibility [6]. In particular, for frames with relatively closely spaced columns and deep spandrel beams, panel zone effects can contribute significantly to overall system deformations, even within the elastic range. A finite element analysis of the influence of panel zone deformations on the period of vibration is now presented. This analysis uses the scissors model proposed by Charney & Marshall [7], which quantifies the elastic shear stiffness of the panel zone. Modal FEM analyses presented herein reveal that the inclusion of panel zone effects yields FEM periods that are more consistent with in-situ observations for Building #3. While Building #3 is categorized as a moment-connected framed tubular structure, its deformation characteristics vary somewhat from that of a traditional moment resisting frame (MRF). In fact, this building deforms through a combination of frame shear and column axial deformations, as underscored in the preceding section.

Table 3. Comparison of periods and critical damping ratios

	Mode 1		Mode 2		Mode 3	
	Period (s)	Damping (%)	Period (s)	Damping (%)	Period (s)	Damping (%)
Behavior	Y-axis translation		X-axis translation		Torsion	
B1: FEM [1]	7.0	1.0	4.9	1.0	2.0	1.0
B1: HPBW[a]	7.1	1.1	4.9	0.6	2.0	0.7
B1: RDT[a]	7.1	0.9	4.9	0.9	2.0	0.9
B1: HPBW[b]	7.1 s	0.9	4.9	1.4	2.0	0.7
B1: RDT[b]	7.1 s	1.0	4.9	0.9	2.0	0.5
B1: RDT[c]	7.0 s	0.8	4.8	0.7	2.0	0.2
Behavior	X-axis translation, slight torsion		Y-axis translation, slight torsion		Torsion	
B2: FEM [1]	6.7 s	1.0	6.4	1.0	4.6	1.0
B2: HPBW[a]	5.6 s	1.6	5.6	2.1	3.4	3.1
B2: RDT[a]	5.6 s	1.4	5.7	2.4	3.4	3.6
B2: HPBW[b]	5.6 s	1.7	5.7	2.5	3.4	3.3
B2: RDT[b]	5.6 s	1.5	5.7	2.9	3.5	4.2
B2: RDT[c]	5.5 s	1.2	5.5	2.1	3.3	2.9
Behavior	Coupled translation (X)		Coupled translation (Y)		Torsion	
B3: FEM [1]	7.7 s	1.0	7.6	1.0	4.5	1.0
B3: HPBW[a]	8.6 s	1.5	8.6	1.1	4.5	1.3
B3: RDT[a]	8.6 s	1.0	8.6	1.2	4.3	1.3
B3: HPBW[b]	8.5 s	1.6	8.5	2.0	4.5	1.3
B3: RDT[b]	8.5 s	1.0	8.6	1.4	4.5	1.5
B3: RDT[c]	8.5 s	1.0	8.3	1.0	4.4	1.6

[a] April 28, 2004 wind event: average wind speed/direction = 20 m/s, 225° [1]
[b] March 7, 2004 wind event: average wind speed/direction = 23.8 m/s, 288° [5]
[c] Averaged over amplitude ranges considered in this study for the events in Table 1

A modal analysis was performed through ETABS v. 8.4.4 in order to determine the effects of panel zone shear deformation on overall frame stiffness of Building #3. For comparison, the results of modal analyses are shown in Table 4 for three cases: a centerline model (CLM), a model with partial rigid offsets (PRM) at the beam-to-column joint equal to 25% of the true panel zone dimensions, and a revised scissors panel zone model (PZM) utilizing rotational springs to represent panel zone shear stiffness (Fig. 3). Note that the PRM result was the design prediction pre-

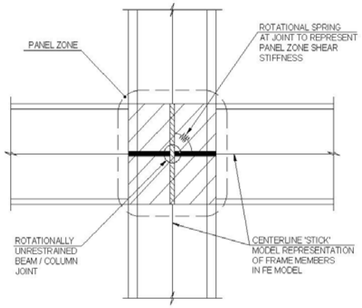

Table 4. FEM and in-situ periods in Building #3

	CLM (s)	PRM (s)	PZM (s)	In-Situ[a] (s)
Mode 1	8.0	7.7	8.2	8.5
Mode 2	7.9	7.6	7.8	8.3
Mode 3	4.7	4.5	4.9	4.4

[a] Average frequency of amplitude-dependent analysis

Figure 3. Beam/column joint with scissors mechanism.

viously reported in Table 3 and in Kijewski-Correa *et al.* [1]. These results are compared to the average period extracted from the preceding amplitude-dependent analysis.

For the relatively closely spaced columns and deep spandrel beams associated with Building #3, explicit modeling of the shear component of panel zone deformations has proven to increase predicted frame flexibility somewhat beyond what was predicted with the centerline model. Although the centerline model had not considered panel zone deformation explicitly, it had overestimated member flexural deformations, which initiate at the member centerline joint.

Note that the scissors model requires that fully rigid links be introduced at the beam-to-column panel zone for proper calibration of the panel zone rotational spring. Therefore, axial and flexural deformations will not occur within this fully-rigid panel zone within the FEMl. Depending upon the geometry of the beam-to-column joint and the height of the building, these deformations could contribute significantly to the in-situ overall frame flexibility. This is especially true for moment-connected framed tubular buildings of considerable height, where frame flexibility may be dominated by column axial deformations. It is anticipated that axial deformation within the panel zone may contribute significantly to the overall frame deformation of Building #3 under lateral loads. Although the effects of axial deformations through the panel zone are currently being studied, the results in Table 4 still reiterate the importance of explicit treatment of panel zone shear deformations for more accurate predictions of response.

5 NEW DIRECTIONS IN WINDYCITY II

Given the sensitivity of building response to structural system and material type, uncertainty in correlating reference surface winds to rooftop winds in urban zones, and a lack of information concerning human sensitivity to motion in actual building environments, a second phase of the program has been launched. This phase involves additional buildings to expand the range of structural systems being monitored, a computational fluid dynamics element to establish and validate the extrapolation of surface winds to other elevations, and a post-storm survey of building occupants to formulate improved comfort criteria. Consistent with the original, ongoing monitoring program, the proposed research will also measure in-situ dynamic characteristics and validate the predictive models used in high-rise design.

The generation of a more realistic habitability criteria has been informally initiated through a web-based survey (www.nd.edu/~tallbldg/survey.html), developed in concert with international researchers experienced in occupant perception surveys [8], in order to collect more anecdotal evidence on human responsivity to motion in tall buildings. The survey has been circulated to various organizations involved in the design and management of tall buildings worldwide and not specifically directed toward the buildings monitored in this program. Physical responses to a number of wind events have been noted for buildings across the country, resulting in symptoms such as headaches, dizziness, and nausea.

The authors received numerous responses to a specific wind event. This provides an excellent opportunity to evaluate the range of experiences of several persons occupying the same building during the same event. The respondents were standing or walking during the event, with 70% of them looking out the window at the time they first perceived the motion. While 60% sensed motion in some form, only 10% indicated strong perception. Interestingly, while 20% were first alerted by visual cues, 80% did acknowledge the role of others in cuing their own perception. This factor is often not captured in controlled laboratory testing. No respondents experienced any ill effects, though they were only subjected for a brief time.

6 CONCLUSIONS

This paper summarizes recent efforts directed toward validation of tall building response characteristics under wind. Major findings from the first phase of this monitoring program were presented and future activities are detailed, including informal assessments of occupant comfort in tall buildings using web-based surveys. Specifically, challenges associated with monitoring wind fields in urban environments were underscored, as was the role of amplitude-dependence, deformation mechanism, and panel zone shear deformations for accurate representations of dynamic properties.

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