DYNAMIC BEHAVIOR OF TALL BUILDINGS UNDER WIND:
INSIGHTS FROM FULL-SCALE MONITORING

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SUMMARY
The wind-induced response of tall buildings is inherently sensitive to structural dynamic properties like frequency
and damping ratio. The latter parameter in particular is fraught with uncertainty in the design stage and may result
in a built structure whose acceleration levels exceed design predictions. This reality has motivated the need to
monitor tall buildings in full-scale. This paper chronicles the authors’ experiences in the analysis of full-scale
dynamic response data from tall buildings around the world, including full-scale datasets from high rises in
Boston, Chicago, and Seoul. In particular, this study focuses on the effects of coupling, beat phenomenon, ampli-
tude dependence, and structural system type on dynamic properties, as well as correlating observed periods of
vibration against finite element predictions. The findings suggest the need for time–frequency analyses to identify
coalescing modes and the mechanisms spurring them. The study also highlighted the effect of this phenomenon
on damping values, the overestimates that can result due to amplitude dependence, as well as the comparatively
larger degree of energy dissipation experienced by buildings dominated by frame action. Copyright © 2007 John
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1. INTRODUCTION
Numerous industries have recognized the value of instrumentation and monitoring to improve their
product’s performance. The reason: despite all the technological advances of modern computational
tools and even experimental capabilities, there is no truer laboratory than full-scale. Sadly, the US
structural engineering community has not been able to follow suit, largely driven by the legal and
fiscal concerns of building ownership tied to the lingering public perception that a monitored building
is somehow a troubled building (Kijewski-Correa and Kareem, 2007). Although these attitudes have
changed in California with the advent of the California Strong Motion Instrumentation Program, this
was largely facilitated by municipal backing and the public’s acceptance that buildings do move and
therefore require monitoring to ensure rapid and safe reoccupation in the wake of earthquakes. Outside
of seismic country, structures are generally governed by serviceability and habitability limit states and
the public perception of acceptable dynamic behavior is quite different.

However, one can readily argue that the need for full-scale monitoring of structures under service
loads is equally important. Specifically, in the case of tall buildings, limiting motion perception by
building occupants (habitability) is often a controlling structural engineering design parameter for tall
buildings, even in moderate wind climates. Significant premium for height in terms of additional

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structural material may become necessary in order to satisfy current acceleration-based motion criteria, beyond that required to meet minimum standards for building strength or lateral drift serviceability criteria related to building partitions or other architectural systems such as cladding and facades (Kijewski-Correa et al., 2006a). The predictions of these accelerations are highly dependent on the dynamic properties of the building, as shown by the following expression used to predict the RMS lateral acceleration from wind tunnel data for a given mode:

\[
\sigma_x = (2\pi f)^2 \frac{\sigma_{\nu}}{K^*} \phi(z) \sqrt{\frac{\pi}{4\zeta} \frac{f S_{\nu}(f)}{\sigma_{f}^2}}
\]  

(1)

Here \(f\), \(\zeta\), \(\phi\) and \(K^*\) are respectively the natural frequency, critical damping ratio, mode shape, and generalized stiffness; \(\sigma_{\nu}\) and \(S_{\nu}(f)\) are respectively the generalized force’s variance and value of its power spectral density (PSD) at the natural frequency.

Uncertainties in finite element modeling of the lateral system can affect a number of terms in this expression, such as \(f\), \(\phi\), and \(K^*\) and thereby \(S_{\nu}(f)\). While the modeling of some systems like steel tubes appears to be straightforward (Kijewski-Correa et al., 2006a), the same cannot be said for steel buildings with substantive panel zone effects (Kijewski-Correa et al., 2007) and reinforced concrete buildings, which are influenced not only by assumed levels of cracking but also the degree of participation of gravity elements in the overall lateral resistance (Erwin et al., 2007). Still, armed with full-scale data, fundamental structural mechanics and modern finite element packages provide a suitable venue to refine our understanding of these features. However, there is not a convenient means to refine our predictive capabilities regarding inherent structural damping, owing to its association with a number of complex mechanisms and even non-structural elements. This situation is unfortunate since, of all the dynamic properties that can be modified, increases in damping are the only ones that have consistently proven to reduce accelerations. Although there have been some efforts to develop empirical predictive tools for damping estimation based on full-scale observations (Jeary, 1986; Satake et al., 2003), there is still significant scatter in the data, as well as limited information for high-rise buildings, which tend to be the most sensitive to acceleration-based perception criteria. This scatter is in part due to the relatively minor participation of damping in overall structural response. Thus, without expansive databases populated by reliable damping estimates encompassing various structural systems, foundation types and materials, a reliable predictive model for damping cannot be achieved to permit accurate estimates of displacement and acceleration in the design phase. Given the importance of dynamic properties in response prediction and the ability to meet habitability limit states critical to the design of tall buildings, this study will explore the insights into dynamic behavior offered by full-scale monitoring under wind, investigating in situ values of damping and the factors which influence it, as well as comparison between in situ dynamic properties and those realized in the design stage.

2. OVERVIEW OF MONITORING PROGRAMS

In this paper, insights into the dynamic behavior of tall buildings will be drawn from three full-scale datasets, which are now described. In all three cases, ownership requested anonymity of the buildings involved. As such the Boston building will be referred to as B1, the Seoul building will be referred to as S1, and the three Chicago buildings will be referred to as C1, C2, and C3.

2.1 Boston building

A 5-year full-scale monitoring program was initiated between 1973 and 1978 on B1, an 800-foot (245·7 m) steel-framed building in Boston, following failure of some building envelope components...
(Durgin and Gilbert, 1994). The extensive monitoring program was initiated by researchers at Massachusetts Institute of Technology (MIT) and included the collection of wind velocity up to 100 feet above the rooftop and pressures and accelerations at a number of locations, as analyzed in greater detail in Brown (2003). The dataset was archived in analog and hard copy and was transferred to the University of Notre Dame, where it was digitized for analysis. The accelerometer data, which are of primary interest in this paper, were acquired at eight locations in the building, with four sensors located on the 57th floor and another four at the 35th, with the exact placements shown in plan view in Figure 1. The two sensors measuring the motion parallel to the longer face of the building were termed the ‘NS’ sensors, while the sensors capturing motions parallel to the building’s shorter axis were termed ‘EW’. The sensor pairs are positioned at opposite corners of the building plan and named based on the column they are associated with, i.e., 2 or 59, as shown in Figure 1.

2.2 Seoul building

S1 is a 73-story tower (264 m) located in Seoul, Korea, as overviewed in Abdelrazaq et al. (2005). The lateral load-resisting system of the tower consists of a high-performance reinforced concrete core wall system, running from the foundation to the roof, which was linked to the exterior composite columns by an indirect outrigger belt wall system at the mechanical levels (16 to 17, and 55 to 56). The interaction of the core wall system and the exterior columns was provided through deformation compatibility, resulting in significant forces in the belt wall system components, which included the belt wall and the floor slabs. The tower has been equipped with six Wilcoxon 731A/P31 seismic accelerometers mounted in orthogonal pairs to the girders at three locations on the 64th floor (64F), as shown in Figure 2, which will be referred to as Locations 1, 2 and 3 in subsequent discussions. This information is supplemented by the on-site wind velocity measured by an FT Technologies FT702 ultrasonic anemometer mounted atop a 2 m mast at the north end of the building’s rooftop. This system has been acquiring data since April 2, 2005.

Figure 1. Plan view of B1 with accelerometer locations and orientations, along with mean wind direction for 10 March 1974 event
2.3 **Chicago dataset**

Despite many of the aforementioned roadblocks, the author and her collaborators at the University of Notre Dame, Skidmore Owings and Merrill LLP and the Boundary Layer Wind Tunnel at the University of Western Ontario were also successful in establishing what may be the longest full-scale monitoring program for tall buildings in the United States, outside of a seismic zone. The Chicago Full-Scale Monitoring Program (http://windycity.ce.nd.edu) has been conducting systematic validations of the performance of three tall buildings in Chicago since 2002 (Kijewski-Correa et al., 2006a). Each instrumented building utilizes straight-shaft reinforced concrete caissons extending to bedrock. A brief description of noteworthy features of each building’s structural system is now provided, with additional structural details available in Kijewski-Correa et al. (2006a).

- **C1:** The primary lateral load-resisting system features a steel tube comprised of exterior columns, spandrel ties and additional stiffening elements to achieve a near-uniform distribution of load on the columns across the flange face, with very little shear lag. As such, lateral loads are resisted primarily by cantilever action, with the remainder carried by frame action.

- **C2:** In this reinforced concrete building, shear walls located near the core of the building provide lateral load resistance. At two levels, this core is tied to the perimeter columns via reinforced concrete outrigger walls to control the wind drift and reduce overturning moment in the core shear walls.

- **C3:** The steel moment-connected, framed tubular system of C3 behaves fundamentally as a vertical cantilever fixed at the base to resist wind loads. The system is comprised of closely spaced, wide columns and deep spandrel beams along multiple frame lines. Deformations of the structure are due to a combination of axial shortening, shearing and flexure in the frame members, and beam–column panel zone distortions.

Each building is equipped with the same primary instrumentation system that features four Columbia SA-107 LN high-sensitivity force balance accelerometers mounted in orthogonal pairs at two opposite corners of the ceiling at the highest possible floor in each building. The primary instrumentation systems were respectively installed in C1, C2, and C3 on April 14, 2002, April 15, 2002, and
April 30, 2003. C3 was subsequently equipped with a pair of ultrasonic anemometers installed on masts 41 m above the rooftop. Finally, in order to recover both the background and resonant components of the dynamic response, a high-precision global positioning system (GPS) was added to C1 on August 26, 2002 (Kijewski-Correa et al., 2006b). The instrumentation is shown schematically in Figure 3.

3. FULL-SCALE DATA ANALYSIS

While the comparison of full-scale accelerations and displacements to wind tunnel predictions is certainly of interest and can be found in a pair of recent publications (Kijewski-Correa et al., 2006a; Kijewski-Correa and Kochly, 2007), this study will focus instead on inherent structural dynamic properties of tall buildings and the insights gained from rare full-scale data. The discussion begins with an investigation of the influence of coupling on energy dissipation (damping) in tall buildings, followed by the amplitude dependence of dynamic properties and their reliance on structural system type.

3.1 Influence of coupling on energy dissipation

The Boston building (B1) represents an excellent example of the detrimental effects of structural coupling and beating of modes lateral and torsional modes. The analysis begins by generating the wavelet marginal power spectra, analogous to the Fourier power spectra, from time histories recorded at each of the 57th floor accelerometers during the March 10, 1974 wind event (Figure 4). Over the

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1Wavelet analysis is a time-varying transform that provides information on frequency content as it evolves in time. Elementary references on the transform can be found from a number of textbooks, though a detailed description of the analyses used by the first author can be found in her dissertation (Kijewski-Correa, 2003).
course of the hour-long excerpt being considered, the mean wind speed, measured at 100 feet above the rooftop, was 53.7 mph, gusting to 77.3 mph, primarily out of the west-northwest at 290°. The spectra in Figure 4(b and d) manifest two modes: the softer of the two is the NS fundamental sway at 0.132 Hz, while the larger peak is associated with the torsional mode at 0.160 Hz. Note interestingly that in Figure 4 the EW sway mode is oddly not present, with only dominant torsion detected. However, an investigation of the larger dataset was able to identify the EW sway component in other records and found that the fundamental modal frequencies from lowest to highest are: NS sway, EW sway, and finally torsion (Brown, 2003). This indicates that all three frequencies lie in within a span of 0.021 Hz, with a potentially strong possibility for beating between the EW sway and torsion. Furthermore, the spectra associated with this event reflect that motion at the corners of the structure is dominated by torsion, which is a disturbing feature since these are the most perceivable motions from an occupant comfort perspective.

Figure 4(a) provides the most interesting point of discussion in this analysis. Here at the windward face of the building the same torsional frequency that was found at #02-EW is not observed, but rather what appears to be a softened version (0.153 Hz). Hence, the spectra for the EW sensors would

![Figure 4. Recorded acceleration time histories and marginal wavelet power spectra for B1 during 10 March 1974 wind event](image)
indicate that the building is twisting at two different frequencies. Further, it is at this windward sensor that the largest amplitudes of motion are detected, indicating that the structure is effectively fishtailing in the wind due to an eccentricity between its mass and elastic centers. Thus the building’s response in both amplitude and frequency varies over the building’s plan, an intriguing feature that will be explored utilizing wavelet instantaneous information.

Wavelets provide one of the few viable strategies to uncover intermittent response features and the evolution of coupled responses that depend greatly on the turbulent structure of the wind. In fact, the relative contributions of each mode, particularly the torsional, may suddenly increase for the same mean wind speed due to the instantaneous pressure imbalances on the face of the building. As this is a time-varying phenomenon, it cannot be identified through classical Fourier spectral techniques. However, the use of instantaneous wavelet spectra provides snapshots of the frequency content at distinct times in the response. This approach is applied to the measurements at sensor #59-EW (Figure 5), where the time instants being analyzed are shown by the numbered lines and the corresponding wavelet ‘spectral snapshots’ are shown below. Note that the highest amplitude response

![Graph showing wind speed, direction, and response](image.png)

**Figure 5.** Wind speed, direction and response of B1 on 10 March 1974 at sensor #59-EW with wavelet instantaneous spectra and inset value of peak frequencies
occurs when the wind direction shifts and aligns with the corner of the building (270°), commensurate with the peak wind speed during this event. As the wind shifts back toward 290°, the response level subdues to an extent. It is just prior to and following this highly energetic burst that the single mode response, identified earlier in Figure 4(a) as 0.153 Hz, is observed. Note the large bandwidth associated with this spectral peak, broadening toward the high frequencies, suggestive of a coalescence of two adjacent modes. The wide bandwidth of the response in conjunction with the wind direction supports the conclusion that both EW sway and torsional responses are sufficiently large and simultaneous in the response at this moment, leading to a melding of spectral bandwidth in a beating phenomenon. This situation can be particularly energetic if both response components are of comparable amplitude. In spectra 1–3, this strong vibration is stimulated by the wind direction shift, proving critical for this building’s geometry within the site-specific surroundings. As the amplitude increases in the 3rd, 4th, and 5th spectra, a shift of this spectral peak to a lower frequency of 0.148 Hz occurs. This is the characteristic softening of frequency that has been observed throughout this dataset (Kijewski and Kareem, 1999; Kijewski et al., 2003) and in other full-scale measurements, as the contacts between structural members diminish and greater slip in joints reduces stiffness. At lower amplitudes of motion, contacts between surfaces again resume and a frequency increase is generally observed. More importantly, as the wind direction shifts back toward 280°, the response level diminishes and spectra 5 and 6 begin to separate into two peaks, reminiscent of what was observed globally at the NS sensors in Figure 4. The torsional response gradually eases and separates toward the higher frequencies, allowing the sway to reappear as the dominant mode. This behavior indicates that the isolated high-amplitude motions here are facilitated by the simultaneous presence of significant EW sway and torsion, facilitating beat phenomena that are treated in spectral representations as a single, energetic peak with large bandwidth.

The ability of the structure to respond in such a manner is certainly facilitated by the close proximity between the sway and torsional frequencies and is more importantly aided by the potential amplitude dependence of stiffness and damping that can shift the torsional frequency toward the EW sway mode and markedly accentuate spectral bandwidths. However, the continuous beating in Figure 5 is not always observed in this structure due to the fact that both response components must be in near proportion. Interestingly, due to the fishtailing phenomenon in this structure, these conditions are achieved more readily at the location of sensor #59-EW than anywhere else. Other analyses in Kijewski-Correa (2003) and Kijewski-Correa et al. (2003) further affirm these behaviors in the response recorded by other sensors and even in other wind events. As a result of these behaviors, this building underwent considerable stiffening and the addition of tuned mass dampers.

The analysis of response time histories to extract estimates of in situ damping values for B1 before the installation of its damper was greatly affected by the beating phenomenon. In the absence of beating, damping values were near 1% critical, as normally expected in steel buildings, but showed a definitive increase with amplitude (Kijewski and Kareem et al., 1999). It is unclear whether the damping increase is due to the classic assumption that larger amplitudes lead to greater frictional losses between components, or whether it is indicative of the energy exchange beginning to occur as the sway and torsional modes couple. Kijewski and Kareem (1999) observed, in the presence of strong coupling, these same accentuated bandwidths leading to damping values in excess of 2%. This then begs the question: how does the very definition of energy dissipation evolve when two modes coalesce?

3.2 The role of amplitude dependence

This phenomenon was also observed in the analysis of S1 by Abdelrazaq et al. (2005). A cross-section of the data was first analyzed using Fourier PSDs to determine the dynamic properties of the building in situ. The resulting spectra for the orthogonal responses at the three measurement locations were
shown previously in Figure 2. The natural periods and critical damping ratios associated with each spectral peak were then extracted via the Half Power Bandwidth Method and were averaged across all three locations and reported in Table 1. It should be noted that the $y$-direction response was found to have consistently higher uncertainties in both period and damping estimation. This can be explained by examining the spectra in Figure 2. Note that the $y$-direction response at some locations, e.g., Location 1, shows evidence of coupling with the $x$-direction response—a characteristic predicted from the finite element modeling of the building (Abdelrazaq et al., 2005). In fact, all three fundamental modes were predicted to have modest levels of coupling, and it is this feature that makes the identification of dynamic characteristics in the $y$-direction particularly difficult.

In situ values were then compared to the designer predictions/assumptions in Table 1. The design periods of vibration are taken from the finite element model for building, while the design damping values represent levels commonly assumed for such structures: 1.5% of critical for the serviceability limit state (Abdelrazaq et al., 2005). Note that for all three response components periods of vibration were 34% lower than the designer’s predictions, though the in situ ratio of the periods in modes 1, 2 and 3 are quite consistent with the design prediction (design: $T_x: T_y: T_q = 1:0.97:0.81$ vs. in situ: $T_x: T_y: T_q = 1:0.96:0.80$). This consistency indicates that the modeling of the overall system behavior was accurate, though some general properties of the structural materials possibly differed in situ. For example, the building was modeled assuming levels of cracking expected over the service life, yielding a reduced modulus of elasticity in the finite element model; however, this level of cracking has yet to be physically realized in this newly constructed building, possibly explaining the stiffer periods observed in situ. Another contributing factor may be that the in situ compressive strengths are higher than those assumed in design, resulting again in a stiffer structure. Meanwhile, despite the significant uncertainty surrounding damping prediction, the observed damping values from the PSD, essentially under ambient vibrations, are quite consistent with the 1.5% value assumed in the design, especially considering that damping levels are generally hypothesized to increase with amplitude (Jeary, 1986). These analyses were twice repeated on different cross-sections of the dataset and showed remarkable consistency with the values reported in Table 1 (Abdelrazaq et al., 2005; Pirnia et al., 2007). Thus, it was comfortably assumed that serviceability and survivability level events will likely experience larger levels of energy dissipation.

Pirnia et al. (2007) later applied the Random Decrement Technique (RDT) to this same dataset, in the hope of quantifying the degree of amplitude dependence in the dynamic properties potentially obscured in PSD analyses. It is widely understood that stiffness (frequency) reduces with amplitude of motion, which was affirmed for both lateral responses, even over the limited amplitude range considered (Figures 6 and 7). A linear fit to the outputs at Location 1 indicates a softening of frequency

<table>
<thead>
<tr>
<th>Mode Characteristics</th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dominant X-sway, slight Y-sway and torsion</td>
<td>Dominant Y-sway, slight X-sway and torsion</td>
<td>Dominant torsion, slight X- and Y-sway</td>
</tr>
<tr>
<td>Design</td>
<td>6.8</td>
<td>1.5</td>
<td>6.6</td>
</tr>
<tr>
<td>Avg. in situ</td>
<td>5.1</td>
<td>1.45</td>
<td>4.9</td>
</tr>
</tbody>
</table>

Table 1. Comparison of design and in-situ periods and critical damping ratios in S1
Figure 6. Amplitude-dependent frequency and critical damping ratio (X-sway) in S1, with inset PSDs for three amplitude regimes.
Figure 7. Amplitude-dependent frequency and critical damping ratio (Y-sway) in S1, with inset PSDs for three amplitude regimes
in the X-sway from its low amplitude threshold by a factor of 0.0027\(\dot{x}\). For the Y-sway, the frequency softens by a factor of 0.0023\(\dot{y}\). Thus the degree of amplitude dependence is more pronounced in the X-axis. While the variation in frequency is less than 1% over the amplitude range considered, it was demonstrated by Pirnia \textit{et al.} (2007) that this can have significant influences on damping estimation in the frequency domain.

Interestingly, the amplitude-dependent damping results demonstrate two distinct regimes. For the X-sway, damping values observed at the various locations converge when the peak responses exceed 0.4 milli-g. The inset PSDs in Figure 6 help explain the source of this behavior. In the low-amplitude regime, the resonant response is quite modest compared to the noise floor, prohibiting full isolation of the resonant contribution and leading to inaccurate damping estimates. As the resonant amplitude increases, its participation in the overall response becomes more significant, and it can be isolated. This results in the converged damping value of 0.84%. Note that this value is approximately half the damping value observed previously in the spectral analysis (Table 1). Similarly for the Y-sway, damping values observed at the various locations converge when the peak responses exceed 0.2 milli-g, despite the X- and Y-sway responses having overall comparable response levels. It becomes apparent from the inset PSDs in Figure 7 that a slight coupling with the X-sway response at low amplitude levels infiltrates the RDT analysis and again leads to inflated damping values. Beyond 0.2 milli-g, a stabilized damping value of approximately 0.5% is observed, again less than half of the previous PSD result (Table 1). However, the stabilized damping values for this building are consistent with the values noted in RDT analyses conducted on other composite and reinforced concrete tall buildings: Di Wang Building (\(\zeta \sim 0.6\%\)) (Li \textit{et al.}, 2005), Central Plaza (\(\zeta \sim 0.5\%\)) (Li \textit{et al.}, 2005), Jin Mao (\(\zeta \sim 0.55\%\)) (Li \textit{et al.}, 2006), and Bank of China (\(\zeta \sim 0.4\%\)) (Li \textit{et al.}, 2003). Still, the discrepancies between the stabilized RDT values and the PSD results are troubling. Although it is understood that spectral bias will lead to overestimates of damping, considering the resolution used in the spectral analysis, these biases are not sufficient to explain these differences. Instead, it is hypothesized that there are two factors at play in S1:

1. neglecting the effects of noise, RDT analyses eventually isolate each mode and determine the damping value strictly associated with it at a given amplitude. If the structure has coupling, then it is characterized by energy exchange and even coalescence between two modes, as was the case with both B1 and S1. In such cases, a spectral analysis will determine a damping value based on this more energetic, coupled response, yielding generally higher damping values. However, the use of filtering in the RDT method will remove any significant contributions of a coupling mode, leading to an estimate of damping that is generally lower.

2. as explained in detail in Pirnia \textit{et al.} (2007), amplitude dependence in both natural frequency and damping cannot be distinguished by a spectral analysis. This analysis will simply average the features present over the hours of data being considered and produce a single spectral peak at the mean value of this frequency range with comparatively wider bandwidth to represent the frequency variation. Similarly, amplitude dependence in damping will further contribute to a widened spectral peak. In either case, a widened spectral peak is interpreted by definition as greater energy dissipation (damping).

Both of these factors imply that spectral analysis of buildings like S1 will produce larger damping values than a time domain analysis like RDT conducted on the same data. Again this returns to the very definition of damping and how it evolves in the presence of both coupling and amplitude dependence. In a practical sense, some additional energy is likely dissipated in the coupling action, implying that the damping values shown in Figures 6 and 7 are lower limit estimates; however, the inability to accommodate amplitude dependence as well as coupling implies that the PSD analysis results in Table 1 are likely unconservative and the actual damping \textit{in situ} is somewhere between the two analyses.
3.3 Effect of structural system type on energy dissipation

While it is acknowledged that the degree of energy dissipation in structures depends on material type, the structural system employed has been given little to no consideration in any prescriptive damping guidelines to date. To explore the role of system type, the same RDT analysis was applied to C1–C3 by Kijewski-Correa et al. (2007). Table 2 and Figure 8, respectively, summarize the level of frequency and damping amplitude dependence observed for each of the buildings. Our discussion begins with C1, whose 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 of C1 show a remarkable consistency over the amplitudes considered, with a discernible increase in damping with amplitude, approaching the 1% value commonly assumed in steel construction (Figure 8a). A markedly stronger degree of amplitude dependence is noted in C2, 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 C3, modes characterized by a greater degree of frame action (vs. cantilever/axial shortening) tend to manifest greater amplitude dependence. These observations are echoed in the damping estimates shown in Figure 8(b), where damping values are comparatively larger for the Y-response and actually exceed the 2% level often assumed for reinforced concrete structures. This is consistent with the findings in Erwin et al. (2007), 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 C2. The differing levels of damping observed between the two lateral responses in this building underscore the importance of linking assumed damping levels to structural system characteristics and not material type alone.

As C3 is generally characterized by ‘symmetry’ in its lateral modes (Kijewski-Correa et al., 2006a), it is not surprising to see both lateral responses manifesting the same softening rate of 2.5% the initial

![Figure 8. Amplitude-dependent critical damping ratios for (a) C1, (b) C2 and (c) C3](image-url)
stiffness (Table 2). Interestingly, despite also being steel in nature, the amplitude dependence in sway periods is nearly doubled in this building in contrast to C1. Furthermore, damping values show a far greater degree of fluctuation in both this building and C2, in comparison to C1. 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 (Kijewski-Correa et al., 2006a). It was previously hypothesized in Kijewski-Correa et al. (2006a) and reiterated in Erwin et al. (2007) that buildings with a greater proportion of frame action demonstrate comparatively larger damping values. Thus while both C1 and C3 are steel, the dominance of axial shortening in the former versus panel zone effects in the latter may indeed offer an explanation of their differences in energy dissipation capability.

Table 3 offers a comparison of the design-predicted dynamic properties for each building and those observed in situ, averaged over the amplitude range considered (Kijewski-Correa et al., 2006a). In C1, periods of vibration show excellent agreement between the predictions and the in situ values.

### Table 2. Summary of linear representations of frequency–amplitude dependence in C1–C3

<table>
<thead>
<tr>
<th>Building</th>
<th>X-sway (Hz)</th>
<th>Y-sway (Hz)</th>
<th>Torsion (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>($R^2$)</td>
<td>($R^2$)</td>
<td>($R^2$)</td>
</tr>
<tr>
<td>C1</td>
<td>$-0.0034x + 0.2078$ (0.92)</td>
<td>$-0.0019x + 0.1438$ (0.94)</td>
<td>$-0.0186x + 0.5059$ (0.74)</td>
</tr>
<tr>
<td>C2</td>
<td>$-0.0062x + 0.1827$ (0.91)</td>
<td>$-0.0204x + 0.1854$ (0.96)</td>
<td>$-0.3251x + 0.3110$ (0.93)</td>
</tr>
<tr>
<td>C3</td>
<td>$-0.0030x + 0.1200$ (0.88)</td>
<td>$-0.0029x + 0.1200$ (0.71)</td>
<td>$-0.0556x + 0.2319$ (0.82)</td>
</tr>
</tbody>
</table>

Best-fit lines project frequency in hertz as a function of acceleration ($x$) in milli-g. $R^2$ (in parentheses) is a measure of error in linear approximation.

### Table 3. Comparison of averaged periods and critical damping ratios from amplitude-dependent analysis and predictions made in design for fundamental sway modes of C1–C3

<table>
<thead>
<tr>
<th>Behavior</th>
<th>Mode 1: Period (s)</th>
<th>Damping (%)</th>
<th>Mode 2: Period (s)</th>
<th>Damping (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1: design</td>
<td>7.0</td>
<td>1.0</td>
<td>4.9</td>
<td>1.0</td>
</tr>
<tr>
<td>C1: in situ</td>
<td>7.0</td>
<td>0.8</td>
<td>4.8</td>
<td>0.7</td>
</tr>
<tr>
<td>Behavior</td>
<td>Mode 1: X-axis translation, slight torsion</td>
<td>Mode 2: Y-axis translation, slight torsion</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C2: design</td>
<td>6.7</td>
<td>1.0</td>
<td>6.4</td>
<td>1.0</td>
</tr>
<tr>
<td>C2: in situ</td>
<td>5.5</td>
<td>1.2</td>
<td>5.5</td>
<td>2.1</td>
</tr>
<tr>
<td>Behavior</td>
<td>Coupled translation (X)</td>
<td>Coupled translation (Y)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C3: design</td>
<td>7.7</td>
<td>1.0</td>
<td>7.6</td>
<td>1.0</td>
</tr>
<tr>
<td>C3: in situ</td>
<td>8.5</td>
<td>1.0</td>
<td>8.3</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Meanwhile, C2 demonstrates periods 11–25% stiffer in situ than predicted by the finite element models. As in S1, this may be attributed to the model’s stiffness reductions for assumed cracking or the difference in-situ modulus of elasticity, although the degree of participation of gravity elements in the overall lateral resistance is another possible source (Kijewski-Correa et al., 2006a). C3, on the other hand, has in situ periods that are generally longer than finite element model predictions, by approximately 10%. As shown in Kijewski-Correa et al. (2007), the explicit modeling of panel zones may indeed be responsible for this discrepancy. Meanwhile, explanations for larger damping values in modes possessing significant ‘frame action’ were previously offered.

4. CONCLUSIONS
While full-scale monitoring is gaining increasing acceptance throughout seismic zones in the United States, its application in the remainder of the country is significantly lacking. Tall buildings under wind constitute an equally viable cross-section of civil infrastructure worth investigating in full-scale, considering the sensitivity of their responses to structural dynamic properties like frequency and damping ratio, as well as the strict acceleration limit states imposed on their design. Damping, in particular, becomes an increasingly important parameter that still cannot be reliably engineered. This is in part due to the challenges in its estimation, as well as its complexities. Through the analysis of full-scale dynamic response data from tall buildings in Boston, Chicago, and Seoul, this study has underscored the effects of coupling, beat phenomenon, amplitude dependence and structural system type on the dynamic properties, and the inadequacies of spectral analysis techniques in capturing these features in full-scale. In particular, given the intermittent features associated with coupled responses, this study advocated the use of time–frequency representations in their characterization and the identification of mechanisms contributing to their evolution.

As suspected, damping was found to increase with amplitude, but also in the presence of beating modes, begging the question of whether the traditional definition of damping need be recast in such situations. This study also underscored the increased energy dissipation and amplitude dependence of buildings characterized by frame action, indicating that features of the structural system type are equally central to any viable prescriptive damping standard. Given these findings, as well as the challenges noted in modeling reinforced concrete lateral systems and steel elements characterized by significant panel zone effects, it becomes evident that full-scale monitoring efforts should not only be continued but also expanded to include a wider cross-section of structural systems and wind climates for a more comprehensive variation of the state of the art in tall building design.

ACKNOWLEDGEMENTS
The authors wish to acknowledge the financial support of the Chicago Full-Scale Monitoring Program by NSF through grants CMS 00-85109 and CMMI 06-01143, as well as the Chicago Committee on High Rise Buildings and the Structural Engineers Foundation (Structural Engineers Association of Illinois). The authors also wish to acknowledge their collaborators on this work: Ahsan Kareem and his students at the University of Notre Dame, Nicholas Isyumov, Jon Galsworthy, Dave Morrish, and the students and staff of the Boundary Layer Wind Tunnel Laboratory at the University of Western Ontario, Bill Baker, Bob Sinn, Brad Young, and the engineers at Skidmore Owings and Merrill, Ahmad Abdelrazaq of Samsung Corporation, and Frank Durgin, formerly of MIT. Furthermore, the authors wish to thank the building owners and management for their participation and cooperation in these monitoring efforts.
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