Full-Scale Validation of Finite Element Models for Tall Buildings

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ABSTRACT
While high-rise construction serves as one of the most challenging projects undertaken by society each year, tall buildings are one of the few constructed facilities whose design relies solely upon analytical and scaled models, which, though based upon fundamental mechanics and years of research and experience, have yet to be systematically validated in full-scale. In response to this deficiency, a full-scale monitoring program was initiated through the combined efforts of the authors and includes three tall buildings in Chicago: two steel and one concrete, providing a sampling of some of the world's 25 tallest buildings, with structural systems common to high-rise construction. Through the course of this program, the response of the buildings, as predicted from wind tunnel tests, is compared to the observed level of response for a variety of wind events.

A secondary validation being conducted in the program, and the subject of the present paper, involves the in-situ periods of vibration and those predicted by finite element models constructed using commercially available software. This paper shall overview the assumptions made in the development of finite element models for each of the buildings monitored in the program. Predicted mode shapes and periods of vibration are discussed. These periods of vibration are then compared to those observed in full-scale to verify the accuracy of assumptions made in their development. A discussion of assumed and observed levels of inherent structural damping is also provided.

Keywords: Full-Scale Monitoring, High-Rise Design, Finite Element Modeling, Periods, Damping

INTRODUCTION

Even though the performance of tall buildings affects the safety and comfort of a large number of people in both residential and office environments, tall buildings are one of the few constructed facilities whose design relies solely upon analytical and scaled models, which, though based upon fundamental mechanics and years of research and experience, has yet to be systematically validated in full-scale. In particular, as state-of-the-art structural analysis software and wind tunnel testing are advancing rapidly, the accuracy and validity of their results needs to be calibrated with respect to actual performance. Understandably, since the development of full-scale models for this type of structure is not feasible, monitoring the performance of actual structures becomes the most viable means for verification and improvement of current design practices and analytical modeling approaches. For example, full-scale monitoring provides the opportunity to directly correlate actual building performance, quantified in terms of lateral and torsional accelerations, to occupant perception criteria. Such efforts may lead to a more refined definition of criteria strongly impacting structural design of tall buildings. Furthermore, full-scale measurements allow the validation of other modeling and design assumptions and expand existing databases of damping levels. These issues are particularly important to insure satisfactory performance, economy and efficiency of future designs of increased complexity and height.

In response to this need, a partnership between the University of Notre Dame (UND), the Boundary Layer Wind Tunnel Laboratory (BLWTL) at the University of Western Ontario (UWO) and Skidmore, Owings & Merrill LLP (SOM) in Chicago was established to initiate the Chicago Full-Scale Monitoring Program[1]. Through the program, the actual performance of three tall buildings in Chicago is compared to predictions, both by finite element and wind tunnel models, thereby providing an important missing link between analytical modeling and actual behavior. Based on these comparisons, the sources of discrepancies are identified to allow enhancement of current design practice. These evaluations also examine the in-situ periods and damping ratios of the buildings under a variety of wind conditions and over a range of response amplitudes. As such, these efforts will enhance existing databases presently lacking substantial information on buildings of significant height and will provide important information on the variation of dynamic properties with amplitude.

In this paper, the authors first overview the monitoring program and then specifically focus on the finite element modeling of each of the buildings in the study. The paper will then present a sampling of the periods and damping ratios observed in full-scale under a specific wind event and compare those to the values predicted/assumed in the design stage, with some discussion of the underlying reasons for discrepancy between design predictions/assumptions and full-scale observations.

DESCRIPTION OF INSTRUMENTED BUILDINGS

The primary objective of this study is to correlate the in-situ measured response characteristics of tall buildings in full-scale, with computer-based analytical and wind tunnel models for the advancement of the current state-of-the-art in tall building design. Such an endeavor requires the selection of several buildings representative of structural systems common to high rise design, all located in the same general locale of downtown Chicago, for which design information and building access are obtainable. Since major effort was expended to establish relationships with the building owners to allow access, the anonymity of the buildings must be assured to guarantee continued access for the life of the program. Note that the reluctance of building owners to permit access to their buildings for instrumentation and monitoring has precluded earlier efforts from being realized not only in the US but also abroad. As such, the structures will be generically referenced as Buildings 1, 2, and 3. Each 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 summarized in Table 1:

Building 1: 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.

[1] A project website has been established at http://windycity.ce.nd.edu.

Building 2: 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.

Building 3: The steel moment-connected, framed tubular system of Building 3 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.

Table 1. General Structural Properties of Monitored Buildings.

	Material	System	Occupancy	Density (pcf)	Floor-to-Floor Height
Building 1	Steel	Tube	Office/ Residential	9.0	9'-0" to 12'-6"
Building 2	Concrete	Shear Wall / Outrigger	Office	18.0	13'-0"
Building 3	Steel	Tube	Office	9.8	12'-10"

In terms of dynamic characteristics, all three buildings have significant separation between their torsional and translational frequencies, being relatively stiff torsionally. As a result of this feature and other attributes, low torsional responses are expected. In light of their unique structural systems and the characteristics listed in Table 1, some hypotheses on inherent damping can also be made. Given that Building 1 has the lightest density and mostly axial deformations due to cantilever action, it is anticipated to have the lowest damping of the three buildings. Building 3, having similar density but comparatively larger contributions of flexure, shear and panel zone effects to its overall deformation mechanism, is expected to have relatively higher damping. On the other hand, Building 2 is expected to have higher damping than either of its counterparts by virtue of its concrete construction. Finally, as each building is rectangular in plan, with the primary axes aligning with North and East, subsequent discussions will reference sway response as North-South (N-S) or y-sway and East-West (E-W) or x-sway for simplicity, as shown in Fig. 1.

INSTRUMENTATION OVERVIEW

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, as shown in Fig. 1. The outputs of these sensors are sampled every 0.12 seconds and archived by a 15-bit Campbell CR23X data logger. The primary instrumentation systems were respectively installed in Buildings 1, 2 and 3 on 06/14/02, 6/15/02 and 4/30/03. Though wind-induced displacements are characterized by both background (quasi-static) and resonant components, only the latter can be recovered by the aforementioned accelerometer system. Therefore, it was of interest to monitor both of these contributions in full-scale using Global Positioning Systems (GPS). A differential GPS sensor pair was installed at the centerline of Building 1 and on a nearby stationary reference building on 8/26/02. In this differential configuration, the Leica MC 500 GPS used in this study are capable of achieving sub-centimeter resolution, based on calibrations conducted before full-scale deployment (Kijewski-Correa, 2003). The performance of the GPS in full-scale has been verified against the accelerometers, as discussed in Kijewski-Correa et al. (2005) and a comparison of GPS-measured displacements with wind tunnel estimates are provided in Kochly and Kijewski-Correa (2005).

In addition, two ultrasonic anemometers were installed on masts 41 m above the rooftop of the tallest building in the program, Building 3, so that the reference wind speed and direction for each event may be measured at this site and reliably converted to represent the wind speed at the top of each instrumented building. This installation was completed in the summer of 2004.

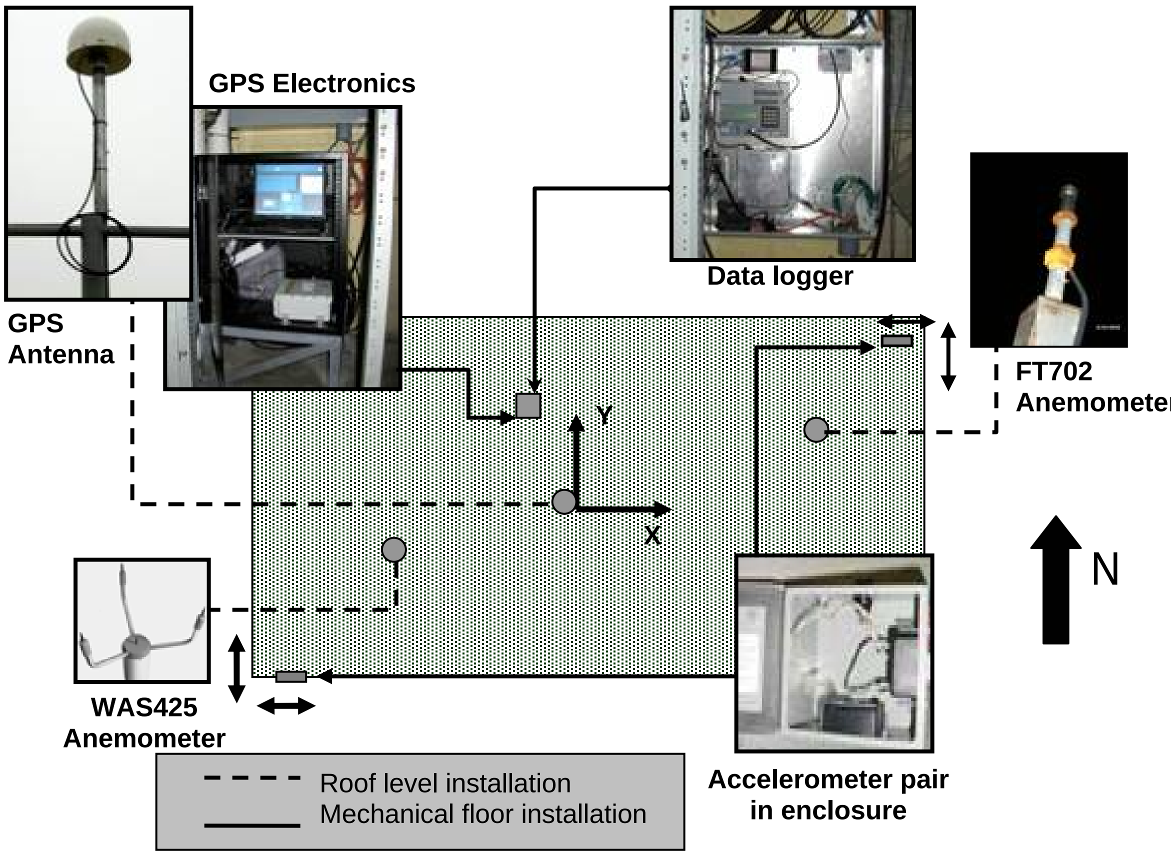

Fig. 1. Generalized sensor array on generic floor plan with inset photographs of equipment.

WIND TUNNEL TESTING

Though aeroelastic model tests would provide direct information on aerodynamic damping effects and, depending on the type of model, contributions of higher modes of vibration to the response, the high-frequency force-balance (HFFB) method was chosen for the wind tunnel tests in this study as it allows the flexibility to repeat response predictions based on the measured modal force spectra but considering different building dynamic properties without the requirement of additional wind tunnel testing. Accordingly, differences between the in-situ and predicted structural properties of the buildings are easily reconciled using the HFFB method as compared to aeroelastic tests. The modeling for the force balance tests conducted in this study consisted of three components: 1) a rigid and lightweight detailed 1:500 scale model of each of the study buildings; 2) a detailed model of the structures surrounding the building sites within a full-scale radius of about 750 m; and 3) a less detailed model of the upstream terrain, chosen to simulate the scaled turbulence intensity and velocity profiles expected at full-scale for each site. All wind tunnel tests were conducted in the high-speed section of the closed-circuit wind tunnel (BLWT II) at the BLWTL at UWO. From the recorded base bending moments, equivalent static wind loads and acceleration levels are predicted. More details on the wind tunnel testing and response prediction, including a comparison with in-situ response levels, are provided in Kilpatrick et al. (2003) and Kijewski-Correa et al. (2005).

FINITE ELEMENT MODELING

Throughout the design stages, structural engineers rely on finite element (FE) models to predict the full-scale behavior of buildings. Such aspects like overall damping, translational and torsional frequencies, and the associated mode shapes define the dynamic characteristics of the structure. These fundamental characteristics are used in wind tunnel testing to predict equivalent static wind loads that are then applied

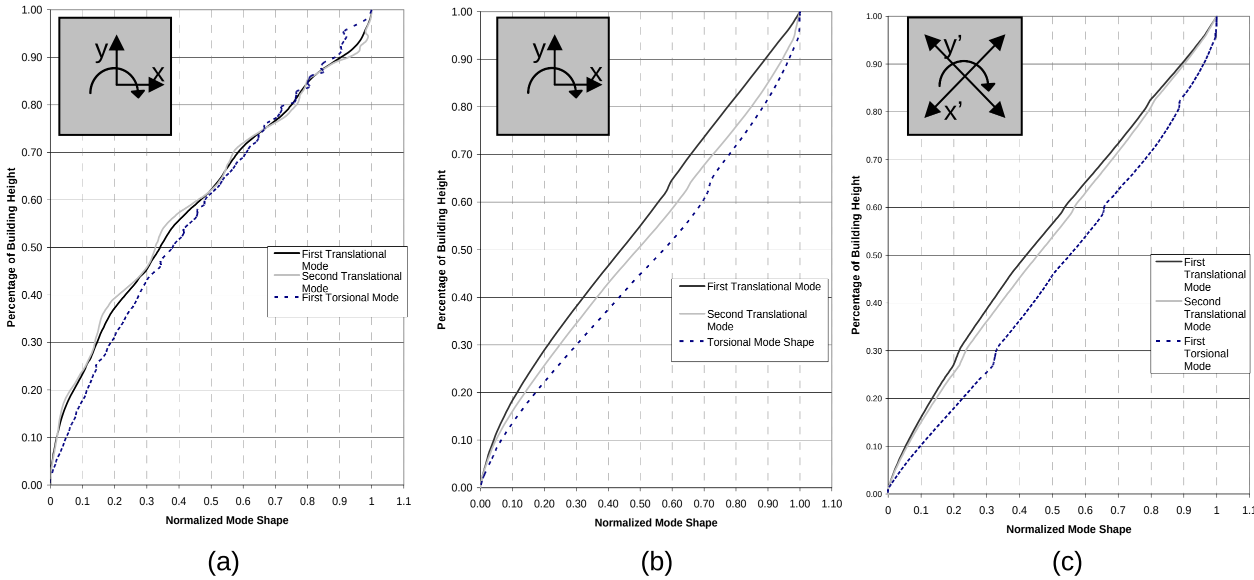

Fig. 2. Normalized fundamental mode shapes for Buildings (a) 1, (b) 2 and (c) 3.

to the building model for the survivability-level design of components and are also used in analyses to predict accelerations for assessing the acceptability of the building motions in terms of occupant comfort. 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, a capability that implicitly relies on the ability to construct an accurate analytical representation through a finite element model. For the buildings associated with this study, finite element models were developed using currently available commercial software: ETABS (ETABS, 2002) and SAP 2000 (SAP, 2002), based upon careful reference to the design drawings. It was not the purpose of this study to apply a unique set of modeling assumptions to the FE models in order to mimic a known, in-situ measured result. Rather, all assumptions regarding the finite element representation of the buildings in this study reflect those commonly applied in design offices for serviceability assessment.

In order to predict the modal characteristics of the study buildings, an eigenanalysis was performed. The mass associated with the self-weight of the structure and the full weight of the exterior cladding system are included in the dynamic analysis. Additionally, special attention is paid to the use of the building and the resulting loading conditions at each floor in order to determine what fraction of the design imposed load to include in the mass calculations for the dynamic analysis. Due to the heights of the study buildings, the analysis includes the effects of building displacement on the frequencies through a second-order (P-Δ) iteration. The buildings were modeled as fixed at the base, such that no base rotation exists. This is thought to approximate the generally high soil-structure interfacial stiffness observed in Chicago for buildings under wind-induced lateral loads.

For Buildings 1 and 3, framed primarily in structural steel, the representation of the member stiffness was straight-forward, as the steel elements remain elastic at service level loadings. For the reinforced concrete building (Building 2), adjustments were made to selected lateral-load resisting elements to represent the post-cracking stiffness of these elements under service level loads. Specifically, the flexural and shearing stiffnesses of the link (coupling) beams within the shear wall system were reduced to one-half and one-fifth of the elastic stiffness, respectively. The beam-supported slab was modeled using shell elements. The flexural stiffness of the slab's shell elements was set to one-half of the elastic stiffness in order to approximate the post-cracking behavior of the slab, which transfers flexure and shear between the perimeter columns and core shear walls. While generally considered to support gravity floor loads alone, explicit modeling of the linkage between the floors, exterior columns, and core often results in a substantial contribution to lateral resistance in reinforced concrete buildings[2].

Fig. 2 shows the mode shapes for each of the buildings, normalized with respect to the top floor displacement. The inset in each figure shows the axes of vibration displayed in the plot. Table 2 summarizes the resulting periods from the FE analyses conducted at SOM and the damping levels assumed by the original designers of the buildings. Buildings 2 and 3 undergo coupled responses, though the extent of coupling in Building 2 is much less than Building 3. Note that although the authors acknowledge that Building 2 can be reasonably expected to have higher damping, the damping values of 1% for habitability/serviceability and 1.5% for survivability reported in Table 2 were the values specified in their design.

Table 2. Predicted Periods of Vibration and Assumed Damping Levels for Finite Element Models of Buildings 1-3.

	Mode 1		Mode 2		Mode 3	
	Period	Damping	Period	Damping	Period	Damping
Building 1	Y-axis translation		X-axis translation		Torsion	
	7.0 s	1%	4.9 s	1%	2.0 s	1%
Building 2	X-axis translation, slight torsion		Y-axis translation, slight torsion		Coupled Torsion	
	6.7 s	1%*	6.4 s	1%*	4.6 s	1%*
Building 3	Fully coupled x-translation		Fully coupled y-translation		Fully coupled torsion	
	7.7 s	1%	7.6 s	1%	4.5 s	1%

*1% used for Accelerations, 1.5% used for Equivalent Static Wind Loads.

COMPARISON TO FULL-SCALE

A total of over 8000 hours of time histories have been collected thus far in the program and are currently under analysis to not only compare in-situ response levels to wind tunnel predictions but also to verify the in-situ dynamic properties against those predicted/assumed in the design stage. To facilitate this discussion, the in-situ dynamic properties estimated during a wind event on April 28-29, 2004 are now presented. The responses of the three buildings during this wind event are analyzed in detail in Kijewski-Correa et al. (2005). During this wind event, winds were from the south-southwest. A second data sampling for Building 1 during a wind event on February 11, 2003 was presented previously in Kilpatrick et al. (2003). As discussed in Kijewski et al. (2005), two methods were utilized to determine the natural frequency and damping of the three buildings: a power spectral approach using the Half-Power Bandwidth Technique (HPBW) (Bendat and Piersol, 1986) and a time-series approach, the Random Decrement Technique (RDT) (Vandiver et al., 1992). Further details of the analysis, including a verification of stationarity, are presented in Kijewski et al. (2005). The Random Decrement estimates of the period and critical damping ratio in the fundamental mode of the two sway and torsional responses are presented in Table 3, along with an estimate of the Coefficient of Variation (CoV), i.e., uncertainty, associated with those estimates. Note that these uncertainties are far more significant for the estimation of structural damping, as expected (Kareem and Gurley, 1996).

For reference, the relative responses of x-sway, y-sway and torsion-induced lateral sway during this event were: 1:1.7:0.17 for Building 1, 1:0.52:0.05 for Building 2 and 1:0.76:0.12 for Building 3. This confirms the lack of torsional response in the buildings, as expected. Further, the behaviors of Buildings 2 and 3 show the amplified response in the E-W direction (x-sway), characteristic of a dominant acrosswind response

[2] It should be noted that while modern finite element computer analysis models account for the actual mass and stiffness distribution throughout the structure, it is believed that lumped-mass models utilized at the time of the original design of Buildings 1-3 correlated well with the more detailed distributed mass models used in this study.

for this wind event. This is not the case in Building 1, however, where the acrosswind axis (x-sway) is considerably stiffer (see Tables 2, 3), yielding a dominant alongwind response for this wind event.

Table 3. Periods of Vibration and Damping Ratios Estimated by RDT with Coefficient of Variation in Parenthesis.

	Building 1	Building 2	Building 3	Building 1	Building 2	Building 3
	Period [s]			Damping [percent critical]		
X-Sway	4.89 (0.10%)	5.61 (0.22%)	8.60 (0.25%)	0.87% (23.9%)	1.42% (7.4%)	1.04% (20.6%)
Y-Sway	7.11 (0.19%)	5.66 (0.68%)	8.60 (0.14%)	0.88% (8.9%)	2.4% (8.0%)	1.21% (23.0%)
Torsion	1.99 (0.07%)	3.41 (0.71%)	4.35 (0.14%)	0.87% (14.9%)	3.59% (13.4%)	1.33% (16.9%)

Comparison of Periods of Vibration

In Building 1, periods of vibration show excellent agreement between the predictions in Table 2 and the in-situ values in Table 3. Building 2 demonstrates periods approximately 10-25% stiffer in-situ than predicted by the FE models. This may be attributed to the FE model's stiffness reductions for cracking that has yet to be observed in the service life of this building. It is equally likely that the in-situ modulus of elasticity is larger than that assumed in the FE modeling. Building 3, on the other hand, has in-situ periods in its sway responses that are generally longer than FE model predictions, by approximately 10%, though showing good agreement in the torsional mode. Further investigation of this building's response has also revealed amplitude-dependence in the sway periods.

Considering the multitude of variables that can affect the frequency of a built structure, both theoretical and measured, that are either unknown or difficult to quantify, the magnitude of the discrepancy between the predicted and measured frequencies for Building 3 is not surprising. However, the fact that the measured frequencies were lower than predicted values is somewhat unexpected and warrants further analytical review and discussion.

Generally, the natural fundamental frequency of a structure is a function of its mass and stiffness characteristics. As an increase in mass and/or a decrease in stiffness can lower the building frequency, an underestimation of overall building mass and/or an overestimation of the overall structural stiffness from an analytical standpoint can result in a predicted frequency that is somewhat higher than the measured in-situ frequency. It is worthwhile to focus discussion on analytical aspects and structural characteristics of Building 3 that could help to account for the difference between the predicted and measured frequencies. Specifically, the following topics are discussed:

1.) Panel Zone Stiffness
2.) Service Condition Mass Variability
3.) Foundation Stiffness
4.) Beam/Column Frame Connectivity

Panel Zone Stiffness. The structural system of Building 1 is a trussed tube, such that lateral displacement is primarily a function of member axial deformations. In comparison, the structural system of Building 3 is a tubular moment frame, such that lateral displacement is a function not only of axial distortions in the columns of the tubular walls but also of frame shear and flexural deformations. The frequency predictions in Table 2 are based upon FE analytical models utilizing beam 'stick' elements in a centerline-to-centerline configuration. These FE beam elements are more suited to represent the axial deformation associated with the trussed tube system of Building 1 than the shear deformations associated with the moment frame system of Building 3. More specifically, the FE 'stick' model represents the beam-to-column connection as a fully rigid interface. It is known that significant elastic deformation can occur within the panel zone of beam-to-column moment connections within steel moment frame structures (Charney and Downs, 2002). This panel zone deformation may have a significant impact on the overall stiffness of the structure. Even utilizing centerline-to-centerline modeling of the beam-to-column joint with no rigid link elements within the connection zone may ultimately overestimate the stiffness at the joint. This softening effect at the panel zone should be considered as a possible source for the discrepancy between the measured and predicted frequencies for Building 3. Because this behavior has been shown to occur in the elastic range, it may also explain the amplitude-dependence of the in-situ frequency that is suggested by the measurements.

Service Condition Mass Variability. Typically, as part of the dynamic frequency analysis, the structural engineer will include the mass associated with the self-weight of the structure and cladding system, as well as an estimation of the imposed loads that are likely to be in place under normal service conditions. While the self-weight of the structure can be estimated relatively accurately, the 'expected' imposed loads carry more variability in their estimates. Further, these imposed loads are subject to change over time depending upon the usage of the building. Considering that Building 3 is an office building and has been in service for over 30 years, it is possible that the imposed loads have increased somewhat over time as file storage volume grows and/or office layout becomes denser. For Building 3, the mass associated with the estimated imposed loads comprises approximately 18% of the overall building mass which is considered in the dynamic analysis. Using the fundamental relationship:

$$f_{n,1} = \frac{1}{2\pi}\sqrt{K_1 / M_1} \tag{1}$$

where $f_{n,1}$ is the natural frequency in the fundamental mode, K_1 is the stiffness in the fundamental mode and M_1 is the mass in the fundamental mode, and considering a variable range of imposed loads of +/- 50% (corresponding to +/- 9% on overall building dynamic mass), we can expect a frequency variation of +/- 4.5%. For a very reasonable range of imposed load variability, we can arrive upon frequency estimations that can vary by as much as 5%. Certainly this mass variability cannot entirely explain the ~11% discrepancy in measured versus predicted frequency that we observe for Building 3, but it can be considered to at least set a baseline for expected accuracy between these two values.

Foundation Stiffness. The FE model that was utilized for Building 3 employs a fixed-base condition at the soil/structure interface. In actuality, some amount of flexibility associated with this interface, and with the foundation system itself, is likely. Inclusion of springs in the FE model to represent this flexibility at the base of the structure would be reflective of common modeling practice for tall buildings and would effectively lower the predicted frequencies somewhat. However, the authors feel that due to the stiff subgrade conditions at the site, the particular foundation system used (short, straight-shaft caissons founded on bedrock), and the transient nature of the wind loads acting upon the structure, the fixed-base model provides a reasonable representation of the in-situ conditions. It is felt that modification to the base conditions within the FE model to reflect the stiffness of this foundation system would not significantly affect the predicted frequencies. In other applications, the foundation stiffness may play a prominent role in the behavior of the structure and more rigorous attention to this effect may be warranted.

Beam/Column Frame Connectivity. Building 3 was constructed in a modular fashion - beam and column cruciform assemblies were shop connected and lifted into place, and spandrel beams were connected near midspan with a web shear connection, at the theoretical flexural inflection point on the frame. The source FE model, however, considers the spandrel beam to be continuous through this joint with full moment transfer capability. A sensitivity study was therefore undertaken to determine the impact of explicitly modeling the spandrel beam midspan hinge condition on the predicted frequency. This study concluded that inclusion of this spandrel hinge in the frequency analysis did not significantly affect the model predictions. Due to the placement of the spandrel shear connection at midspan, the moment transfer demand at this location is minimal. Further, a force couple mechanism can be developed between this spandrel shear connection and the continuous composite metal deck slab spanning across the top of the spandrel, which is capable of transferring a nominal moment across this hinge. It is therefore reasoned that the nature of the connection between the modular frame assemblies does not significantly increase the flexibility of the overall structural system and therefore does not contribute significantly to the discrepancy between the predicted and measured frequency values.

Upon comparison of the frequency analyses from Buildings 1 and 2, it may seem that Building 3 shows poor correlation between its predicted and measured frequencies, particularly in light of the fact that the measured frequencies show a more flexible behavior than what was predicted by the FE model. However, examination of the possible sources for this discrepancy and consideration of the array of variables that may affect the measured and/or predicted frequencies indicates that this discrepancy may very well be within a range that should be expected when correlating across analytical and as-built conditions. Still, it is valuable to consider the characteristics of the built structure and the nature of the analytical model to better understand the sources for these discrepancies. Here, it seems likely that the flexibility associated with possible elastic deformations within the panel zone may be a significant factor contributing to the flexibility of the structure. This is a characteristic unique to moment frame systems and one which will not be represented accurately through the use of a FE beam model unless careful attention is given to the joint stiffness properties. Finally, the authors emphasize that there is inherent variability in the considerations the structural engineer makes in his/her estimates of the building imposed loads and associated mass, as well as the flexibility of the foundation system, and the representation of other characteristics that may be unique to each individual structure.

Comparison of Damping Values

Recall that the relative levels of damping were speculated for each of the buildings in the Description of Instrumented Buildings section. This speculation indicated that Building 1 would have the least damping and Building 2 the most. This speculation has indeed been confirmed by the in-situ damping levels in Table 3 for each response component. Given also that the return period of this event is approximately annual, and the assumed damping levels are intended for larger return periods, then the use of 1% damping in the design of these three buildings was likely appropriate for Building 1 and even conservative for Building 3, given the general assumption of amplitude-dependence in damping (Jeary, 1986). In the case of Building 2, the assumption of 1% seems highly conservative, as expected for concrete structures.

CONCLUDING REMARKS

This paper introduces a study established to allow the first systematic validation of tall building performance in the US using full-scale data in comparison with wind tunnel and finite element models used in design. For each of the three tall buildings currently monitored in the City of Chicago, instrumentation is overviewed and wind tunnel and analytical modeling approaches are summarized. A comparison of the dynamic properties observed in full-scale to those assumed/predicted in design is provided. This comparison indicates that with respect to fundamental periods of vibration, standard modeling assumptions can reliably predict in-situ periods of the uncoupled steel building (Building 1). However, the assumptions made in the modeling of the reinforced concrete building (Building 2) cannot be wholly validated in light of the fact that the levels of cracking assumed in the various service states have not been realized yet in full-scale, coupled with the likelihood that the in-situ of modulus of elasticity is higher than assumed in the model. Reasons for the discrepancies and amplitude-dependence in the sway periods of Building 3 may stem from a number of sources discussed herein, including panel zone stiffness, service condition mass variability, foundation stiffness and beam/column frame connectivity.

With respect to damping, the assumption of damping levels at 1% for serviceability design were likely conservative for the concrete and even coupled steel building (Buildings 2 and 3, respectively), given the common presumption of amplitude-dependence. The 1% damping assumption seems more appropriate for the uncoupled steel building (Building 1). These damping estimates further support speculative opinions on the relative damping in each building, by virtue of their unique deformation mechanisms and construction materials. Finally, it should be cautioned that these observations were based on the analysis of an isolated wind event. Therefore, the appropriateness of these assumed damping levels, as well as predicted periods, will be more thoroughly verified with the continued analysis of data collected in the program.

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