Exploratory Studies on a Bilinear Aeroelastic Model for Tall Buildings

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ABSTRACT: The wind-induced response of tall buildings is usually dominated by dynamic crosswind loading. Currently, for design of tall buildings, analysis of structural behavior and building response under wind loading is linear. In this paper, a simple bilinear aeroelastic model was designed and constructed for wind tunnel testing of tall buildings. The proposed aeroelastic model is composed of a rigid model on a variable stiffness base sway flexure which can achieve reduced stiffness and increased damping properties at ultimate wind conditions. The Wall of Wind facility at Florida International University will be used in future to generate extreme winds so that the model can be tested in ultimate load conditions. From this pilot study, a better understanding of the non-linear behavior of a tall building in ultimate load conditions can be obtained. This could affect the way tall buildings are designed for wind in future.

KEY WORDS: Tall buildings; Bilinear Stiffness; Aeroelastic Model; Wind Tunnel.

1 INTRODUCTION

The world is currently undergoing the biggest wave of tall building construction in history. Tall buildings are currently designed assuming elastic behavior up to the ultimate wind speed conditions. Approximate assumptions are made with respect to reduction of stiffness and increase in damping as deflections increase under load but the analysis of structural behavior and building response under wind loading is almost without exception still linear. As a result it is likely that tall buildings have a larger margin of safety than typically assumed. This implies the structural systems may be more costly and have a larger carbon footprint than necessary. The design approach used to handle earthquake loads is different and it is accepted that the response at design conditions will be non-linear. The different approach used for wind probably stems from the historical concept of wind as being a steady force pushing in one direction, a concept that has some justification on buildings that are not dynamically sensitive but has little justification on tall buildings.

The wind-induced response of tall buildings is often dominated by crosswind loading which is almost entirely dynamic. However, the current state of the art is incomplete when it comes to assessing how these effects are in reality mitigated by non-linear effects as the structure becomes very highly stressed. There are detailed questions currently as to how the stiffness and damping of tall buildings vary as the loading increases and it is expected that more full scale measurements will in future shed further light on these questions for different types of structural systems. But, even though the precise variations of damping and stiffness with deflection on real buildings still need further research, something can be learned about the general implications of these non-linear effects by experimenting with simplified non-linear scale models in fully simulated extreme wind conditions. This can help understand how conservative and potentially wasteful of resources we are being at present by insisting that tall buildings remain within the linear range of elastic behavior right up to ultimate wind conditions. This can also help evaluate the extent of economic benefits by moving from linear to non-linear design approaches.

Recently structural engineers at Walter P. Moore and Associates applied non-linear analysis on a US government building, combined with the time history of wind loads provided by RWDI to show the building had significantly higher margin of safety than a simple linear analysis would give. This made the difference between demolishing the building completely and simply replacing the curtain wall system (Griffis et al, 2012). The wind loads obtained from the wind tunnel in that case were collected using a model with linear behavior and the non-linear structural behavior was introduced in the structural analysis computations.

This paper proposes a new “aeroelastic” scale model with non-linear behavior. It is intended that in future the model will be tested in the Wall of Wind (WOW) at Florida International University (FIU) in simulated wind conditions that take it well beyond the elastic range. The results from this pilot study could then be used as the basis for pursuing a more comprehensive study.
Model-scale testing in boundary layer wind tunnels has long been the main means to determine wind loads on tall buildings. The wind loading on a structure is only truly modeled when the wind and the structure are both correctly modeled, that is, the model responds to the loading system in the same way as the full-scale structure. In tall slender buildings a large portion of the generated internal forces in the structure, are due to the inertia of the building as it sways and twists in wind. A realistic simulation of inertial forces and the direct wind forces can be obtained from wind tunnel testing on aeroelastic models which are designed to sway and twist in wind in the same way as the full scale building does. There are different methods for conducting an aeroelastic wind tunnel testing including “Leg Model”, “Rigid Model on Gimbal Mounting”, “Rigid Model on Base Sway Flexure” and “Torsionally Flexible Model on Base Sway Flexure” (Irwin, 1982). In aeroelastic model testing, in addition to the similarity of the wind flow characteristics and the structure’s exterior geometry, the similarity of the inertia, stiffness and damping characteristics of the building are required. Having selected the geometric scale, the dynamic behavior of the model and prototype should be similar which requires equality of the following ratios in the model and in full-scale building:

Density scaling:
\[
\frac{\rho_b}{\rho} = \frac{\text{inertia forces of the building}}{\text{inertia forces of the flow}}
\]  
(1)

Stiffness properties:
\[
\frac{E}{\rho v^2} = \frac{\text{elastic forces}}{\text{inertia forces of flow}}
\]  
(2)

Damping:
\[
\xi_s = \frac{\text{dissipative structural forces}}{\text{inertia forces of the flow}}
\]  
(3)

- The model air density is usually equal to that of the prototype. So, Equation (1) results in the constant bulk density of the building between model and prototype \((\rho_{bm} = \rho_{pb})\). Similarity of mass between prototype and model comes from Equation (1). Mass scale is defined as \(\lambda_M = M_m/M_p\), where \(M\) is mass and length scale is defined as \(\lambda_L = L_m/L_p\), where \(L\) is a reference length. \(m\) and \(p\) denote model and prototype quantities, respectively. Since \(\rho_{bm} = \rho_{pb}\), we have:

Translation mode:
\[
M_m = \lambda_M M_p = \lambda_L^3 M_p
\]  
(4)

where:
\[
\rho_{bm} V_m^2 \rightarrow \lambda_M = \frac{\rho_{bm} V_m^2}{\rho_{pb} V_p^2} = \lambda_{pb} \left(\frac{\rho_{bm}}{\rho_{pb}}\right) = \lambda_M
\]  
(5)

Rotation mode:
\[
I_m = \lambda_I I_p = \lambda_L^5 I_p
\]  
(6)

where \(V_m\) and \(V_p\) are the volume of the model and the prototype, respectively. \(\lambda_I\) is the scaling factor between the moment of inertia of the model and prototype and is defined as \(\lambda_I = I_m/I_p\) in which \(I_m\) and \(I_p\) are the moment of inertia of the model and the prototype, respectively. The correct mass distribution on the model is usually obtained by first building the model too light and then adding weights at appropriate locations.

- Similarity of stiffness between prototype and model comes from Equation (2). The equivalent for Equation (2) for the response of a tall building in a particular mode of vibration with frequency \(f\) becomes:

\[
\left(\frac{F}{V}\right)_m = \left(\frac{F}{V}\right)_p
\]  
(7)

where \(f\) is the frequency, \(D\) is a reference length and \(v\) is the flow velocity. So, for a consistent scaling of all relevant modes of vibration, the velocity scale becomes:

\[
\frac{v_m}{v_p} = \lambda_v = \frac{\lambda_L}{\lambda_L^2}
\]  
(8)

where \(T\) is the time or period of vibration. In absence of Froude number scaling requirement, the velocity scale is constrained only by the need to achieve a minimum body Reynolds number. Having chosen the velocity scale for simulation, the stiffness scaling \((\lambda_K = K_m/K_p)\) becomes:

\[
\omega = 2\pi f = \sqrt{K/M} \rightarrow K = (2\pi f)^2 M
\]  
(9)

Translation:
\[
\lambda_K = \lambda_L^2 \lambda_M = \frac{\lambda_L^2 \lambda_M}{\lambda_L} = \frac{\lambda_L^2 \lambda_{pb} \lambda_I^2}{\lambda_L} = \lambda_L^2 \lambda_L
\]  
(10)
Rotation:

\[ \lambda_K = \lambda^3 \beta^2 \]

where \( \omega \) is the angular frequency and \( K \) is the stiffness.

- The damping of sway motion is adjusted using hydraulic dampers at the base of the model or by introducing sources of energy dissipation (e.g., rubber strips) at joints in the model.

3 DESIGN OF A NONLINEAR AEROELASTIC MODEL

One simple approach for performing an aeroelastic test on tall buildings in a wind tunnel is using a rigid model on a base sway flexure as shown in Figure 1a. In this type, the deflection shape of the full scale sway modes of vibration is approximated by a straight line. However, simulation of the torsional motions is not possible with this model which rotates as a rigid body about horizontal axes at its base. In practice, the flexure is just a rectangular steel column with linear behavior. In this paper, a new flexure with bilinear softening stiffness behavior is proposed and designed to be used with a rigid scaled model of tall buildings for wind tunnel testing.

There are several methods proposed in the literature for developing variable stiffness elements. There are “smart materials” which can rapidly respond to a stimulus. Shape memory alloys (a class of alloys whose mechanical properties, including the modulus of elasticity, changes with temperature), piezoelectrics (sometimes referred to as variable stiffness elements which can provide a virtual variable stiffness by changing the force of these elements in an instantaneous manner using a control algorithm), magnetorheological elastomers (their mechanical properties including the elasticity can be controlled rapidly and reversibly by an applied magnetic field), etc. are all different categories of smart materials. In addition, variable stiffness elements can be obtained by designed mechanical systems e.g., changing the number of the active coils in a coil spring or changing the geometry of a compliance mechanism. With mechanical systems, designing a flexure with softening stiffness behavior is more challenging than with hardening behavior (Azadi et al, 2009).

The aeroelastic model considered in this paper is schematically shown in Figure 1b. It is composed of a rigid model on a base sway flexure with bilinear softening stiffness behavior. In this bi-linear behavior, the stiffness is constant up to a certain strain and then transitions to another constant, but lower value. To do so, two design options were proposed and investigated. The flexure initially was designed as “Option 1” shown in Figure 1b. It was composed of two hollow steel tubes initially compressed together using a tensioned rod. The idea was that the flexure would have the full cross-section activated until the strain becomes sufficient to open up the crack in the flexure roughly at its midpoint. Further increase in load would result in the bending moment now being resisted primarily by the increased tension in the central rod and compression of one side of the flexure. This flexure was designed and tested in Titan Structures Lab at FIU. However, there were slippage problems at the crack level which prevented a smooth transition from higher stiffness to the lower stiffness. “Option 2” was then proposed with the same concept. In this case, the flexure is one hollow steel tube, compressed initially to the base using the tensioned rod. Four rectangular thin steel tabs were then welded to each side of the flexure and attached to the base using small bolts. The stiffness of each tab can be changed by changing its dimensions and moving the attachments to the base further out from the flexure. One unknown is how much damping will be introduced once the gap in the flexure starts to open and close. This can be examined by some bench testing of the model.

In order to design the dimensions of the base flexure for each specific project we have to consider the frequency and stiffness scaling requirements as explained previously. The deflection of the model is mainly determined from the slope of the cantilever deflection curve at its free end. This determines the rotation of the model about its base, and the cantilever can then to first approximation be treated as a rotational spring (stiffness \( Nm/rad \)) with the model rotational inertia. The lateral deflection at the tip of the cantilever adds a small additional flexibility which can also be included in the analysis. In other words, the flexure is assumed to be as a cantilever beam with an end moment. The flexure in the proposed model is a hollow steel tube, the dimensions of which come from \( K_{m,rotation} = EI/L \) in which \( E \) is the modulus of elasticity of steel.
The next step is to calculate the required tension in the rod. The free body diagram of the flexure is shown in Figure 2. The model imparts a bending moment and a shear force to the top of the flexure. The shear force is the same at all levels on the flexure but the bending moment does vary slightly depending on where on the flexure we measure moment.

The crack at the base opens up when the tensile stress created at the outer edge of the flexure due to the bending moment becomes equal to the initial compressive stress created by the tensioned rod.

\[ f_{\text{crack}} = 0 \rightarrow \frac{P}{A_{\text{rod}} + A_{\text{flexure}}} = \frac{M}{S_{\text{rod}} + S_{\text{flexure}}} \] (12)

where \( f_{\text{crack}} \) is the stress and \( M \) is the bending moment at the crack level. \( A_{\text{rod}} \) and \( S_{\text{rod}} \) are the cross sectional area and the section modulus of the tensioned rod. \( A_{\text{flexure}} \) and \( S_{\text{flexure}} \) are the cross sectional area and the section modulus of the flexure without considering the tension rod (hollow tube). By assuming a value for \( A_{\text{rod}} \), the required tension in the rod, \( P \), which causes the crack to open up at a specific displacement can be calculated using Equation (12). Moment \( M \) at point of zero deflection which is approximately in the middle of flexure can be calculated using the following formula:

\[ M = \int_0^H \bar{m} \omega^2 x z d z = H \int_0^1 \bar{m} \omega^2 x (z/H)^2 d(z/H) = \frac{1}{3} \bar{m} \omega^2 H^2 x_{\text{H}} \] (13)

where \( \bar{m} \) is the model mass per unit height. \( x, x_{\text{H}}, z, H \) and \( M \) are defined in Figure 2. \( x_{\text{H}} \) is the critical displacement at the model height, a point after which the model is required to go non-linear. The shear force at the point of zero deflection can be calculated from:

\[ F = \int_0^H m_{\text{model}} \omega^2 x d z = H \int_0^1 m_{\text{model}} \omega^2 x (z/H)^2 d(z/H) = \frac{1}{2} m_{\text{model}} \omega^2 H x_{\text{H}} \] (14)

Having the moment and shear force at the point of zero deflection, the moment at the base can be calculated.

As a starting point, the tension in the rod is set in such a way that when the tip deflection of the model gets to about \( H/200 \), the non-linear behavior starts. It should be noted that after the crack opens, the rod and a small portion of the flexure would contribute to carry the applied load on the structure. For the rod diameter assumption, one should take into account the capacity of the system to carry loads after crack opening.

4 CASE STUDY

In this paper a super tall building with dimensions of 50 m x 50 m x 500 m was considered for which the general aerodynamic response, including vortex shedding excitation is known. Model scale of \( \lambda_c = 0.003 \) was chosen. The model density was taken to be similar to a typical building density (\( \rho_p = 250 \text{ Kg/m}^3 \)). The first mode natural frequency was set to give a vortex shedding resonance peak at a convenient speed for the wind tunnel. For example, assuming a Strouhal number \( (fL/v) \) of about 0.1, to have a vortex shedding peak at around 15 m/s we would need a frequency of about 10 Hz. Too high frequencies should be avoided so that the model support system could be kept stiff enough so that the model does not shake the support excessively.
Finite Element Analysis (FEA) using ANSYS commercial software was used in considering various options before the final design. Figure 3 shows the FEA results for two of various configurations considered for designing the flexure to show softening stiffness behaviour. Note that just half of the models are shown so that the tension rod can be visible inside the flexure. A fixed condition was considered at one end of the flexure. PRETS179 pretension elements were used for applying the required tension force in the rod. Contact elements were also used to avoid unacceptable penetration of different parts together. Figure 3.a shows the Option 1 explained in Figure 1.a which was the initial idea for designing flexure with bilinear behavior. FEA results clearly shows the slippage problem at the crack level. In the flexure shown in Figure 3.b, a hollow cylinder was used instead of a hollow square tube with chamfered edges. Although this setup did not show slippage problems at the crack level, the actual model tested in the lab showed that this configuration was very unstable to the application of lateral loads.

Figure 3. FEA using ANSYS

The final flexure (Option 2, Figure 1.b), sized using the procedures explained in Section 3, resulted in the length=0.1524 m, outer dimension 2.54 cm and thickness of 2 mm. The rod diameter was chosen as 1.27 cm resulting in the required tension in the rod equal to 21254 N. Finite element analysis was performed for sizing the tabs and their attachment points to the base (Figure 4). The final tabs designed were 2.54 cm by 7.62 cm rectangular plates with 0.51 mm thickness. Different dimensions can be assumed for flexure components each resulting in different amount of stiffness reduction (Figure 4; obtained from FEM analysis).

Figure 4. (a) FEA using ANSYS showing the crack at washer location; (b) Load-displacement curves for two different flexure sizes
Experiments were performed in the Titan America Structures lab at FIU to find out the behavior of the flexure (Figure 5a). The tension rod was run throughout the system to compress the hollow steel tube to the base using a hydraulic jack which applied the tension load of 21254 N to the rod. An electrical jack was then used to apply the lateral load gradually from zero to the amount needed to open the crack. A combination of straps, clamps, welded steel plates, and wood plates were used to provide a fixed support.

Figure (5b) shows the load-deflection diagram for the final design compared to FEM results. The results are promising and the stiffness at large deflection was reduced to 31% of the initial low deflection stiffness. This can be modified by changing the flexure dimensions. Free vibration testing of the system is in progress for finding out the damping characteristics of the system.

Figure 5. (a) Test setup; (b) Load-displacement behavior of the designed flexure

5 CONCLUSIONS AND FUTURE WORK

The objective of this paper was to design and construct a simple bilinear aeroelastic model for wind tunnel testing of tall buildings with reduced stiffness and increased damping as ultimate wind conditions are approached. In future, the intention is that testing will be performed in extreme wind conditions in the FIU Wall of Wind using the proposed model. Wind-induced loads will then be compared with results for a linear behavior model and load modification factors will be estimated. It is anticipated that the lessons learned from these studies will provide useful information on how buildings truly behave as they are stressed by wind loading beyond their elastic range. The changes in response measured on non-linear physical models of the type investigated here can be used in future to help validate non-linear time domain structural analyses based on aerodynamic data from linear models.

REFERENCES