WIND ISSUES IN THE DESIGN OF TALL BUILDINGS

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Wind Issues for Structural Design

- Structural integrity under ultimate loads
- Deflections under service loads
- Building motions and occupant comfort
- Uncertainties in building structural properties (stiffness, damping)
- Uncertainties in wind loading
- Uncertainties in wind climate
- Codes and standards
- Computational Fluid Dynamics
Relationship between importance of wind and height
Vortex shedding

Shedding frequency $N$ is given by

$$N = S \frac{U}{b}$$

$S$ = Strouhal number
$U$ = wind speed
$b$ = building width

Directions of fluctuating force

wind
Peak response due to vortex excitation

Crosswind Response

Magnitude of peak response

\[ \propto \frac{1}{\text{density} \times \text{damping}} \]

\[ U_{\text{CRIT}} = \frac{N_b}{S} \]

Vortex shedding

No vortex shedding

Wind velocity, \( U \)
Vortex excitation on a tapered spire
Mode 1
Vortex excitation on a tapered spire Mode 2
Shape strategies to reduce excitation

- Softened corners
- Tapering and setbacks
- Varying cross-section shape
- Spoilers
- Porosity or openings
Taper effect - Petronas towers
Taipei 101 – corner softening

25% REDUCTION IN BASE MOMENT
Shape effect, Taipei 101 wind tunnel tests
Burj Khalifa – 828 m

Set backs, changing cross-section, orientation

Completed Building

Early 1:500 scale wind tunnel tests
Shanghai Center – 632 m
Twisting and tapering
151 storey tower in Korea

Creation of bleed slots at edges to suppress vortex shedding

Full scale rendering

1:500 wind tunnel model
Use of corner slots to bleed air through corners on a tall building

Crosswind motion

Plan view without slots

Plan view with slots

Base moments reduced by 60%

Wind

Wind
Reliability considerations for flexible buildings

Expression for load factor $\lambda_w$

$$
\lambda_w = \frac{1}{K_w} e^{\alpha^2 \beta V_w}
$$

$V_w = \text{Coefficient of variation of wind load}$

$\beta = \text{Reliability index}$

$K_w = \text{Bias factor}$

$\alpha = \text{Combination factor}$
First order, second moment reliability analysis for rigid buildings

Code analysis method

\[ V_w = (V_{qi}^2 + V_{an}^2)^{1/2} = (0.2^2 + 0.3^2)^{1/2} = 0.36 \]

\[ \lambda_w = \frac{1}{K_w} e^{\alpha^2 \beta V_w} = \frac{1}{1.3} e^{(0.75^2 \times 3.5 \times 0.36)} = 1.56 \]

Wind tunnel method

\[ V_w = (0.2^2 + 0.1^2)^{1/2} = 0.22 \]

\[ \lambda_w = \frac{1}{1.0} e^{(0.75^2 \times 3.5 \times 0.22)} = 1.54 \]

Both the coefficient of variation and bias factor are smaller in the wind tunnel method. Load factor ends up at about the same.
Reliability of flexible buildings with monotonic response

Wind load varies as power law

\[ W \propto U^n \]

Rigid building

\[ V_w = (V_{qi}^2 + V_{an}^2)^{1/2} \]

Flexible building

\[ V_w = (V_{qi}^2 + V_{an}^2 + V_{dyn}^2)^{1/2} \]

\[ V_{dyn} \approx \sqrt{n_d^2 \zeta^2 + (n-2)^2 V_f^2 + (n-2)^2 V_v^2} \]

- Damping term
- Frequency term
- Velocity sensitivity term
Required load factors for flexible buildings with monotonic response

Rigid building

$$V_w = (V_{qi}^2 + V_{an}^2)^{1/2} = (0.2^2 + 1^2)^{1/2} = 0.22$$

$$\lambda_W = e^{(0.75^2 \times 3.5 \times 0.22)} = 1.54$$

Typical flexible building

$$V_w = (V_{qi}^2 + V_{an}^2 + V_{dyn}^2)^{1/2} = (0.2^2 + 0.1^2 + 0.11^2)^{1/2} = 0.249$$

$$\lambda_W = e^{(0.75^2 \times 3.5 \times 0.249)} = 1.63$$

Highly flexible building

$$V_w = (V_{qi}^2 + V_{an}^2 + V_{dyn}^2)^{1/2} = (0.2^2 + 0.1^2 + 0.22^2)^{1/2} = 0.31$$

$$\lambda_W = e^{(0.75^2 \times 3.5 \times 0.31)} = 1.84$$

Note:- Wind tunnel method is assumed in all cases.
Wind loads determined directly at ultimate return period by wind tunnel method

<table>
<thead>
<tr>
<th>Building type</th>
<th>( n )</th>
<th>( n_{damp} )</th>
<th>Required Load Factor</th>
<th>Actual Load Factor</th>
<th>Load Factor Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rigid</td>
<td>2</td>
<td>0</td>
<td>1.54</td>
<td>1.60</td>
<td>1.04</td>
</tr>
<tr>
<td>Flexible</td>
<td>2.5</td>
<td>0.25</td>
<td>1.63</td>
<td>1.80</td>
<td>1.10</td>
</tr>
<tr>
<td>Very Flexible</td>
<td>3</td>
<td>0.5</td>
<td>1.84</td>
<td>2.02</td>
<td>1.10</td>
</tr>
</tbody>
</table>

*Slightly conservative*
Consequence of vortex shedding on reliability assessment – Nakheel Tower

Low ratio of 1000 year to 50 year loads

Uncertainty in loads is mostly dictated by uncertainty in this peak

Need to carefully assess uncertainty in peak vortex shedding response since the normal assumption that uncertainty in wind speed governs is no longer valid. Damping and frequency uncertainties become important.
Reynolds number effects

- Originate from viscosity of air
- Can cause changes in flow patterns on a small scale model relative to full scale
- Not a concern on sharp edged structures
- Can be a concern on curved shape buildings
- Is lessened by high turbulence and surface roughness
Effect of Reynolds number on pressure coefficient around a circular cylinder

Reynolds number

$$R_e = \frac{Ub}{v}$$

Kinematic viscosity of air

Sub-critical Re < $2 \times 10^5$

Transcritical Re > $3 \times 10^6$
1:50 Scale Model of Upper Portion of Burj Khalifa High Reynolds Number Tests, $Re \sim 2 \times 10^6$

Note:
Full scale $Re \sim 7 \times 10^7$
Comparison of Mean Pressure Coefficients at Low and High Reynolds Number - Burj Dubai

Wind Direction = 320deg.
High Reynolds number tests on Shanghai Center

Example of Mean Pressure Coefficients

Pressure tap count  →
Damping and dynamic response beyond the elastic limit

\[ \zeta_{\text{eff}} = \frac{1}{2\pi} \frac{\delta E_{1/2}}{E} \]

\[ \eta = \frac{x_2 - x_1}{x_1} \]

\[ \zeta_{\text{eff}} = \frac{1}{\pi} \frac{\left(1 - \frac{k_2}{k_1}\right)\left(\eta + \frac{1}{2} \frac{k_2}{k_1} \eta^2\right)}{1 + 2\eta + \frac{k_2}{k_1} \eta^2} \]
Effect of stiffness reductions and inelastic deflections on effective viscous damping ratio

\[ \zeta_{eff} = \frac{1}{2\pi} \frac{\delta E_{1/2}}{E} \]

Example: For 20% deflection beyond elastic limit effective increment in damping ratio = 0.024. This would be additive to the damping ratio below the elastic limit which is typically assumed to be 0.01 to 0.02.
Damping and inelastic response research

- Non-linear time domain analysis of structures under realistic wind loading will allow us to evaluate whether more efficient structures can be developed.
- How feasible is it to load the structure beyond its elastic limit and how far can one go with this?
- Can we learn how to keep the structure stable and robust after going plastic. What can be learnt from earthquake engineering?
- More full scale monitoring needed at representative deflections.
Building motions and criteria

- Problem is complex due to variability amongst people
- What return period should be used?
- What quantity best encapsulates comfort: acceleration; jerk; something in between; angular velocity; noises combined with motion, etc?
- Designers have to make decisions and move on.
- What is the actual experience using traditional approaches?
Building Motion Criteria. Historical review of 19 buildings wind tunnel tested by RWDI in the 1980s and 1990s.

<table>
<thead>
<tr>
<th>Building Number</th>
<th>Building Height (m)</th>
<th>First Order Modes Frequencies (Hz)</th>
<th>Assumed Damping Ratio (% of critical)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1*</td>
<td>249.4</td>
<td>0.193 0.199 0.295</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>163.4</td>
<td>0.176 0.224 0.250</td>
<td>2.0</td>
</tr>
<tr>
<td>3</td>
<td>198.1</td>
<td>0.189 0.184 0.300</td>
<td>1.5</td>
</tr>
<tr>
<td>4</td>
<td>137.2</td>
<td>0.185 0.135 0.323</td>
<td>2.0</td>
</tr>
<tr>
<td>5</td>
<td>236.2</td>
<td>0.154 0.169 0.400</td>
<td>2.0</td>
</tr>
<tr>
<td>6</td>
<td>178.0</td>
<td>0.244 0.250 0.400</td>
<td>2.0</td>
</tr>
<tr>
<td>7</td>
<td>215.0</td>
<td>0.177 0.149 0.331</td>
<td>1.5</td>
</tr>
<tr>
<td>8</td>
<td>110.9</td>
<td>0.192 0.164 0.250</td>
<td>2.0</td>
</tr>
<tr>
<td>9</td>
<td>163.0</td>
<td>0.195 0.208 0.224</td>
<td>1.5</td>
</tr>
<tr>
<td>10</td>
<td>124.4</td>
<td>0.170 0.224 0.204</td>
<td>2.0</td>
</tr>
<tr>
<td>11*</td>
<td>207.8</td>
<td>0.147 0.171 0.250</td>
<td>1.5</td>
</tr>
<tr>
<td>12*</td>
<td>145.2</td>
<td>0.411 0.213 0.440</td>
<td>2.0</td>
</tr>
<tr>
<td>13</td>
<td>94.0</td>
<td>0.278 0.243 0.139</td>
<td>2.0</td>
</tr>
<tr>
<td>14</td>
<td>141.2</td>
<td>0.370 0.216 0.356</td>
<td>2.0</td>
</tr>
<tr>
<td>15</td>
<td>143.3</td>
<td>0.135 0.180 0.333</td>
<td>2.0</td>
</tr>
<tr>
<td>16</td>
<td>259.4</td>
<td>0.131 0.125 0.263</td>
<td>1.25</td>
</tr>
<tr>
<td>17</td>
<td>175.4</td>
<td>0.201 0.157 0.200</td>
<td>1.5</td>
</tr>
<tr>
<td>18*</td>
<td>247.8</td>
<td>0.131 0.154 0.211</td>
<td>2.0</td>
</tr>
<tr>
<td>19*</td>
<td>259.4</td>
<td>0.179 0.159 0.236</td>
<td>2.0</td>
</tr>
</tbody>
</table>

* For these buildings, wind tunnel studies predicted peak resultant accelerations above the 15 – 18 milli-g range.
Summary of computed frequencies of 19 buildings
Summary of Predicted and Improved Acceleration Responses

![Graph showing acceleration responses across buildings](image)

- **15-18 milli-g range accelerations**
- **Above 15-18 milli-g range accelerations**
- **Reduced acceleration responses using SDS**
## Buildings with Supplemental Damping System

<table>
<thead>
<tr>
<th>Building Number</th>
<th>Building Name and Location</th>
<th>SDS Type Installed</th>
<th>10-Year Peak Resultant Acceleration (milli-g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Without SDS</td>
</tr>
<tr>
<td>1</td>
<td>Park Tower, Chicago, IL</td>
<td>TMD</td>
<td>24.0</td>
</tr>
<tr>
<td>11</td>
<td>Random House, New York, NY</td>
<td>TLCD</td>
<td>25.3</td>
</tr>
<tr>
<td>12</td>
<td>Wall Centre, Vancouver, BC</td>
<td>TLCD</td>
<td>28.0</td>
</tr>
<tr>
<td>18</td>
<td>Bloomberg Tower, New York, NY</td>
<td>TMD</td>
<td>22.5</td>
</tr>
<tr>
<td>19</td>
<td>Trump World Tower, New York, NY</td>
<td>TMD</td>
<td>27.4</td>
</tr>
</tbody>
</table>
Park Tower, Random House and Wall Center with Damping Systems

Chicago

New York

Vancouver
Trump World Tower and Bloomberg Center with Damping Systems
Chicago Spire – Very Long period – Higher Mode Effects
Higher modes can affect response ~ questions re frequency dependent motion criteria such as ISO comfort criteria.
Chicago spire - motion and deflection control through use of damping system.

**Chicago Spire**
Number of Residents Per Floor
Perceiving Motion Yearly
Mode 1 & 2

**Notes:**
- Approximately 192 residents (8% of entire building) will perceive motion with no AMD incorporated.
- Approximately 15 residents (0.6% of entire building) will perceive motion with an AMD incorporated.

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**Chicago Spire**
Number of Residents Per Floor
Perceiving Motion Yearly
Mode 3 & 4

**Notes:**
- Approximately 243 residents (10% of entire building) will perceive motion with no AMD incorporated.
- Approximately 4 residents (0.2% of entire building) will perceive motion with an AMD incorporated.
Assessing building motions.

ISO Criteria for single frequency

Moving room simulations of multiple frequencies
Meso-scale Modelling of June 1988 Event

Wind Field

500m
MM5v3 @ 12km Grid_Spacing

June 23, 1988 1:00:00
Min= 0.0 at (33,40), Max= 19.5 at (5,55)

Dubai
Thank you