STUDY ON DOWNBURST-INDUCED TRANSMISSION TOWER’S DAMAGE CHARACTERISTICS

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ABSTRACT

Downburst winds, which are sources of extreme wind loadings in nature, have caused numerous transmission towers’ failures around the world. It is essential to study wind-resisting ability and downburst wind-induced damage characteristics of tower. Despite the fact that extensive research has been carried out on transmission lines subjected to normal wind loads, their damage behavior under downburst is poorly defined. The tower’s failure mode and damage region under downburst are entirely different from that of normal wind for the extremely significant different profiles. To acquire the transmission tower’s failure differences under two wind loads mentioned, this paper studied the tower’s section wind-resistance capability and damage region under two wind loads. The tower’s ultimate section bending capability model was presented and the wind-induced section moment analysis method under downburst was proposed. Then on condition that the moments on the bottom section of tower induced by two winds are equal and reach the ultimate section bending capability, the comparison method of wind-resistance capability under two winds was established to study the failure region of the tower. As a numerical example, the failure characteristics of a transmission tower were investigated. The analytical results demonstrate that the peak values of the section moments exceed the ultimate section bending capability below 40m under downburst, while the bottom of tower is the first part to be observed with damages under normal wind loads. Furthermore, it verifies the fact that most collapsed high-rise transmission towers’ damage regions under downburst were indeed in lower-middle parts instead of the bottom part.

KEYWORDS: DOWNBURST WIND LOADS, WIND-RESISTANCE CAPABILITY, ULTIMATE SECTION BENDING CAPABILITY, DAMAGE CHARACTERISTICS

1 Introduction

Wind hazard is the one which has the widespread influence in the nature disaster. Transmission towers, which are prone to wind excitations due to their flexibility and small damping, have experienced a number of failures due to strong wind events with devastating economical and social consequences. The field investigation in the Eastern China No.5237 Renshang 500kV transmission tower’s [Xie Q. (2006)] indicated that the bottom members of tower are relatively complete without severe deforming or buckling. But the members of second and third panels above the bottom are completely destroyed with fraction of partial members. It is clear that these damage observations are not caused by the normal wind excitations because its velocity is increase with height in exponential or logarithm law.

Researchers have reported that the majority of all weather related transmission line failures were the results of downburst. For the incomplete statistics by [Zhang (2006)], 18 transmission line structures of 500 kV and 57 transmission line structures over 110 kV were collapsed in China in 2005 due to the strong wind events such as downburst, tornado and typhoon. It is reported that that out of 94 structural failures in Australia that were recorded, more than 90% of the failure events were induced by severe thunderstorms, such as
downbursts [Hawes and Dempsey (1993)]. Investigations of transmission line failures in Americas, Australia, South Africa, and many other utility organizations have reported that more than 80% of the majority of all weather related line failures were the results of high intensity winds, ranging from fully mature tornadoes to various forms of downbursts that are associated with the occurrence of thunderstorms [Dempsey and White (1996)].

These statistical figures emphasize the needs for further investigation on the failure behaviors of transmission lines under downbursts in order to testify that the failure characteristics investigated are consistent with that by downburst, and develop procedures that can be used to design structures that would resist loading associated with this type of severe wind excitations.

This paper aims to propose a novel analytical framework to study the transmission tower’s section wind-resistance capability and explore the possible damage regions in the tower subjected to downburst loads as well as normal wind loads. Of particular interest herein is whether the transmission structures by normal-wind-loads-governed design can take the impact of downburst wind loads. The proposed model of tower’s ultimate section bending capability is then established and the wind-induced section moments of tower are analyzed by considering the influence of lines. The wind-induced section moments based on two types of wind loads are compared with the ultimate section bending capability model to examine the structural safety and thereby determine the damage region by the assumption that the bottom section moments reach the ultimate section bending capability. According to this method, an arbitrary transmission tower’s working status or damage characteristics can be achieved if the ultimate section bending capability model and downburst wind loads are known beforehand.

2 Theory and Method

2.1 Design Wind Loads on Transmission Lines

To investigate failure characteristics of a strong wind induced high-rise transmission tower, the equivalent design wind loads of the normal wind and downburst wind are studied respectively.

According to field measurement, there are two types of laws of wind pressure along height in boundary layer, one is the exponential law, and the other is the logarithm law. Logarithm law is a reference wind speed multiply by a ratio determined by altitude and terrain:

\[ M_{z\text{,\text{cut}}} = \alpha \ln z + \beta z + \gamma \]  

where \( \alpha, \beta, \gamma \) are parameters terrain related.

The design wind frictional drag force shall be taken for structures and parts of structures as Australian design codes [AS/NZS 1170-2 guidelines]:

\[ F = 0.5 \rho_{\text{air}} [V_{\text{des,} \theta}]^2 C_{\text{fg}} C_{\text{dyn}} A_f \]  

where \( \rho_{\text{air}} \) denotes air density, which is 1.2kg/m\(^3\); \( C_{\text{fg}} \) is aerodynamic shape factor; \( A_f \) is the projected tributary area of all members connected to the point of interest perpendicular to along-wind direction; \( V_{\text{des,} \theta} \) is building orthogonal design wind speed.

Downburst commonly accompanies the storm’s translational motion. The lateral motion of the downdraft causes an increase in the peak downburst velocity in front of the storm, and a decrease in the velocity on the trailing side. The time history of downburst velocity is not a stationary process adopted for normal winds. In practice, the mean wind velocity is obtained from a time history, or time series, by averaging the data in some time interval. The mean, however, is actually an instantaneous mean. If one assumes that the mean wind velocity over the height achieve their maximum values simultaneously, the distribution of wind velocity with height is the vertical profile of the maximum mean wind velocity.
Three empirical models for the vertical profile are available. They were presented by [Oseguera and Bowles(1988)], [Vicroy(1992)] and [Wood and Kwok(1999)].

Oseguera and Bowles’ model is expressed as:

$$V(z) = \left( \frac{\lambda R^2}{2r} \right) e^{-z/\zeta} \left( e^{z_*/\zeta} - e^{-z_*/\zeta} \right)$$  \hspace{1cm} (3)

where $V(z)$ is the maximum mean wind speed at height $z$; $r$ is the radial coordinate from the center of the downburst; $R$ is the characteristic radius of the downburst “shaft”; $z^*$ is a characteristic height out of the boundary layer; $\zeta$ is a characteristic height in the boundary layer; $\lambda$ is a scaling factor, with dimensions of 1/time.

Vicroy’ model is expressed as:

$$V(z) = 1.22 \times \left[ e^{-0.15z/z_{max}} - e^{-3.2175z/z_{max}} \right] \times V_{max}$$  \hspace{1cm} (4)

where $V_{max}$ is the maximum speed in the profile; $z_{max}$ is the height at which the maximum speed occurs.

Wood’s model is expressed as:

$$V(z) = 1.55(\frac{z}{\delta})^{\frac{1}{6}} \times \left[ 1 - erf(0.7 \frac{z}{\delta}) \right] \times V_{max}$$  \hspace{1cm} (5)

where $\delta$ is the height where the velocity is equal to half its maximum value; $erf$ is the error function.

The wind speed of a downburst at any height $z$ at anytime $t$ is the summation of a moving average mean wind speed and a fluctuating wind speed:

$$U(z,t) = \overline{U}(z,t) + u(z,t)$$  \hspace{1cm} (6)

where $U(z,t)$ is the wind speed at height $z$ and time $t$; $\overline{U}(z,t)$ is the moving average mean wind speed, which is a deterministic function. $u(z,t)$ is the fluctuating wind speed which is a stochastic process with zero mean. The fluctuating wind speeds of downbursts are non-stationary stochastic processes. [Chen and Letchford (2004)] presented an approach for combining turbulence with a non-turbulent downburst wind field based on an evolutionary power spectral density method. The wind speed fluctuation is obtained by amplitude modulating the process:

$$u(z,t) = \alpha(z,t) \kappa(z,t)$$  \hspace{1cm} (7)

where the modulation function $\alpha(z,t)$ is $0.08\overline{U}(z,t) \sim 0.11\overline{U}(z,t)$, $\kappa(z,t)$ is a stationary Gaussian stochastic process with standard variation of one.

To obtain a similar design wind loading expression of normal wind, the reference wind velocity of downburst is selected as $\overline{U}_{max}(z)$ simultaneously at sometime when $\overline{U}(z,t)$ reaches its maximum value. Then the downburst design wind load acting on a node of a tower is defined by

$$F = 0.5 \rho_{air} \overline{U}_{max}^2 C_{fg} C_{dyn} A_f$$  \hspace{1cm} (8)

where $\rho_{air}$, $C_{fg}$, $C_{dyn}$, and $A_f$ are assumed in consistent with expression of boundary layer wind. $\overline{U}_{max}(z)$ is the maximum wind velocity in the downburst wind profile, which will be discussed as follows.

2.2 Wind-resistance Capability Verifying Formula of Transmission Tower

The basic requirements without damage of transmission tower subjected to wind loads is that the wind-induced section moment $M_w$ is small or equal to the ultimate section bending capability $M_u$

$$M_w \leq M_u$$  \hspace{1cm} (9)

The structural section wind-resistance capability $SWRC$ can be expressed:
where wind resistance request is satisfied when \( SWRC \) is less than 1, whereas intensity damage occurs when \( SWRC \) is more than 1.

### 2.2.1 Ultimate section bending capability of transmission tower

The ultimate section bending capability at any height \( z \) of a tower structure is equal to the section moment if the stress of at least one member reaches yielding stress. Then the ultimate section bending capability’s function \( M_R(z) \) can be determined by acting horizontal loads on top of tower by increasing loads step by step. Concentrated loads are applied on top nodes of tower in x direction, and members’ stresses of each panel are calculated by static analysis. Then by increasing concentrated loads with given load level and repeating the method mentioned above until one member’s stress reaches its yielding strength, the moment of the panel at this time is the ultimate section bending capability in x direction. Similarly, the ultimate section bending capability in y direction can also be achieved.

We assume that the ultimate section bending capability is satisfied with exponential profile distribution:

\[
M_R(z) = ae^{-b(z-a)} + c
\]

where \( a, b \) and \( c \) are parameters identified by least squares fitting using every panel’s ultimate section bending capability.

### 2.2.2 Wind-induced section moment

Transmission towers are constructed by two dimensional members and present strong geometrically nonlinear behavior and the wind-induced vibration characteristics of a transmission tower are strongly influenced by its geometric configuration. Thus, the transmission line shall be included in the structural finite element model in order to accurately simulate the structural response of the transmission lines and tower under wind loads. Therefore, the section moment at arbitrary height under design wind loads is composed of the moment induced directly by wind loads on the tower \( M_{w1} \) and the moment induced indirectly by the deformation of transmission line \( M_{w2} \).

The moment caused directly by wind loads on the tower is expressed as:

\[
M_{w1}(z) = \int_{z_0}^{H} F(z) (z-z_0)dz
\]

where \( z_0 \) is arbitrary height of tower; \( H \) is total height of tower; \( F(z) \) is design wind loads acting on tower, which is given by Eq. (2) under normal wind loads or Eq. (8) under downburst wind loads.

When transmission lines keep the balance under gravity loads, as shown in Fig. 2, the resultant force of bilateral tension’s horizontal components is equal to zero, and vertical components are equivalent to increase the weight of the tower. Therefore, the tension force of transmission line cannot induce the section moment of tower. If the displacement and deformation of transmission lines are produced by wind loads, bilateral tension’s horizontal components along the line is balanced since the swing of the conductors. But bilateral tension’s horizontal components perpendicular to the line (Line’s Force for short) are not balanced and may produce section moment \( M_{w2} \) of tower.

The following steps are utilized in the evaluation of the moment caused indirectly by the deformation of transmission lines subjected to wind loads: 1) Establish finite element model of transmission lines. 2) Non-linear analyses are performed for the lines to evaluate the Line’s Force. 3) The Line’s Force is applied to the tower and a linear static analysis of the tower is conducted. 4) Based on the force calculation procedure described above, the tower’s section moment indirectly caused by transmission lines are evaluated.

In order to investigate characteristics of transmission line’s force, downburst wind loads were assumed to be the most dangerous case to the tower, whose direction was
perpendicular to line span and the maximum wind speed appeared at the location of tower. Parametric examination is carried out for exploring the effects of various factors such as line span $L$, distance between storm center and tower $d$, maximum wind speed $V_{\text{max}}$ and initial tensile force of line $PF$ respectively, here other invariant factors are shown in Table 1. Line’s Forces to the specific configuration are evaluated as described below.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Sectional area/mm²</th>
<th>Mass per unit length/(Kg/m)</th>
<th>Elastic modulus/GPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>666.55</td>
<td>2.06</td>
<td>63</td>
</tr>
</tbody>
</table>

Fig. 1 indicates the variations of Line’s Force with initial tensile force and line span. The distance between storm center and tower $d$ is 1500m and downburst maximum wind speed $V_{\text{max}}$ is set as 60m/s. It is clear from Fig. 1 that Line’s Force is sensitive to the variations of initial tensile force, $PF$. Line’s Force decrease with the increasing $PF$. In addition, the varying trend is more significant in large line span but not obvious in small line span. This observations from the mechanism that line’s geometric nonlinear lateral stiffness increases with initial tensile force, and wind loads acted on the line decrease with the shorter line span. Thus, line’s displacement and deformation decrease and that will result in little Line’s Force.

Fig. 2 demonstrates the variations of Line’s Force with downburst maximum wind speed and line span. The distance between storm center and tower $d$ is 1500m and line’s initial tensile force $PF$ is set as 50kN. As shown in Fig. 2, the Line’s Force is sensitive to the variation of $V_{\text{max}}$ and the Line’s Force increases with the increasing $V_{\text{max}}$. In addition, the varying trend is more significant in large line span but not obvious in small line span. This behavior results from the nonlinear deformation of the line.

### 2.2.3 Comparison Method of wind-resistance capability

For the great differences between downburst and normal wind, what are the differences between the tower’s force-bearing characteristics under two kinds of wind loads? What is the safety status of a tower if it is designed using the current design standard subjected to a strong downburst event? To solve the problems discussed above, this paper proposes a comparison method of tower’s section wind-resistance capability under different horizontal loads by analyzing their collapse models with downburst and normal wind and recognizing the force-bearing characteristics.

To compare wind-resistance capability under two kinds of design wind loads, the following assumptions are adopted: 1) the bottom section moments of tower induced by two
kinds of wind loads are equal and reach the ultimate section bending capability; 2) section stiffness at the bottom of tower meets the requirement. Then comparison can be carried out by two steps: 1) based on the assumption 1, repeated pilot calculation through dichotomy is conducted to determine parameters of the two loads, which may result in bottom’s section moment equal to ultimate section bending capability. 2) Section moment at arbitrary height is figured out respectively under the action of two wind loads, which can be used to be compared with the ultimate section bending capability, and consequently the safety of the any height is tested using Eq. (10).

3 Analysis of Engineering Example

3.1. Engineering situation

To examine the performance of proposed approach, a 500 kV double circuit suspension tower was constructed in China is analyzed. The analysis was conducted to compare tower’s wind-resisting ability and unsubstantial places under different types of wind excitations. The configuration of the tower is displayed in Fig. 3. The tower has a square base of 20.24m×20.24m and a height of 122m. The tower was modeled using 654 elements and 211 nodes. The analytical model has a total of 1242 degrees-of-freedoms. Two conductors, both with a diameter of 45mm and spanning 300m, attach to the tower at a height of 108m. The line’s parameters were showed in Table 1.

3.2. Ultimate section bending capability model

In order to examine the failure mode of tower under downburst, the structure is divided into 19 panels by its height. Based on tower’s ultimate section bending capability of each panel (loop in Fig.4) calculated by the method derived above, ultimate section bending capability function’s parameters are obtained by least squares fitting:

\[ a = 1.0987 \times 10^8, \quad b = 1.3268 \times 10^{-2}, \quad c = -2.0204 \times 10^7 \]  

Thus, the function is described as

\[ M_r(z) = 1.0987 \times 10^8 \cdot e^{-1.3268 \times 10^{-2} z} - 2.0204 \times 10^7 N \cdot m \]  

Fig.7 shows that exponential model is accurate to represent the varying trend of ultimate section bending capability with height.

3.3. Comparison of wind-resistance capability and failure mode

To investigate tower’s wind-resistance capability and failure mode by downburst, wind-induced section moment at any height by normal wind and three kinds of downburst loads with different vertical profiles are analyzed when the bottom moment reach ultimate section bending capability. Wind loads’ pertinent coefficients were selected based on the AS/NZS 1170-2 guidelines. The terrain type was selected as Category 2 since high-rising transmission towers located at a river coast. The terrain/height multiplier is

\[ M_{z,\text{cut}} = 0.1 \ln z + 0.0001z + 0.77 \]  

wind direction multiplier is set at a constant value,0.95; shielding multiplier \( M_s = 1.0 \) and topographic multiplier \( M_t = 1.03 \) are used in the analysis. Therefore, it can be ascertained a
constant value of normal wind $V_r = 60.83 \text{ m/s}$ by repeated pilot calculation through dichotomy until bottom’s sectional moment equal to ultimate section bending capability.

As for downburst, wind loads on transmission tower and lines can be calculated base on Eq. (8). $U_{\text{max}}(z)$ lies on the type of downburst wind’s velocity profile model, $\rho_{\text{air}}$, $C_{\text{fg}}$, $C_{\text{dy}}$, and $A_f$ were selected the same as those adopted in analysis using boundary layer wind. As for Oseguera and Bowles’ model, the number of parameters is simplified into 3 ($p$, $z^*$, and $z^*$) by defining:

$$p = \frac{(\lambda R^2)}{2r}[1 - e^{-(r/R)^2}]$$  \hspace{1cm} (16)

![Figure 5. Difference between wind-induced moment and bending capability](image5.png)

![Figure 6. Profile models of different wind](image6.png)

<table>
<thead>
<tr>
<th>Height (m)</th>
<th>0.00</th>
<th>7.50</th>
<th>15.00</th>
<th>22.25</th>
<th>29.25</th>
<th>36.25</th>
<th>43.00</th>
<th>48.25</th>
<th>55.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon$ (m)</td>
<td>30.00</td>
<td>7.50</td>
<td>15.00</td>
<td>22.25</td>
<td>29.25</td>
<td>36.25</td>
<td>43.00</td>
<td>48.25</td>
<td>55.50</td>
</tr>
<tr>
<td>$p$</td>
<td>89.5</td>
<td>457.24</td>
<td>8.967</td>
<td>8.069</td>
<td>7.175</td>
<td>6.329</td>
<td>5.535</td>
<td>4.779</td>
<td>4.085</td>
</tr>
<tr>
<td>$z^*$ (m)</td>
<td>30.00</td>
<td>445.06</td>
<td>8.967</td>
<td>8.068</td>
<td>7.173</td>
<td>6.326</td>
<td>5.531</td>
<td>4.779</td>
<td>4.081</td>
</tr>
</tbody>
</table>

![Table 2. Section moment under Osegua profile downburst and normal wind](image2.png)

The parameters of three types of downburst velocity profiles are calculated by repeated pilot calculation through dichotomy, until structural bottom’s section moment reaches ultimate section bending capability. Listed in Tab.2 are the various combinations of different parameters of the Osegua profile model and wind-induced moments at different heights, where the bottom moment reach the ultimate state. Vicroy profile model and Wood profile model’s results are similar as Osegua’s, and the data tables are omitted. In addition, wind-induced moments at different heights are compared with its corresponding ultimate section bending capability as listed in Tabs. 2 with the moments beyond the ultimate being emphasized.
by red italics. It is seen from the tables that the downburst wind loads inducing section moments larger than the ultimate is mainly located in the area of 0-40m, which reveals that the damage of weak intensity would be occurred in this altitude range, while the damage of weak intensity under normal wind loads would be occurred in the bottom of the structure since section moments above the bottom are always less than ultimate section bending capability. The results are consistent with observations in some field investigation on the collapse of wind-excited transmission tower.

Since section moment have over 7 orders of magnitude in standard unit, the difference between wind-induced moment and ultimate moment is not obvious. Herein, ultimate moments are subtracted from wind-induced moments, then damage would be observed in the place where the subtract result is more than 0. The results of one work condition are indicated in Fig. 5. The maximum velocity of three downburst model and boundary layer wind, occurring when bottom’s sectional moment reach ultimate section bending capability, are shown in Fig. 6. These results indicate that normal wind’s velocity increase with height while downburst’s maximum velocity increases with height firstly and then decreases after reaching its peak value in the altitude scope of most high-rise structures. Therefore, the downburst wind loads are more adverse than normal wind loads.

4 Conclusions

Owing to the great differences between the vertical profiles of downburst and normal wind, wind-induced responses of the tower present distinct difference. Downburst is more adverse to the safety of towers than normal wind load.

The analytical framework for studying the transmission tower’s section wind-resistance capability and damage regions proposed in this paper can be utilized to identify the weak places and damage characteristics of high-rise structures subjected to various horizontal loads if only the distribution of horizontal loads and structural ultimate section bending capability were determined.

Case study verified the fact that the damage of most collapsed high-rise transmission tower under downburst is observed not at the bottom but in the altitude scope about 0-40m. The damage of weak intensity under normal wind loads would be occurred at the bottom of the tower. These results are consistent with field investigations on the collapse of some wind-induced transmission.

References