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Siu-Lai Chan
Department of Civil and Environment Engineering, The Hong Kong Polytechnic University

Zhi-Xiang Yu
School of Civil Engineering, Southwest Jiaotong University

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Preface

These proceedings contain the papers presented at the TENTH INTERNATIONAL CONFERENCE ON ADVANCES IN STEEL STRUCTURES (ICASS 2020) held in Chengdu, China, from 21 to 23 August 2022. The international conference series on Advances in Steel Structures was initiated in 1996 under the support of The Hong Kong Polytechnic University, which remains very active in fostering its continuation-joined a few years later by the Hong Kong Institute of Steel Construction.

These proceedings bring together most recent findings in numerical, theoretical and experimental research, as well as its practical implementation in design practice in the areas of Assembled Structure, Bridge, Cold-formed Steel, Composite, Connections, Corrosion, Fracture & Collapse, Design & Analysis, Direct Analysis, Fatigue, Fire, High-Strength Steel, Impact and Protection, Intelligent Construction, New Material, Seismic Resistance, Stability, Stainless Steel, Structure Systems, Testing & Monitoring. The papers presented in these proceedings come from a wide range of countries/regions and will be a great reference source.

Specially, the subject matter has been categorized under the broad heading of:

**Volume I:** Keynotes Lectures, Assembled Structure, Bridge, Cold-Formed, Composite, Connections, Corrosion, Fracture & Collapse, Design & Analysis, Direct Analysis, Fatigue


Each of the papers was subjected to stringent review by a panel of experts in the respective area. This peer review began with an assessment of the submitted abstracts and following this, authors were invited to submit their full manuscripts. Each manuscript was then carefully reviewed by relevant experts, and their recommendations on accepting, rejecting or modifying the submissions were strictly adhered to, before inclusion in the conference proceedings.
ANTHI-WIND CAPACITY CHECK AND COLLAPSE ANALYSIS OF EXISTING TRANSMISSION TOWER

W.T. Zhang 1*, Y.Q. Xiao 1, C. Li 1 and Q.X. Zheng 2

1 School of Civil and Environmental Engineering, Harbin Institute of Technology, Shenzhen, China
E-mails: zhangwt1990@126.com, xiaoyq@hit.edu.cn, lichaosz@hit.edu.cn

2 Shenzhen General Institute of Architectural Design and Research Co. Ltd, Shenzhen, China
E-mail: 984431630@qq.com

Abstract: With the implementation of the new code for the design of overhead transmission line in China, the design check, maintenance and reinforcement for the existing transmission towers are indispensable. In this paper, based on an existing transmission line project, firstly, the differences of the wind load calculations between different design codes are analyzed contrastively, and the safety stock of the tower primary members is checked. Then, the tower’s ultimate bearing capacity is analyzed, in which the parameters of the fiber hinges are determined by the solid element model simulation. Last, the tower collapse process is simulated, and a similar accident case is investigated briefly. It is revealed that by the new code, the evaluated safety stock of this existing tower decreases obviously; at the 1/3 of the tower height and the position under the lowest crossarms, the primary members perform as the weak parts; and the progressive collapse resistance of the tower is low, which is triggered by the primary member buckling. Therefore, to reinforce the weak parts of the existing towers, increase the redundancy to resist a local failure and avoid the total collapse of the entire structural system is imperative.

Keywords: Transmission tower; wind load; safety stock; ultimate bearing capacity; collapse analysis

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1 INTRODUCTION

High-voltage overhead transmission line is an important type of lifeline project and highly concerned by the community. This type of structure is very sensitive to wind load due to its light mass and low damping. Recently, severe weather, such as strong typhoon or convective storm, happens frequently, which increases the probability of wind-induced damage of the electric power infrastructure [1]. Therefore, it is very worthy to study the wind load and the collapse mechanism of transmission tower, which will help improve the wind-resistance design and the intelligent decision support for the wind damage evaluation.

Although almost all of the design codes over the world determine the wind load of the transmission line by the equivalent static method [2][3][4], the obtained values may be different even for a same tower, because of the adopted different theories and parameters. In China, compared with the other current design codes for transmission line, DL/T5551-2018 introducing the wind vibration coefficient to concern the pulsation effect of the transmission line provides a more rational way to determine the wind load. However, different design values and distribution forms of the wind load will obtain different assessment results of the structural anti-wind capacity. Concerning that the wind load distribution is random in fact, it can be
recommended to take several distribution forms of wind load to design the towers and check their capacity, which can help increase the reliability of transmission line [5][6].

The wind-induced damage of transmission line is happening frequently, which reflects that some inherent defects may exist in the current design theory. Some full-scale tests of failure mechanism reveal that the failure is mostly triggered by the buckling of the main chord components; the capacity is also relevant to the layout of the web members [7][8][9]. Even though, the research is still not enough. The angle steel is widely used in lattice towers, and the tubular and T-section steel are also often used in some special projects. It is acknowledged that the buckling effect of the compression bars made of the above steel is very complex, which is influenced by the residual stress in cross section, initial bending of straight rod, joint construction and so on, so that it can’t be fully and exactly concerned in the numerical simulations [10]. The more rational model of the equivalent constitutive relation is very necessary to be concerned in the future research [11][12].

In this paper, based on an existing 220kV transmission line project, firstly, the wind load calculated by two different current codes are analyzed contrastively, the internal force of the primary members and their safety stock are checked under the designed wind load. Then, the fiber hinge model is introduced to concern the material nonlinearity and its parameters are obtained by the solid element model simulation. With this, the tower’s ultimate bearing capacity is analyzed under different wind load distributions. Besides, the collapse process of the tower is simulated by the alternate path method, and a similar accident case is investigated briefly. Last, some conclusions are conducted.

2 CONSIDERED TRANSMISSION LINE PROJECT

In this work, the considered 220kV transmission line project is located in Guangdong province, China. It was designed in 2007 based on the code for design of 110kV~750kV overhead transmission line, DL/T 5092-1999, which has been abolished. The terrain roughness around the line belongs to the type B according to Chinese load code for the design of building structures, GB50009-2012, and the corresponding basic wind speed is 32.5m/s. It is noteworthy that the line has been working for 14 years which is very near to its design reference period, i.e., 15 years.

The target tower, BZ361-24 shown in Figure 1, is an angle steel tower, its total height is 40.5m and its nominal height is 24m. The main components of the tower body and crossarms are Q345 steel, the others are Q235 steel. The type of ground wire is OPGW-48B1-120, the type of conductor wire is double bundled JL/LB20A-630/45, and the type of insulator is the single type I FXBW-220/100C. Their parameters are listed in Table 1 and Table 2.

![Figure 1: The transmission tower BZ361-24.](image)
Table 1: The specification of the angle steel.

<table>
<thead>
<tr>
<th>Segment</th>
<th>Chord member</th>
<th>Web member</th>
<th>Segment</th>
<th>Chord member</th>
<th>Web member</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>L56×4</td>
<td>L40×4</td>
<td>6</td>
<td>L110×7</td>
<td>L63×5、L50×4</td>
</tr>
<tr>
<td>2</td>
<td>L70×5、L63×5</td>
<td>L45×4</td>
<td>7</td>
<td>L110×8</td>
<td>L63×4、L64×4</td>
</tr>
<tr>
<td>3</td>
<td>L75×5</td>
<td>L45×3</td>
<td>8</td>
<td>L110×10</td>
<td>L56×4、L50×4</td>
</tr>
<tr>
<td>4</td>
<td>L75×5、L70×6</td>
<td>L45×3</td>
<td>9</td>
<td>L110×10</td>
<td>L56×4</td>
</tr>
<tr>
<td>5</td>
<td>L75×5</td>
<td>L45×4</td>
<td>10</td>
<td>L125×10</td>
<td>L63×5、L56×4</td>
</tr>
</tbody>
</table>

Table 2: The physical parameters of wires and insulator.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Ground wire</th>
<th>Single conductor</th>
<th>Insulator</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter / (m)</td>
<td>0.0115</td>
<td>0.0336</td>
<td>0.250</td>
</tr>
<tr>
<td>Lineal weight / (kg/m)</td>
<td>0.587</td>
<td>2.007</td>
<td>121.46</td>
</tr>
<tr>
<td>Elastic modulus / (GPa)</td>
<td>121.5</td>
<td>65</td>
<td>280</td>
</tr>
<tr>
<td>Rated breaking force / (kN)</td>
<td>837</td>
<td>143.9</td>
<td>100</td>
</tr>
<tr>
<td>Designed length / (m)</td>
<td>350</td>
<td>350</td>
<td>2.45</td>
</tr>
</tbody>
</table>

3 ANTI-WIND DESIGN CHECK OF TRANSMISSION TOWER

In China, the current national code for 110kV~750kV overhead transmission line design, GB50545-2010, and the power industry code, DL/T5154-2012, have no definitely distinction from DL/T 5092-1999 in the part of the wind load determination. However, the load code for the design of overhead transmission line implemented in 2018, i.e., DL/T 5551-2018, induces some big changes. In this section, the calculated wind loads on tower by DL/T5154-2012 and DL/T5551-2018 are compared, and the safety stock of the tower is also analyzed.

3.1 Wind load on transmission tower

The wind load on tower includes two parts mainly, i.e., the load transferred from the wires $F_t$ and the load directly acting on the tower $F_s$, which can be respectively expressed as

$$F_t = \mu_z \mu_{sc} \beta_c \alpha_L d L_p \sin^2 \theta W_0$$

(1)

$$F_s = \mu_z \mu_s A_s W_0$$

(2)

where $\mu_z$ is the height variation factor of wind pressure; $\mu_{sc}$ and $\mu_s$ are the shape factors of the wire and the tower respectively; $\beta_c$ is the pulsation coefficient of wire; $\alpha_L$ is the span factor of wires; $\beta_s$ is the vibration factor of the tower along wind direction; $d$ is the diameter of the wires, unit: m; $L_p$ is the length of the horizontal span, unit: m; $\theta$ is the angle between the wind direction and the wires, unit: °; $A_s$ is the wind aero of the tower, unit: m²; $W_0$ is the reference wind pressure, unit: kN/m². The parameters are analyzed comparatively below.

(1) Actually, $\mu_z$ is a revision based on the mean wind profile model, determined by $\mu_z = b \left( \frac{z}{10} \right)^{-2a}$, where $b=1.0$ for the type B terrain; the factor $a$ in DL/T5551-2018 has been decreased from 0.16 to 0.15 compared with DL/T5154-2012.

(2) The values of $\mu_{sc}$ are shown in Table 3. It is found that the values from DL/T5551-2018 are smaller. The main reason is that DL/T5551-2018 refers to the latest research conducted by domestic researchers, besides, the foreign codes also reflect the similar results. On the other hand, although $\mu_s=1.3(1+\eta)$ is adopted in both codes, in which $\eta$ is a reduction factor, the calculated values of $\mu_s$ are less than the test results and those by the other foreign codes [2].

(3) The wind-induced vibration coefficient of the wire, $\beta_{zd}$, can be expressed as the product of $\alpha_L$ and $\beta_c$, i.e., $\beta_{zd}=\alpha_L \beta_c$. 

465
<table>
<thead>
<tr>
<th>Diameter</th>
<th>DL/T5154-2012</th>
<th>DL/T5551-2018</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;17 mm</td>
<td>1.2</td>
<td>1.1</td>
</tr>
<tr>
<td>≥17 mm</td>
<td>1.1</td>
<td>1.0</td>
</tr>
</tbody>
</table>

For DL/T5154-2012, the uneven coefficient $\alpha_L$ is to consider the influence of the spatial correlation effect of wind speed, which is less than 1.0 and decreases as the basic wind speed increases. $\beta_c$ is to concern the possible amplification effect of dynamical response of the wire, where $\beta_c > 1.0$ for the 500 kV and higher voltage transmission lines and $\beta_c = 1.0$ for the other lower voltage lines. It is noteworthy that both of them are determined by engineering experience.

![Figure 2: The calculation model of wires.](image)

For DL/T5551-2018, $\beta_{zl}$ is obtained by rigorous derivation based on the structural random vibration theory. For the model of lines shown in Figure 2, the wind load on the middle support is transferred from the two spans of lines and its wind-induced vibration coefficient is calculated by $\beta_{zl} = \frac{\overline{\gamma}_{Dk} + \hat{\gamma}_{Dk,b}}{\overline{\gamma}_{Dk}}$, in which $\overline{\gamma}_{Dk}$ is the standard value under the mean load; $\hat{\gamma}_{Dk,b}$ is the background response value under the fluctuating wind force. For $L_x = L_y = L_P$, it can be obtained

$$
\beta_{zl} = 1 + 2g \varepsilon_c I_z \sqrt{\frac{12 L_x L_p^4 + 54 L_x^4 L_p - 36 L_x^4 L_p - 72 L_x^4 e^{L_p/L_x} + 18 L_x^4 e^{2L_p/L_x}}{3 L_p^2}}
$$

in which $g$ is the peak factor, $g = 2.5$; $\varepsilon_c$ is the reduction factor; $I_z$ is the turbulence intensity at the height $H = z$; $L_x$ is the integral length of the correlation function of the wind speed along the direction $X$; $e$ is the universal constant.

For practical application, by decomposing into $\beta_{zl} = \alpha_L \beta_c$, it obtains

$$
\beta_c = \gamma_c \left[ 1 + 2g \varepsilon_c I_z \left( \frac{z}{10} \right) \right]
$$

$$
\alpha_L = \frac{1 + 2g \varepsilon_c I_z \delta_L}{1 + 5I_z}
$$

in which $\gamma_c$ is also a reduction coefficient; $\delta_L$ is called as the integral factor of the spans correlation, namely

$$
\delta_L = \frac{\sqrt{12 L_x L_p^4 + 54 L_x^4 L_p - 36 L_x^4 L_p - 72 L_x^4 e^{L_p/L_x} + 18 L_x^4 e^{2L_p/L_x}}}{3 L_p^2}
$$
As far as BZ361-24 is concerned, the calculated $\beta_{zl}=0.75 < 1.0$ by DL/T5154-2012; $\beta_{zl}=1.19$ for the ground wire and $\beta_{zd} = 1.21$ for the conductors by DL/T5551-2018. It is obviously irrational that the obtained wind load by DL/T5154-2012 is less than the mean wind load.

(4) In DL/T5551-2018, the lumped mass model is proposed to analyze the wind-induced vibration coefficient $\beta_z$ for all types of the towers referring to GB50009-2012. However, in DL/T5154-2012, this is proposed only for those towers higher than 60m referring to GB50009-2002, and for the lower towers, $\beta_z$ is set to one same value for one tower. Even though, for the designed towers higher than 60m based on DL/T5154-2012, the calculated $\beta_z$ may be different from that by DL/T5551-2018, since the adopted peak factor $g$ and the reference scale of turbulence $I_{10}$ in GB50009-2002 are different from GB50009-2012.

For BZ361-24, the obtained $\beta_z$ by DL/T5154-2012 is 1.35, the calculated $\beta_z$ at different height by DL/T5551-2018 is shown in Table 4. It can be found that the $\beta_z$ of Segments 6, 4, 2 and 1, i.e., the low, middle, upper horizontal crossarms and the tower head, fluctuates from 1.88 to 2.35; the $\beta_z$ of the segments below the lowest crossarms increases from 1.0 to 1.41; the others are 1.40 and 1.47 respectively. Therefore, it can be inferred that $\beta_z$ in DL/T5154-2012 would be underestimated for the upper segments.

Table 4: The obtained values of $\beta_z$ by DL/T5551-2018.

<table>
<thead>
<tr>
<th>Segment</th>
<th>Height/m</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.0</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>11.7</td>
<td>1.08</td>
</tr>
<tr>
<td>3</td>
<td>17.5</td>
<td>1.22</td>
</tr>
<tr>
<td>4</td>
<td>22.9</td>
<td>1.41</td>
</tr>
<tr>
<td>5</td>
<td>24.5</td>
<td>2.23</td>
</tr>
<tr>
<td>6</td>
<td>29.0</td>
<td>1.40</td>
</tr>
<tr>
<td>7</td>
<td>31.0</td>
<td>2.35</td>
</tr>
<tr>
<td>8</td>
<td>33.8</td>
<td>1.47</td>
</tr>
<tr>
<td>9</td>
<td>37.0</td>
<td>1.88</td>
</tr>
<tr>
<td>10</td>
<td>39.0</td>
<td>2.27</td>
</tr>
</tbody>
</table>

(5) The partition coefficient of wind load is adopted to assign the skewed wind load, rather than direct resolution of the wind vector, shown in Table 5 and Figure 3.

Table 5: The partition coefficient of skewed wind load.

<table>
<thead>
<tr>
<th>Wind angle $\theta(\degree)$</th>
<th>Wind load from wire $X_Y$</th>
<th>Wind load on tower body $X_Y$</th>
<th>Wind load on crossarm $X_Y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>$0.25W_X$</td>
<td>$0$</td>
<td>$0$</td>
</tr>
<tr>
<td>45</td>
<td>$0.5W_X$</td>
<td>$0.15W_X$</td>
<td>$0.424(W_{sa}+W_{sb})$</td>
</tr>
<tr>
<td>60</td>
<td>$0.75W_X$</td>
<td>$0$</td>
<td>$0.747W_{sa}+0.249W_{sb}$</td>
</tr>
<tr>
<td>90</td>
<td>$W_X$</td>
<td>$0$</td>
<td>$W_{sa}$</td>
</tr>
</tbody>
</table>

Figure 3: Skewed wind diagram.

In Table 5, the bold coefficients in parentheses are adopted in DL/T5154-2012 and the out are adopted in DL/T5551-2018; the items without parentheses marked means that they are adopted in both codes. It can be found that the partition coefficients of wind load from the wire and those on the tower body aren’t changed. For the wind load on crossarms, the corresponding partition coefficients on direction $X$ are changed from 0.4 to 0.35 at angles 45°, 0.4 at 60°, 0.45 at 90°, respectively; on direction $Y$, it is reduced from 0.7 to 0.55 at angle 60°. Hence,
DL/T5551-2018 raises the wind load on crossarms at angle 90° and reduces at angles 45° and 60°.

For BZ361-24, it is judged that the worst wind direction is $\theta=90°$. The corresponding calculated wind loads from the wires and on the tower at different heights are shown in the Figure 4. It can be found that the trends of wind load distribution along the height obtained by both codes are similar. However, for the loads from wires, the values by DL/T5551-2018 are 41.9%–46.1% greater than by DL/T5154-2012. For the loads on the tower body, below the height 22.9m, the values by DL/T5551-2018 are less than by DL/T5154-2012, and as the height increases, the deviation decreases from 25% to 9%; on the crossarms, the values by DL/T5551-2018 are 39.4%–74.5% greater than by DL/T5154-2012.

![Figure 4: The calculated wind load on BZ361-24.](image)

3.2 Structural linear static analysis

In this work, using the finite element analysis software SAP2000, the tower model is established. The rod of the tower is simulated by the frame element, its direction is adjusted correspondently with actual orientation, and the feet nodes are all fixed. Acting the calculated wind loads $F_s$, $F_l$ and the gravity of the wires $G_l$ on BZ361-24, shown in Figure 5(a), the structural linear static analysis is conducted.

![Figure 5. The linear static analysis of BZ361-24](image)

Under the gravity and wind loads, the stress distribution result of the tower by DL/T5154-2012 is shown in Figure 5(b) and the result by DL/T5551-2018 is shown in Figure 5(c). It is obviously reflected that the stress values of case (c) are larger than case (b), especially on the primary chord members circled in Figure 5(c).
For BZ361-24, the results of the axial force of the main compression members on the tower body are listed in Table 6, where $F_k$ is the calculated buckling strength based on the Chinese code for the design of steel structure GB50017-2017; $F_{12}$ and $F_{18}$ are the calculated axial forces by DL/T5154-2012 and DL/T5551-2018, respectively; $K = F_k/F$. As shown in Table 6, $F_{18}$ is 1.36–1.43 times greater than $F_{12}$ in general; correspondingly, the evaluated degree of safety by DL/T5551-2018 is lower, since the $K_{12}$ ranges from 1.93 to 3.48 and the $K_{18}$ ranges from 1.38 to 2.49. According to the distribution of $K$, it is found that the rod under the lowest crossarms is the most unsafe position, which is also with the smallest slenderness ratio $\lambda = 39.9$.

Table 6. Results of the main compression members

<table>
<thead>
<tr>
<th>Type</th>
<th>Slenderness ratio $\lambda$</th>
<th>$F_k$ (kN)</th>
<th>DL/T5154-2012</th>
<th>$F_{12}$ (kN)</th>
<th>$K_{12}$</th>
<th>DL/T5551-2018</th>
<th>$F_{18}$ (kN)</th>
<th>$K_{18}$</th>
<th>$F_{18}/F_{12}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>L75×6</td>
<td>57.6</td>
<td>275.0</td>
<td>86.0</td>
<td>3.20</td>
<td>122.6</td>
<td>2.24</td>
<td>1.43</td>
<td></td>
<td></td>
</tr>
<tr>
<td>L100×7</td>
<td>56.4</td>
<td>387.4</td>
<td>111.2</td>
<td>3.48</td>
<td>155.3</td>
<td>2.49</td>
<td>1.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>L110×8</td>
<td>46.2</td>
<td>400.7</td>
<td>195.0</td>
<td>2.06</td>
<td>275.1</td>
<td>1.46</td>
<td>1.41</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>51.4</td>
<td>512.0</td>
<td>225.6</td>
<td>2.27</td>
<td>314.8</td>
<td>1.63</td>
<td>1.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>L110×10</td>
<td>51.2</td>
<td>512.3</td>
<td>236.2</td>
<td>2.17</td>
<td>329.2</td>
<td>1.56</td>
<td>1.39</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>44.3</td>
<td>523.0</td>
<td>270.7</td>
<td>1.93</td>
<td>377.8</td>
<td>1.38</td>
<td>1.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>39.9</td>
<td>714.7</td>
<td>296.6</td>
<td>2.41</td>
<td>414.4</td>
<td>1.72</td>
<td>1.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>L110×10</td>
<td>46.6</td>
<td>702.1</td>
<td>307.0</td>
<td>2.29</td>
<td>427.4</td>
<td>1.64</td>
<td>1.39</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>54.0</td>
<td>685.8</td>
<td>318.2</td>
<td>2.16</td>
<td>440.2</td>
<td>1.56</td>
<td>1.38</td>
<td></td>
<td></td>
</tr>
<tr>
<td>L125×10</td>
<td>51.3</td>
<td>692.1</td>
<td>336.7</td>
<td>2.06</td>
<td>462.5</td>
<td>1.50</td>
<td>1.37</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>49.2</td>
<td>791.6</td>
<td>337.9</td>
<td>2.34</td>
<td>460.3</td>
<td>1.72</td>
<td>1.36</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4 ULTIMATE BEARING CAPACITY ANALYSIS OF TRANSMISSION TOWER

Generally, the trigger of the wind-induced transmission tower collapse is the buckling of the compression member. In this work, the fiber hinge [13] is introduced into the tower model to consider the influence of buckling effect during the nonlinear static analysis process.

4.1 Fiber hinge model of the angle steel

4.1.1 Fiber hinge model

Fiber hinge is a kind of plastic hinges, which integrally computes the axial force and bending moment of the cross-section based on the plane cross-section assumption and the constitutive relation of the fiber material. It only works after the members yield or buckle in the process of the nonlinear static analysis or the nonlinear dynamical direct integral time-history analysis.

For application, multi-stages linear model is adopted to define the fiber hinge. As shown in Figure 6 and Eq.(7), it allows that the fibers keep a low stress rather than broke after arriving at the ultimate tension or compression strain. It is noted that $\sigma$ and $\varepsilon$ represent the stress and strain of the material respectively; $E$ is the Young modulus; $E_t$ is the tangent modulus after yielding; $\sigma_y$ and $\varepsilon_y$ are the yield stress and strain respectively; $\sigma_u$ and $\varepsilon_u$ are the ultimate stress and strain respectively; $\sigma_{cr}$ and $\varepsilon_{cr}$ are the equivalent critical buckling stress and strain respectively; $E_p$ is the equivalent tangent modulus after buckling; $\varepsilon_u$ is the ultimate compression strain; $\sigma_i$ and $\sigma_p$ are the residual stress after the ultimate tension and compression strain respectively.

4.1.2 Determination of the parameters for single angle steel

In order to determine the coefficients in Eq.(7) with respect to single angle steel, the solid element models of angle steel components are established using the software ANSYS, and the
cell size is set to the limb thickness $t$. The multi-points coupling method is used to simulate the hinge supports in the end of the component; the steel material is assumed as the ideal elastic-plastic that the yield strength is set to $f_y = 345 \text{ MPa}$ for Q345, the Young modulus $E = 2.06 \times 10^5 \text{ MPa}$, the Poisson ratio $\nu = 0.3$.

$$\begin{align*}
\sigma &= \begin{cases}
\sigma_y, & \varepsilon_y \leq \varepsilon \\
\sigma_y + E \varepsilon, & 0 \leq \varepsilon \leq \varepsilon_y \\
E \varepsilon, & \varepsilon \leq \varepsilon_y \\
\sigma_{cr} + E_p (\varepsilon - \varepsilon_{cr}), & \varepsilon_{cr} \leq \varepsilon \leq \varepsilon_{cr} \\
\sigma_{cu}, & \varepsilon \leq \varepsilon_{cu}
\end{cases} \\
&= \begin{cases}
\sigma_y, & \varepsilon_y \leq \varepsilon \\
\sigma_y + E \varepsilon, & 0 \leq \varepsilon \leq \varepsilon_y \\
E \varepsilon, & \varepsilon \leq \varepsilon_y \\
\sigma_{cr} + E_p (\varepsilon - \varepsilon_{cr}), & \varepsilon_{cr} \leq \varepsilon \leq \varepsilon_{cr} \\
\sigma_{cu}, & \varepsilon \leq \varepsilon_{cu}
\end{cases}
\end{align*}$$

Figure 6: Constitutive relation of the fiber material.

Figure 7: The residual stress distribution in cross section of angle steel

Figure 8: The ultimate bearing capacity comparison of angle steel components
On the other hand, the critical initial imperfections are also concerned, that the initial bending shape is set to the first order modal shape of their characteristic buckling and the corresponding amplitude is set to \(L/1000\); the residual stress distribution in the cross section is set to the type shown in Figure 7, with the amplitude \(\beta=0.1\) in this paper.

Using the arc length loading method, conduct the nonlinear static analysis under tension and compression respectively, obtain the ultimate bearing capacity \(F_u\) and plot the Force-Displacement \((F-D)\) graph, as shown in Figure 9. The obtained \(F_u\) are compared with \(F_k\) obtained by GB50017-2017, as shown in the Figure 8. Since the initial imperfections behave randomly in fact, the deviation in this work ranging from -6.5% to 6.8% is rational.

Take the two angle steel components at the weak parts of the tower for example and their obtained \(F-D\) curves by ANSYS are shown in the Figure 9. In order to define the character parameters in the model of fiber hinge, \(4\Delta\) and \(8\Delta\) are selected as the reference points, where \(\Delta\) is the critical yielding or buckling displacement, as shown in Figure 9.

![Figure 9. The Force-Displacement \((F-D)\) graph](image)

### 4.2 Structural nonlinear static analysis under wind load

Concerning the influence of the existing installation error and other random factors, the initial geometry of the tower is revised by plus the first order buckling modal shape and the corresponding amplitude is set to \(H/1000\). Besides, the defined fiber hinges are inserted into the respective rods in order to concern the influence of material nonlinearity. Push-over analysis is conducted on the tower under the wind loads in different distributions obtained by DL/T5154-2012 and DL/T5551-2018 respectively. The failure mode in forms of the displacement is shown in Figure 10. The structural performance curves are shown in Figure 11, in which the horizontal axis represents the displacement \(D_t\) of the tower top and the vertical axis represents the total reaction force \(F_b\) of the foundation.

As shown in Figure 10, the failure of tower is triggered by the buckling of members at about \(H/3\) height above the ground in both cases. However, it can be found that the tower performs in different behaviors of \(F_b-D_t\) under different distributions from the Figure 11, that it performs larger ultimate displacement and lower ultimate resistance by DL/T5551-2018 than by DL/T5154-2012. By translating, the largest wind speeds at the ultimate states are 39.5m/s for DL/T5551-2018 and 42.6m/s for DL/T5154-2012 respectively.

The property points at the design load level are also marked in Figure11. It is obvious that the surplus degree \(K_u=F_{bu}/F_{bk}\) evaluated by DL/T5154-2012 is larger than by DL/T5551-2018, i.e., \(K_{u12}=2.03\) and \(K_{u18}=1.43\). It is noteworthy that the evaluated weakest rods \((\lambda=39.9)\) in the linear static analysis does not broke first. Because the obtained buckling strength by the solid element model is larger than the calculated buckling strength by GB50017-2017 and the geometric nonlinearity and the initial imperfection may affect the failure way of the tower.
5 COLLAPSE ANALYSIS OF TRANSMISSION TOWER

In this section, the alternate path method (APM) [13] is adopted to analyze the collapse of the tower under static wind load. Assume that the wind load is kept unchanged at the moment when the tower failure is triggered. Based on the structural shape and stiffness at the ultimate limit state obtained by Push-over analysis, demolish the goal members which triggers the fiber hinges at 0.1s and apply the concentrated forces to both end nodes of the member meanwhile. Then, reduce the applied forces to zero gradually within 0.1s. The process of the collapse is shown in Figure 12.

From Figure 12, the displacement of the tower top is 0.447m at the beginning, after demolishing the first buckling rods, the displacement increased rapidly and at the 0.6s, it has been 0.981m exceeding the restricted value $H/50 = 0.81$m. It also can be seen that the diagonal members are bent down and lose the bearing capacity, then the compression force transferring path is cut off and the tension members are bent directly. On the other hand, since the load can’t be transferred to the tower legs with the path cut off, the legs deforms slightly. Therefore, this tower has very little of the resistant redundancy to progressive collapse.

In 2014, a strain section of a transmission line project collapsed under the Typhoon Rammasun, in which the towers are the same type with BZ361. A typical case of the collapsed tower is shown in Figure 13, the crossarms with wires inserted vertically into the ground in the same direction with the tower falling, the main chord members and the web members buckled, and no obvious damage happened at the tower feet and foundation. Hence, it can be deduced
that when the tower was destroyed, the external loads should orthogonally act on the transmission line, and the collapse was triggered by the buckling of member at H/3 of the tower. Since the landform around the accident place is flat and with few of vegetation, the wind field wouldn’t been weaken to some extent. According to the measured data of the Rammasun from the nearby meteorological station, the max mean wind speed at the reference height 10m in 10min ranged in the domain 35~42.7m/s. Hence, the site investigation findings are consistent with the analysis results above.

As mentioned above, for this type of tower, the position at the H/3 of the tower is comparatively weak and the necessary reinforcement measures should be conducted to improve the surplus degree of safety.

![Figure 12. Collapse process of the tower BZ361-24](image)

![Figure 13. The wind-induced collapse mode of a typical transmission tower](image)
6 CONCLUSIONS

In this paper, based on a practical transmission line project, the wind load calculation in different Chinese codes for the design of transmission line are compared, especially about the determination of the parameters. The linear and nonlinear static analysis are respectively conducted on the tower and the fiber hinge model is introduced to simulate the buckling effect. The progress of the tower collapse is re-performed by APM and the site investigation about an actual accident is conducted briefly. The following conclusions can be obtained. (1) DL/T5551-2018 performs more rational in the wind load calculation, and it raises the design wind load compared with DL/T5154-2012. (2) The wind load distributions obtained by DL/T5551-2018 and DL/T5154-2012 are different, which will affect the distribution of the internal force. (3) DL/T5154-2012 may over-evaluate the resistant capacity of the tower to wind and let the designed tower stay below the expected level of safety. (4) The fiber hinge model performs well in the nonlinear static analysis of the angle steel tower. (5) The anti-wind capacity check of the existing transmission towers is very necessary and the corresponding reinforcement measures should be adopted to raise their safety degree.

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REFERENCES

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