Study on the secondary stresses of joint effects in transmission tubular towers

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Abstract Transmission tubular towers are widely used in urban and large span transmission network, analyzed as ideal frame-truss systems without explicitly considering slippage effects in bolted joints. In this paper, a special beam element considering bolt slip effect is established based on the secondary development module UPFs provided by ANSYS software. A series of semi-rigid model of transmission tubular tower with different parameter features are simulated by directly calling the special beam elements instead of using spring elements and ordinary beam elements. Distribution form and influence factor of secondary stresses of main members in transmission tubular tower leg considering the bolt slippage effect is analyzed. The results indicate that secondary stresses are mainly related to three factors: the layout scheme of auxiliary members, the main member slenderness ratio, the angle between main members and diagonal members. By contrast, it is found that secondary stresses of two lattices are mainly larger than that of a lattice. Under two lattices the angle between the main member and the diagonal member is the major influence factor, while, a lattice, the secondary stresses are mainly depended on the main member slender ratio. Design suggestions are given based on the analysis results.

1. Introduction

Lattice transmission towers are vital components of overhead transmission lines which plays an important role in the operation of electrical power systems. The power industry is moving toward the use of high-voltage transmission lines to meet the demand for greater transmission capacity. This results in taller, more slender transmission towers that are subjected to heavier loads and undergo larger displacements, and then steel pipes are more inclined to be chosen as main member of transmission towers. Transmission tubular towers are widely used in urban and large span transmission grids. In China, maybe in other related area, tower designers still tend to adopt three-dimensional truss model in the design of transmission tubular towers, because of convenient calculation and clear force transmission path. Meanwhile, end joint stiffness effect of members is usually considered through adjusting slenderness ratio, which may lead to conservative result. In some literatures, the stresses corresponding to the moment caused by joint stiffness in truss structures called secondary stress which may cause the strength failure in the member end. It is necessary to investigate the secondary stress magnitude for the design margin.

Fine analysis has become the new trend of transmission tower research, furthermore, the bolt slip effect has become a new research hotspot. Traditionally, lattice towers are analyzed as ideal trusses or frame-truss systems without explicitly considering slippage effects in bolted joints. Such effects are always observed in full-scale tower tests and introduce great differences in the ultimate bearing
capacity and failure modes obtained from classical linear analysis models. Knight and Santhakumar [1] conducted tests on a full-scale quadrant of the lowest panel of a transmission tower and compared the measured results with the classical analysis results. They pointed out that the secondary stresses caused by bolted joint effects could be significant enough to cause failure of leg members even under normal working-load conditions.

In this paper, a new special beam taking bolt slip effect into account is established and included in the element library using the secondary development module User Programmable Features (UPFs) provided by ANSYS software. The semi-rigid models of transmission tubular tower are built by directly calling the special beam elements, and the secondary stress magnitude and distribution patterns of main members are studied in conjunction with existing experimental results.

2. Study on the development of beam element based on the UPFs

2.1 UPFs introduction

Classic ANSYS mainly contains three secondary development tools: ANSYS Parametric Design Language (APDL), User Programmable Features (UPFs), User Interface Design Language (UIDL). UPFs are ANSYS capabilities to write users’ own routines. Using UPFs, user can tailor the ANSYS program for your organization’s needs. For instance, UPFs can be used to define a new material behavior, a special element, or a modified failure criterion for composites. The modified codes of UPFs need to be recompiled and connected on the Fortran or C compiler matching the ANSYS to generate customized version. One of the prominent advantages of using UPFs is that user can directly utilize the preprocessor, solver, and post-processor of ANSYS program.

At present, there are few literatures about UPFS, and the most authoritative one is still the help documents provided by ANSYS for UPFs modules: Guide to ANSYS User Programmable Features [2]. Because the secondary development of the element needs to call multiple function to access the ANSYS database, it is very important to clarify the parameters of the sub-functions. Most of the annotations for these parameters are scattered, with no particular set of instructions, which brings difficulty to subsequent development work.

2.1.1 Beam element large spatial rotation. Beam element has 6 or 7 degrees of freedom at each node. These include translations in the x, y, and z directions and rotations about the x, y, and z directions. The seventh degree of freedom warping magnitude is ignored in this paper. Taking geometric nonlinearity into account, under each iteration, the translational increments can be linearly superimposed to get the total displacement, while the rotation increment can be not conducted linear superposition.

Three-dimensional space rotation is more complicated than two-dimensional rotation. According to Euler’s theorem: Any rotation of a rigid body is equivalent to the rotation of a plane around a certain axis and given by a finite rotation vector. The definition and detailed calculation methods of Euler angle can be referred to the literature [3-5]. An approximate algorithm for rotation transformation around an arbitrary axis:

Given rotation axis \( A = (a_x, a_y, a_z) \) and rotation angle \( \theta \), the rotation matrix \( T_{\theta} \) can be determined as follows:

\[
\hat{A} = \begin{bmatrix}
a_x a_x & a_x a_y & a_x a_z \\
a_y a_x & a_y a_y & a_y a_z \\
a_z a_x & a_z a_y & a_z a_z
\end{bmatrix}
\]

\[
\hat{A} = \begin{bmatrix}
0 & -a_z & a_y \\
a_z & 0 & -a_y \\
-a_y & a_z & 0
\end{bmatrix}
\]

\[
T_{\theta} = \hat{A} + \cos \theta (I - \hat{A}) + \sin \theta \hat{A}
\]

Where: \( a_x, a_y, a_z \) is the direction cosine of the axis of rotation.

Under current load increment step, the rotation increment is \( \Delta \theta_x, \Delta \theta_y, \Delta \theta_z \), and the rotation vector is
defined as:

\[ A = (\Delta \theta_x / \theta, \ \Delta \theta_y / \theta, \ \Delta \theta_z / \theta) \]  

(3)

\[ \theta = \sqrt{\Delta \theta_x^2 + \Delta \theta_y^2 + \Delta \theta_z^2} \]  

(4)

Then, the rotation matrix \( T_0 \) can be got by substituting Eq. (3), (4) into Eq. (2).

2.1.2 Element coordinate transformation matrix. Considering geometric nonlinearity, the sections orientation of the elements constantly change as shown in Figure 1, so it is necessary to update the transformation matrix in each iteration (maybe in each incremental load step). For the convenience of calculation, the section coordinate system in the middle of the element is selected as the element coordinate system (ECS).

Element stiffness matrix is formed under ECS, and then transformed to the global coordinate system (GCS) to assemble global stiffness matrix. Transformation matrix \([R]\) indicates the relative relationship between ECS and GCS, expressed as:

\[
[R] = \begin{bmatrix}
[r]
[r]
[r]
[r]
\end{bmatrix} = \begin{bmatrix}
r_{11} & r_{12} & r_{13} \\
r_{21} & r_{22} & r_{23} \\
r_{31} & r_{32} & r_{33}
\end{bmatrix}
\]  

(5)

Where, \([r]\) is element orientation matrix. \( r_{ij}(i, j = x, y, z) \) represents the directional cosine of the angle between the ECS axis \( j \) and GCS axis \( i \). The first column of \([r]\) is the directional cosine of the angles between the ECS axis \( x \) and the GCS axis \( X, Y, Z \), expressed as:

\[
\begin{align*}
r_{11} &= \frac{x_j - x_i}{L} \\
r_{21} &= \frac{y_j - y_i}{L} \\
r_{31} &= \frac{z_j - z_i}{L}
\end{align*}
\]  

(6)

Under the current iteration, the length of the element can be expressed as:

\[ L' = \sqrt{(x_j' - x_i')^2 + (y_j' - y_i')^2 + (z_j' - z_i')^2} \]  

(7)

Where, \((x_i', y_i', z_i'), (x_j', y_j', z_j')\) refer to the nodes \( i, j \) coordinate in GCS.

In order to ascertain the second and the third column of \([r]\), that is, the directional cosine of the
angles between the ECS axis y, z and the GCS axis X, Y, Z respectively. Oran proposed the concept of "node reference line (NRL)", "node orientation matrix (NOM)" and "end section direction matrix (ESDM)". NRL is defined always orthogonal and fixed on the end nodes in the process of deformation to describe the space node rotation. NOM (expressed as Eq.8) is direction cosine matrix of NRL, of which each column represent the directional cosine of the angles between the NRL and the global coordinate axis X, Y, Z.

\[
\begin{bmatrix}
\alpha_{11} & \alpha_{12} & \alpha_{13} \\
\alpha_{21} & \alpha_{22} & \alpha_{23} \\
\alpha_{31} & \alpha_{32} & \alpha_{33}
\end{bmatrix}
\]  

(8)

It is usually assumed that NRL in the initial configuration are parallel to the global coordinate systems, so the initial NOM can be expressed as:

\[
[\alpha]_{i_0} = [\alpha]_{j_0} = \begin{bmatrix}
1 & 0 & 0 \\
0 & 1 & 0 \\
0 & 0 & 1
\end{bmatrix}
\]  

(9)

It is assumed that when \([\alpha]_i\) has been obtained at time \(t\), and the rotation increment of time \(t+\Delta t\) is \([\Delta\theta]_i\), then the node orientation matrix at time \(t+\Delta t\) can be obtained according to the Eq (2).

\[
[\alpha]_{i,t+\Delta t} = T_{\Delta\theta} [\alpha]_i
\]  

(10)

The first column of \([p]\) defined as ESDM is the directional cosine of the normal line of the member end section, and the other two columns are the directional cosine of section centroid spindle. According to the relationship between NOM, ESDM, and \([r]\), \([p]\) can be given as follows:

\[
[p]_{i_{\text{end}}} = [\alpha]_{i_{\text{end}}} [r]_{i_0}
\]

\[
[p]_{j_{\text{end}}} = [\alpha]_{j_{\text{end}}} [r]_{j_0}
\]

(11)

Under current iteration, the line connecting the nodes i, j is treated as the x axial of ECS. The angle between the x axial and the normal of the end section can be obtained. The average of the other two principal axes of the end section rotated according to the angle are treated as the y, z axial of ECS respectively. Details can be referred to literature [6,7]:

Because the deformation of the element is small, define:

\[
[e]_{i_{\text{end}}} = \begin{bmatrix}
1 & \Delta\theta_{j3} & -\Delta\theta_{j2} \\
-\Delta\theta_{j3} & 1 & 0 \\
\Delta\theta_{j2} & 0 & 1
\end{bmatrix}
\]

\[
[e]_{j_{\text{end}}} = \begin{bmatrix}
1 & \Delta\theta_{j3} & -\Delta\theta_{j2} \\
-\Delta\theta_{j3} & 1 & 0 \\
\Delta\theta_{j2} & 0 & 1
\end{bmatrix}
\]  

(12)

Where \(\Delta\theta_i\) are rotation increment of the end nodes.

The ESDM updated is given by Eq.13

\[
[p]_{i_{\text{end}}} = [p]_{i_{\text{end}}} [e]_{i_{\text{end}}}
\]

\[
[p]_{j_{\text{end}}} = [p]_{j_{\text{end}}} [e]_{j_{\text{end}}}
\]  

(13)

The element orientation matrix \([r]\) is taken the average of ESDM, expressed as follows:

\[
[r]_{t+\Delta t} = \frac{1}{2} ([p]_{i_{\text{end}}} [e]_{i_{\text{end}}} + [p]_{j_{\text{end}}} [e]_{j_{\text{end}}})
\]  

(14)
2.1.3 Beam section direction at first iteration. The beam element established in this paper is a three-node (i, j, k) element as shown in Figure 2. The nodes i, j are the two end nodes of the beam element, and the k is the auxiliary node, which is used to ascertain the beam element section orientation.

![Figure 2 Schematic Diagram of Beam Element](image)

The direction cosine of the angle between ECS axis x and the GCS axis X, Y, Z can be calculated with the Eq.6 and Eq.7. Set the midpoint of element is \( c (x_c, y_c, z_c) \), and the direction cosine of the angle between the z axis and the GCS X, Y, Z is:

\[
    r_{xz} = \frac{(x_k-x_c)}{\sqrt{(x_c-x_k)^2 + (y_c-y_k)^2 + (z_c-z_k)^2}} \tag{15}
\]

\[
    r_{yz} = \frac{(y_k-y_c)}{\sqrt{(x_c-x_k)^2 + (y_c-y_k)^2 + (z_c-z_k)^2}} \tag{16}
\]

\[
    r_{zz} = \frac{(z_k-z_c)}{\sqrt{(x_c-x_k)^2 + (y_c-y_k)^2 + (z_c-z_k)^2}} \tag{17}
\]

The ECS axis y can be determined according to the right-hand screw rule.

2.2 Classic example test

Based on the UPFS method provided by the ANSYS platform, the beam element considering geometric nonlinearity is developed, and the classic example is calculated and compared to verify the algorithm logic of the program.

2.2.1 Hexagonal star dome structure balanced path. The equilibrium path research of hexagonal star dome structure is also a classical example of geometrical nonlinear analysis. The structural diagram is shown in Figure 3. In this paper, the section orientation is slightly different from that in the literature [8]. Because of the same displacement shape functions, the element BEAM4 in the ANSYS element library was used to compare the difference with the beam developed based on UPFs. The results are shown in Figure 4~Figure 7. The members were divided into 1 element in Figure 4, Figure 6, and 4 elements in Figure 5, Figure 7. (Table 1). Figure 4 and Figure 5 are load- vertical displacement curves of point 1. Figure 6 and Figure 7 are the load - horizontal displacement UX curves of point 2 (Table 1).

It can be seen from the comparison that the load displacement curves of the two types of elements are basically coincident, indicating the rationality of the UPFs program.

![Figure 3 Hexagonal star dome structure diagram](image)
Table 1 Load - displacement With Different Element Number

| Number of element divisions | Load - vertical displacement of point 1 | load – horizontal displacement UX of point 2 |
|-----------------------------|----------------------------------------|---------------------------------------------|
| 1                           | Figure 4                               | Figure 6                                    |
| 4                           | Figure 5                               | Figure 7                                    |

![Figure 4](image) Load- vertical displacement curve of point 1 (1 Element)

![Figure 5](image) Load- vertical displacement curve of point 1 (4 Elements)

![Figure 6](image) Load – horizontal displacement UX of point 2 (1 Element)

![Figure 7](image) Load – horizontal displacement UX of point 2 (4 Elements)

2.2.2 Geometric nonlinear analysis of a 12 member hexagonal space frame. The last example of the geometrical nonlinearity analysis is the 12 member hexagon spatial rigid frame (Figure 8), and the BEAM4 model result is also compared. The results are shown in Figure 9 (1 element) and Figure 10 (4 elements). Both of the two group load- displacement curves are in good agreement.

![Figure 8](image) 12 member hexagon spatial rigid frame
3. Study on the stiffness matrix of beam element considering bolt slip.

3.1 Load-displacement curve of bolt slip

Bolts are widely used in transmission towers as shear connectors shown as Figure 11. Under external loads, relative slip maybe happen between connectors, because of the gap between bolts and bolt holes, shown as Figure 12. Therefore, the joint is no longer a complete rigid connection, but exhibits the characteristics of semi-rigid.

Figure 11 typical connection of main member and diagonal member

(a) Before slippage
(b) After slippage

Figure 12 Joint slippage in bolted joint

The ordinary bolt slips displacement under shear is mainly composed of two parts: clearance slip $\Delta 1$ depending on the gap between screw and bolt holes, deformation slip $\Delta 2$ including screw and bolt holes deformation. Many calculation models are put forward, according to the characteristics of bolt slips, such as ideal connection slip model, linear model and exponential model et al.

W.Q. Jiang [10] proposed a parameterized slip model for shear connection and verify the validity of the model: $\Delta 1$ according to the parameterized equation of the material nonlinear stress-strain relation expressed as Eq.18, $\Delta 2$ referring to the function expression of the load-deformation process described by Eq. 19.

$$\Delta_1 = \delta_y \frac{P}{P_y} + \beta \delta_y \left( \frac{P}{P_y} \right)^N$$

(18)

Where: $\beta$ and $N$ are shape parameters, determined by test. $P_y$ is yield load. $\delta_y$ is the deformation
corresponding to $P_y$.

$$\Delta_2 = \delta_0 \left( \frac{p}{p_s} \right)^n \left[ 1 + \left( \frac{p}{p_s} \right)^{\frac{1}{n}} \right]^m$$  \hspace{1cm} (19)

Where: $\delta_0$ is the gap between the screw and the bolt hole. $P_s$ is slip load. $n, m$ are shape parameters.

$$\Delta = \Delta_1 + \Delta_2$$  \hspace{1cm} (20)

The relevant parameters in the Eq. 18 and Eq. 19 can be checked in Table 2 and Table 3.

### Table 2 Parameters for Single-leg Bolted Joint

| Bolts Num | $\delta_0$ | $\delta_r$ | $p_s$ | $p_y$ | $\beta$ | $N$ | $m$ | $n$ |
|-----------|------------|------------|-------|-------|--------|-----|-----|-----|
| 1         | 1.3        | 2.74       | 9.29  | 65.03 |        |     |     |     |
| 2         | 1.3        | 2.40       | 20.14 | 91.51 | 0.02   | 10  | 20  | 20  |
| 3         | 1.3        | 2.40       | 29.28 | 152.9 |        |     |     |     |
| 4         | 1.3        | 2.40       | 46.95 | 168.2 |        |     |     |     |

### Table 3 Parameters for Lap-splice Bolted Joint

| Bolts Num | $\delta_0$ | $\delta_r$ | $p_s$ | $p_y$ | $\beta$ | $N$ | $m$ | $n$ |
|-----------|------------|------------|-------|-------|--------|-----|-----|-----|
| 2x2       | 1.3        | 2.26       | 43.30 | 216.40|        |     |     |     |
| 2x3       | 1.3        | 2.26       | 64.95 | 324.60| 0.02   | 10  | 5   | 5   |
| 2x4       | 1.3        | 2.26       | 86.60 | 432.80|        |     |     |     |
| 2x5       | 1.3        | 2.26       | 108.25| 514.00|        |     |     |     |

3.2 **Stiffness matrix of beam element considering bolt slip**

Based on the research of ordinary beam element, the spring-beam model, as shown in Figure 13, is used to consider the joint semi-rigid effect. The overall stiffness matrix of the spring-beam model is used as the stiffness matrix of a special beam element for the analysis of the transmission tower through UPFs.

**Figure 13 Spring-beam Model**

The bending moment of spring can be expressed as follows:

$$\{ M \} = [K]\{ \theta \} \quad \text{ (21)}$$

$$\{ M \} = \begin{bmatrix} M_{ya} & M_{za} & M_{yat} & M_{zat} & M_{yb} & M_{zb} & M_{ybi} & M_{zbi} \end{bmatrix}^T$$  \hspace{1cm} (22)

$$\{ \theta \} = \begin{bmatrix} \theta_{ya} & \theta_{za} & \theta_{yat} & \theta_{zat} & \theta_{yb} & \theta_{zb} & \theta_{ybi} & \theta_{zbi} \end{bmatrix}$$  \hspace{1cm} (23)

Where, $\theta_{yat}, \theta_{zat}, \theta_{ybi}, \theta_{zbi}$ is the additional freedom degree of internal node. $M_{yat}, M_{zat}, M_{ybi}, M_{zbi}$ is corresponding moment. $R_{ki}$ is the rotational stiffness of the spring. Beam element has 6 degrees of freedom at each node. Considering four internal degrees of freedom of the spring, the degree of freedom of the model is changed to 16. The overall stiffness matrix of the model can be deducted as follows:
\[
\begin{bmatrix}
R_{xy} & 0 & -R_{xy} & 0 & 0 & 0 & 0 & 0 \\
0 & R_{xy} & 0 & -R_{xy} & 0 & 0 & 0 & 0 \\
-R_{xy} & 0 & R_{xy} & 0 & 0 & 0 & 0 & 0 \\
0 & -R_{xy} & 0 & R_{xy} & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & R_{xy} & 0 & -R_{xy} & 0 \\
0 & 0 & 0 & 0 & -R_{xy} & 0 & R_{xy} & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & -R_{xy} \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & R_{xy}
\end{bmatrix}
\]

\((24)\)

\[
\begin{bmatrix}
\{f\}_a = \left[K\right]_a \{u\}_a \\
\{f^{12}\}_a = \left[K'_j\right]^{12:12} \left[K'_j\right]^{12:4} \left[U^{12}\right] \\
\{f^4\}_a = \left[K'_j\right]^{4:12} \left[K'_j\right]^{4:4} \left[U^4\right] \\
\{f\}_a = \begin{bmatrix}
F_{xa} & F_{yb} & F_{ya} & M_{xa} & M_{yb} & M_{ya} & F_{ha} & F_{jab} & F_{jab} & M_{hab} & M_{jab} & M_{jba} & M_{jha} & M_{jhb}
\end{bmatrix}
\end{bmatrix}
\]

\((25)\)

\[
\begin{bmatrix}
\{u\}_a = \begin{bmatrix}
u_a & w_a & \theta_{ya} & \theta_{xa} & u_b & v_b & w_b & \theta_{yb} & \theta_{xb} & \theta_{yab} & \theta_{xab} & \theta_{yax} & \theta_{xay} & \theta_{ybx} & \theta_{xby}
\end{bmatrix}
\end{bmatrix}
\]

\((26)\)

By condensing the internal degrees of freedom, the equation (25) can be expressed as:

\[
\{U^4\} = \left(K'_j\right)^{4:4} \left(K'_j\right)^{4:4} \left(K'_j\right)^{4:4} \left(K'_j\right)^{4:4} \left(U^{12}\right)
\]

\((28)\)

Substitute Eq. (25) into Eq. (28):

\[
\{f^{12}\}_a = \left[K'_j\right]^{12:12} \{U^{12}\}
\]

\((29)\)

where:

\[
\left[K'_j\right]^{12:12} = \left(K'_j\right)^{12:12} - \left(K'_j\right)^{12:4} \left(K'_j\right)^{4:4} \left(K'_j\right)^{4:12}
\]

\((30)\)

\[
\{f^{12}\}_a = \{f^{12}\} + \left(K'_j\right)^{12:4} \left(K'_j\right)^{4:12} \left(f^4\right)
\]

\((31)\)

Substituting Eq. (30), (31) into Eq. (29), the equilibrium equation considering joint effect is obtained.

3.3 Semi-rigid finite element model validation

In order to verify the FEA reliability, the semi-rigid model of the space tower leg model test (Figure 14) in the literature [11] was built to compare differences in secondary stress ratio. The secondary stress ratio \(\alpha\) (as shown in Eqs. 32, 33) is used to reflect the magnitude of the maximum stresses and the axial stresses of main members.

\[
\sigma_s = \sigma_{max} - \sigma_{axial}
\]

\((32)\)

\[
\alpha = \frac{\sigma_s}{\sigma_{axial}}
\]

\((33)\)

Figure 14 Space Tower Leg Structure Model Test
The results of test specimen under different finite element models are shown in Table 4. It can be seen that the secondary stress ratios $\alpha$ results have little difference, which proved that the semi-rigid model is of good reliability, and, the multi-scale model considering the stress distribution state of the circular pipe section in detail is closer to the test result.

| Calculation model | Semi-rigid model | multi-scale model [11] | Test [11] |
|-------------------|-----------------|------------------------|----------|
| $\alpha$ /%       | 52              | 55                     | 58       |

4. Finite element analysis of semi-rigid model

4.1 Analysis of secondary stresses of whole tower

A 500/220kV multi-high-voltage tower with 84.8 m height (shown as Figure 15, the angle between main member and diagonal member was 29° and the slenderness ratio of main member was 36.) commonly employed in Chinese urban transmission grids is chosen as a research object to study the secondary stresses distribution rule of the transmission tubular tower.

In the semi-rigid FEM, the main members are simulated with Beam188, diagonal member with the special beam element developed based on UPFs. As the height of the prototype tower is larger and the overall structure is "soft", the geometric nonlinear analysis is adopted to consider the influence of large deformation. In FEA, the load applied on the tower is the 60° wind condition.

Figure 16 shows the secondary stress magnitude and distribution rule of the main member at different height of the overall tower, shown as follows:

1) Internodes 110-230, the secondary stresses ratio $\alpha$ were largest, just because of small axial stresses rather than large secondary stresses, so, the total stress of the cross section is relatively small.

2) Internodes 930-1110, 1110-1160 located in the cross arm height, and 1490-1560, 1560-1570 at the diaphragm height, the secondary stress ratio $\alpha$ is up to 21%, mainly because the main member slenderness ratio $\lambda$ is only 14 and 16 respectively.

3) The secondary stress ratio $\alpha$ of main members in the tower leg (internode 2400-2460) is as high as 29%. Bresle [12] indicated secondary stresses less than 20% of the axial stress could be neglected, consistent with the provision in Chinese standard code [13]. Consequently, study on the secondary stresses of main member in tower leg is of great significance and urgency.

4.2 Influence factors and rules of secondary stress of main member

The auxiliary members are usually used to reduce the calculation length of the main members, and to make the design more reasonable [1]. The setting of auxiliary members changes the deformation mode of the main member, which raise the appearance of the inflection point in the main member deformation (Figure 17). In Figure 17, $\delta_e$ is horizontal displacement of the diaphragm, and $\delta_x$ is horizontal displacement of the inflection point, and $\theta$ is the tower leg angle. It can be seen from Figure 17 that when the tower leg divided into a lattice, the bending moment magnitude of the main member at the foot is mainly dependent on horizontal displacement of the diaphragm $\delta_e$. So, the line stiffness (slenderness ratio) of the main member has a significant influence on the bending moment of the foot, but not sensitive to the change of the tower leg angle. The secondary stresses at the member end essentially depends on the slenderness ratio of members, which has also been discovered in the truss structures, and related formulas was put forward [14]. As for the tower leg divided into two lattices, tower leg deformation has inflection point, bending moment in the tower foot depends on the horizontal displacement $\delta_e$, it is inferred that the smaller the tower leg angle, the more prone to have inflection point.
Maintaining the whole tower leg members slenderness ratio ($\lambda = 36$) unchanged, Secondary stresses of tower with different kinds tower leg angle $\theta$ are compared. As shown in Figure 18, when the tower leg divided into two lattices, the secondary stresses decline markedly as the angle $\theta$ increasing, as for a lattice, the secondary stress changes little, the maximum difference of 3%.

With the same angle ($\theta = 33^\circ$) unchanged, by adjusting the section of leg main members, the tower models with different kinds slenderness ratio of main members is established. The influence of slender ration on the secondary stresses of main leg members are analyzed. As shown in Figure 19, when tower leg divide into two lattices the secondary stresses gradually decreases along with the slenderness ratio increasing but change little. While, a lattice, the secondary stresses more obviously reduce as the slenderness ratio increasing.

In practice, because of restrictions of the existing steel pipe specifications, it does not generally appears the condition that the angle changes, while slenderness maintained constant in the transmission tower design. Therefore, the comprehensive effect of the angle and slender ratio on secondary stresses is investigated.
It can be seen from Table 5 and Figure 20 as follows:

1) Under the same utilization rate of the cross section, secondary stresses of the leg main member of two lattices are greater than that of a lattice.

2) Maintaining the utilization rate of the cross section constant, when the tower leg divided into two lattices, the secondary stresses variation law is consistent with the influence of the tower leg angle $\theta$, which illustrates that the secondary stresses is governed by the tower leg angle, while having little contact with slender ration of leg main member.

3) Maintaining the utilization rate of the cross section constant, when the tower leg of a lattice, the secondary stress variation law is consistent with the influence of the slender ratio of leg main member, it means that the secondary stress of main leg is depend on the slender ratio of leg main member.

| $\theta$ | 15° | 18° | 20° | 24° | 29° | 33° | 37° |
|---------|-----|-----|-----|-----|-----|-----|-----|
| A       |
| Lattice | $\lambda$ | 50  | 42  | 38  | 37  | 30  | 26  | 23  |
| $\alpha$ | 16  | 17  | 19  | 20  | 20  | 20  | 21  |
| Two Lattices |
| $\lambda$ | 27  | 23  | 20  | 20  | 17  | 14  | 13  |
| $\alpha$ | 39  | 35  | 33  | 31  | 29  | 25  | 22  |

The calculation of the main member is controlled by the overall stability, of which slenderness ratio is generally controlled between 35 and 55. If the slenderness ratio of the main member is too large, the auxiliary member is usually installed to reduce the calculation length. From the above analysis, a conclusion can be drawn for design: for the narrow base tower, the root opening of the tower is smaller, the tower leg main member is suggested to be divided into a lattice by lowering the height of diaphragm surface (Figure 21). For the ordinary tower, the root opening is often larger. If program used in the narrow base tower is still adopted, the structure will become unreasonable because of oversize joint plate, and not easy to be construct. Therefore, it is suggested to control the main slope properly for the ordinary tower.
5. Conclusions
The semi-rigid models of transmission tubular towers are built by directly calling the special beam elements developed by UPFs taking bolt slip effect into account. Secondary stresses of the whole tower were calculated and the influence factors of secondary stresses of the tower leg main members are analyzed, conclusions could be drawn as followings:

1. The larger secondary stresses of main members mainly located at the height of the cross arm and tower diaphragm.

2. The secondary stresses of main leg members of two lattices are governed by the tower leg angle and the smaller angle, the greater secondary stresses.

3. When the main leg member of a lattice, the secondary stresses depend on the slender ratio of main leg member, and obviously reduces as the slenderness ratio increasing.

4. Under the same utilization rate of the cross section, the secondary stress of main member of tower leg of two lattices is larger than that of tower leg of a lattice.

5. The narrow base tower has a smaller opening. When the angle of the main inclined material of the tower leg is small, it is suggested the tower leg main member divided into a lattice by lowering the height of the diaphragm surface. As for ordinary towers, it is suggested that the angle between main member and diagonal member be reasonably controlled.

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