Influence of Double-Limb Double-Plate Connection on Stable Bearing Capacity of Quadrilateral Transmission Tower

Tengfei Zhao 1, Hong Yan 2,3,*, Panpan He 1, Lei Zhang 3, Zhiwen Lan 2 and Mojia Huang 2

1 College of City Construction, Jiangxi Normal University, Nanchang 330022, China; 13870937708@163.com (T.Z.); panpanhe99@163.com (P.H.)
2 Institute of Engineering Mechanics, Nanchang University, Nanchang 330031, China; zwlan@ncu.edu.cn (Z.L.); mojiahuang@hotmail.com (M.H.)
3 College of Architectural Engineering and Planning, Jiujiang University, Jiujiang 332005, China; 6130085@jjju.edu.cn
* Correspondence: yhtj@jjju.edu.cn

Abstract: Transmission tower connection joint is an important connection component of the tower leg member and diagonal member. Its axial stiffness directly affects the stable bearing capacity of a transmission tower. The axial stiffness of the joint is mainly related to the connection form of joint. This paper takes the double-limb double-plate connection joint as the research object. Through the comparative study with the single-limb single-plate connection joint, the influence law of single-limb single-plate and double-limb double-plate joint on stable bearing capacity of quadrilateral transmission tower is studied from three aspects of model test, theoretical analysis and numerical simulation. Through the scale model test, it is found that the elastic stiffness of the double-limb double-plate joint is 3.12 times that of the single-limb single-plate joint, which can increase the bearing capacity of the joint by 26.1%. Through the energy method, the theoretical calculation expression of the stable bearing capacity of the quadrilateral tower considering the influence of the axial stiffness of the joint is derived. Compared with the effect of the single-limb single-plate connection joint, the double-limb double-plate joint can improve the stable bearing capacity of the quadrilateral tower by 15.6%. Considering the influence of geometric nonlinearity of tower and connecting joint, it is found that the double-limb double-plate connecting joint can improve the nonlinear stability bearing capacity of a transmission tower by 14.9%. The results show that the double-limb double-plate connection joint can not only improve the bearing capacity of the joint, but also greatly improve the stable bearing capacity of the tower. The research results can provide reference for the engineering application and design of double-limb double-plate connection joints.

Keywords: single-limb single-plate; double-limb double-plate; elastic stiffness; model test; geometric nonlinearity; nonlinear stable bearing capacity

1. Introduction

With the increasing energy demand worldwide, the development of power technology plays an important driving role [1,2]. As an important carrier of power transmission, the transmission line system, and the transmission tower is the skeleton of the transmission line system, the safety and stability of its structure will directly affect the normal operation of the power system [3–6]. In the traditional transmission tower structure design, the space truss method with hinged joints is often used to calculate the internal force of the tower, without considering the impact of joint stiffness on the transmission tower structure, which may have potential safety hazards and material waste.

The form of tower connection joint will affect the stiffness of the joint. At present, the research on the mechanical characteristics of transmission tower joints by scholars at home and abroad mainly focuses on the test and the theoretical calculation of traditional and innovative joints. Ma et al. [7–9] studied the mechanical properties of the new beam
column connection joint of long-span steel hyperbolic tower under out of plane bending moment and shear through test and numerical simulation, and further studied the role of the joint in the stability of the cooling tower based on the test results. Martins et al. [10] obtained the mechanical properties of I-shaped steel joints under cyclic load through tests. Zhang et al. [11] studied the seismic performance of prefabricated steel frame structures with I-shaped steel all bolt connection. Li et al. [12] considered the initial defects of the new circular pipe double-layer flange connection joint, and gave the actual design method through test and finite element analysis. Song et al. [13] carried out experiments and numerical analysis on the mechanical properties of K-type and KK type concrete-filled circular steel tubular joints, and proposed the design method of K and KK joints of concrete-filled spherical steel tubular joints. Tian et al. [14] studied the failure mode, the load displacement curve, the load strain curve, deformation development and ultimate strength of X-shaped pipe joints of new steel tube transmission tower through test and numerical simulation. Li et al. [15,16] compared and analyzed the ultimate bearing capacity of plane round steel tube K-type and multi plane DK type joints through test and numerical simulation. Liu et al. [17] proposed an effective method to consider the chord connection area in the finite element model of tubular transmission tower in linear elastic stage through theoretical analysis. He et al. [18] studied the influence of the performance of beam column joints in box section on the continuous collapse resistance of the structure through test and numerical simulation. Ma et al. [19] conducted experimental and numerical research on the performance of the new single double transformation connection node, and further improved the existing theoretical model of gusset plate design. Abdelrahman et al. [20] proposed an improved node slip model based on Numerical Simulation for the second-order direct analysis of transmission towers. Gan et al. [21] proposed a simplified joint slip model for bolted joints, and verified the theoretical model through finite element numerical simulation. An et al. [22] proposed a general node element to designate the node slip effect and introduced it into the structural analysis of lattice transmission tower. Balagopal et al. [23] proposed a simplified bolted connection model considering axial stiffness and rotational stiffness, and deduced the rotational stiffness formula of slip due to eccentric load transfer in angle steel section bolted connection. Szafran et al. [24] presented a reliability evaluation method of steel lattice communication tower based on the reliability of tensioning node through test and numerical simulation. Souza et al. [25] evaluated the reliability of the transmission tower structure considering the mechanical model uncertainty caused by the connection details. Yang et al. [26] improved the calculation expression of K-joint displacement of transmission tower, and verified the theoretical calculation through numerical simulation and full-scale test. The mechanical analysis and comparison of several commonly used auxiliary materials are manufactured. The research model is plane tower leg, which does not take into account the impact of the actual spatial structure on the mechanical performance of tower leg. Ahmed et al. [27] studied the mechanical properties of a transmission tower under load by using finite element method considering the experimental sliding behavior of bolted joints of the transmission tower. Research on the connection joints of transmission towers at home and abroad mostly focuses on the test and numerical simulation of circular steel pipe joints, angle steel bolt joints, I-steel joints and box joints. In this paper, a new type of double-limb double-plate connection joint [28] connecting circular steel pipe and angle steel is studied. The mechanical characteristics of the new type of double-limb double-plate joint and its influence on the stable bearing capacity of quadrilateral tower are studied through test, numerical simulation and theoretical analysis. The research results can provide reference for the engineering application and design of double-limb double-plate connection joint.

2. Bearing Capacity Test of Double-Limb Double-Plate Joint

2.1. Test Purpose

Double-limb double-plate joint is a type of steel pipe angle steel joint [28]. The tensile bearing capacity test of double-limb double-plate joint is designed and carried out in order
to obtain the mechanical characteristics of the new double-limb double-plate joint and the actual tensile bearing capacity of the joint, so as to provide a basis for theoretical research, engineering design and engineering application.

2.2. Test Model

In order to understand the advantages and disadvantages of the new double-limb double-plate connection joint, the scale models of the traditional single-limb single-plate joint and the new double-limb double-plate joint are selected for the test. The connection models of the traditional single-limb single-plate joint and the new double-limb double-plate joint are shown in the Figure 1.

For convenience, the single-limb single-plate joint is hereinafter referred to as SLSPJ, and the double-limb double-plate joint is hereinafter referred to as DLDPJ.

According to the connection form of the joint [28], the processing drawings of the two joints are designed according to the scale of 1:5, as shown in Figures 2 and 3.

The test scale models of SLSPJ and DLDPJ processed according to the construction drawing are shown in Figure 4. The joint specimen is made of Q355 steel, and bolts shall be Grade 8.8 M8 bolt, the standard torque is 25 N·m, and the torque of 20 N·m (80% of the standard torque value) is used for the joint connections.
Figure 2. (a) Elevation of 1:5 scale model of SLSPJ; (b) front view of connecting angle steel of 1:5 scale model of SLSPJ; (c) section of 1:5 scale model of SLSPJ.
Figure 3. (a) Elevation of 1:5 scale model of DLDPJ; (b) connection diagram of long and short angle steel of 1:5 scale model of DLDPJ; (c) M-M side view of long and short angle steel of 1:5 scale model of DLDPJ; (d) 1-1, 2-2 and 3-3 sections and cushion blocks of 1:5 scale model of DLDPJ.
2.3. Loading System and Test Loading Device

The SLSPJ and DLDPJ specimens are made of Q355 steel, according to the joints’ construction drawing (Figures 2–4), the specification of connecting angle steel is L50 × 5, and the specification of loading bar is Φ30. The yield strength of steel is 345 MPa, through analysis and calculation, the yield loads of SLSPJ and DLDPJ specimens are 164 kN and 244 kN, respectively. According to the yield load of components, it is proposed to determine the load classification of SLSPJ and DLDPJ, as shown in Table 1.

Table 1. Load classifications of the specimens of SLSPJ and DLDPJ.

| Loading Stage | 1   | 2   | 3          | 4          |
|---------------|-----|-----|------------|------------|
| Load size (kN) | SLSPJ | 60  | 100        | destruction |
|               | DLDPJ | 60  | 105        | 150        | destruction |

The SLSPJ test is a monotonic loading static test, which is carried out in the way of graded loading, which is divided into three stages:

- The first six stages of loading in the test shall be loaded from zero to 60 kN, and each stage of loading shall be increased by 10 kN. After each stage of loading is maintained for 30 s, the corresponding load strain and displacement values shall be recorded.
- After that, it is loaded to 100 kN in two stages, and each stage is increased by 20 kN. After each stage of loading is maintained for 30 s, the corresponding load strain and displacement values shall be recorded.
- Finally, load until the test specimen is damaged, after each stage is loaded with 10 kN and maintained for 30 s, collect data until the specimen is damaged.

The graded loading test of DLDPJ is divided into four stages:
• The first six stages of loading in the test shall be loaded from zero to 60 kN, and each stage of loading shall be increased by 10 kN. After each stage of loading is maintained for 30 s, the corresponding load strain and displacement values shall be recorded.
• After that, it is loaded to 105 kN in three stages, and each stage is increased by 15 kN. After each stage of loading is maintained for 30 s, the corresponding load strain and displacement values shall be recorded.
• Load to the design ultimate load of the test piece, and load it to 150 kN in three levels, with an increase of 15 kN for each level.
• Finally, load until the specimen is damaged, load 10 kN at each stage and collect data after each stage is loaded with 10 kN and maintained for 30 s until the specimen is damaged.

The 600 kN microcomputer controlled electro-hydraulic servo universal testing machine is used for test loading, as shown in Figure 5, the test loading process can be controlled automatically, and the force displacement and force time test curves can be drawn automatically, the strain distribution in the specimen can be obtained by pasting a strain gauge on the surface of the specimen, as shown in Figure 6. According to the test requirements, the test loading device for SLSPJ and DLDPJ are built, as shown in Figure 6, the bottom of the test device clamps the gusset plate of the joint test piece through the fixture, and the top clamps the loading bar of the joint test piece through the fixture.

Figure 5. The 600 kN microcomputer controlled electro-hydraulic servo universal testing machine.
2.4. Test Results

The failure process of SLSPJ and DLDPJ specimens is basically the same. The failure process is as follows: in the initial stage of loading, the deformation of each member of the specimen is in the elastic stage, and the load and displacement show a linear change trend; when the load continues to increase, the connecting plate and angle steel enter the plastic deformation stage. At this time, the joint deforms rapidly, the load continues to increase, the deformation of the gusset plate and angle steel intensifies, and finally reaches the ultimate bearing capacity of the specimen.

With the increase of load, the stress of the angle steel member of the single leg single plate joint specimen increases sharply, and the failure occurs first. The failure form is mainly the tensile failure at the section of the first row of bolts on the angle steel member, as shown in Figure 7. The damage of DLDPJ is mainly the damage of long and short connecting angle steel, as shown in Figure 8.

The load displacement curves of SLSPJ and DLDPJ specimens are given in Figure 9. The load displacement curve shows that the bearing capacity of SLSPJ specimen is 131.4 kN and that of DLDPJ specimen is 165.7 kN. The test results show that the bearing capacity of DLDPJ can be increased by 26.1%. 

Figure 6. (a) Loading device for test piece of SLSPJ; (b) loading device for test piece of DLDPJ.
Figure 7. (a) Connecting angle steel after unloading of SLSPJ; (b) Connecting bolts after unloading of SLSPJ.

Figure 8. (a) Failure of DLDPJ after unloading; (b) connecting short angle steel after unloading of DLDPJ; (c) connecting long angle steel after unloading of DLDPJ.

The load displacement curves of SLSPJ and DLDPJ specimens are given in Figure 9. The load displacement curve shows that the bearing capacity of SLSPJ specimen is 131.4 kN and that of DLDPJ specimen is 165.7 kN. The test results show that the bearing capacity of DLDPJ can be increased by 26.1%.
3. Calculation Method of Stable Bearing Capacity of Tower

3.1. Stable Bearing Capacity of Tower without Considering the Influence of Joint Axis Stiffness

In the engineering design, the transmission tower structure belongs to the truss structure of batten combination, and the influence of joint stiffness is generally not considered. The connection joint between the column limb and batten is generally simplified as hinge joint. When the structure is unstable, each pole in the transmission tower only causes additional axial force. The calculation of the transmission line tower is mostly simplified as a cantilever structure with fixed bottom and upper free part. The calculation model is shown in Figure 10.

Figure 9. Load-displacement curve of joint specimen.

![Load-displacement curve of joint specimen](image_url)

Figure 10. Theoretical calculation model of cantilever tower.
In the calculation model, the leg member of transmission tower is round pipe steel, and the cross-sectional area is \( A_1 \); Angle steel is adopted for the diagonal member, with a cross-sectional area of \( A_2 \), and the included angle between the upper and lower diagonal members and the horizontal direction is \( \theta \). It is assumed that the curve of transmission tower instability is sinusoidal, i.e.,

\[
y = a \sin \frac{\pi x}{2l}
\]

(1)

In Equation (1), \( l \) is the total height of the tower, and \( x \) is the distance from the top of the tower.

The bending moment at any point on the axis of the tower is,

\[
M = F_p y = F_p a \sin \frac{\pi x}{2l}
\]

(2)

Shear force is,

\[
F_Q = \frac{dM}{dx} = F_p a \pi \frac{\sin \frac{\pi x}{2l}}{2l}
\]

(3)

The axial force of tower leg member \( F_N^1 \) and diagonal member \( F_N^2 \) can be obtained by approximate calculation of truss. As shown in Equations (4) and (5).

\[
F_N^1 = \pm \frac{M}{2b} = \pm F_p a \frac{\sin \frac{\pi x}{2l}}{2b}
\]

(4)

\[
F_N^2 = \pm F_Q \frac{4}{4 \cos \theta} = \pm F_p a \frac{\pi \cos \frac{\pi x}{2l}}{8l \cos \theta}
\]

(5)

In Equations (4) and (5), \( b \) is the root opening of the tower; \( \theta \) is the included angle between the diagonal member and the horizontal axis, as shown in Figure 10.

The strain energy of the tower is,

\[
U = \sum \frac{F_N^2 \varepsilon}{2EA}
\]

(6)

In Equation (6), \( \varepsilon \) is the length of each member and \( E \) is the elastic modulus of the member material.

After substituting Equations (4) and (5) into Equation (6), the following Equation (7) is obtained.

\[
U = \frac{1}{2EA} \left( \sum_{1}^{4n} \frac{(F_p a \sin \frac{\pi x}{2l})^2 d}{A_1} + \sum_{1}^{4n} \frac{(F_p a \pi \cos \frac{\pi x}{2l} \cos \frac{\pi x}{2l})^2 b \cos \theta}{A_2} \right)
\]

(7)

In Equation (7), \( A_1 \) is the area of the leg member; \( A_2 \) is the area of diagonal member; \( d \) is the height between tower joints; \( b \) is the root opening of the tower; \( \theta \) is the angle between the diagonal member and the x-axis; \( n \) is the number of joints of the tower, the total number of diagonal members is the sum of \( 4n \) diagonal members, and the total number of leg members is the sum of \( 4n \) leg members.

Generally, the number of joints of the tower structure is large, which can be taken in actual calculation,

\[
d = \Delta x \approx dx
\]

(8)

Use the change of Equation (9),

\[
\sum_{1}^{4n} \sin \frac{\pi x}{2l}^2 d \approx 4 \int_{0}^{l} (\sin \frac{\pi x}{2l})^2 dx = 2l
\]

\[
\sum_{1}^{4n} \cos \frac{\pi x}{2l}^2 d \approx 4 \int_{0}^{l} (\cos \frac{\pi x}{2l})^2 dx = 2l
\]

(9)
And use the relationship of Equation (10),

$$d = 2b \tan(\theta) \tag{10}$$

The strain energy of the tower can be written as following Equation (11),

$$U = \frac{F^2 a^2}{4E} \left[ \frac{1}{A_1 b^2} + \frac{\pi^2}{32l^2} \tan(\theta) \cdot \frac{1}{A_2 \cos^3 \theta} \right] \tag{11}$$

The potential energy of external load is,

$$U_P = -F_P \int_0^l \frac{1}{2} (y')^2 dx = -F_P \frac{a^2 \pi^2}{16l} \tag{12}$$

According to the energy method [29,30], stationary condition of potential energy is as shown in Equation (13).

$$\frac{dE_P}{da} = 0 \tag{13}$$

So the expression of critical load can be obtained. As shown in Equation (14).

$$F_{Pcr} = \frac{\pi^2 EI}{l^2} \frac{1}{4 + \frac{\pi^2}{8} \left( \frac{b}{2} \right)^2 \frac{A_1}{A_2 \sin \theta \cos^3 \theta}} \tag{14}$$

In Equation (14), $I$ is the moment of inertia of the tower section to form mandrel. As shown in Figure 10.

$$I = 4A_1 \left( \frac{b}{2} \right)^2 = A_1 b^2 \tag{15}$$

3.2. Stable Bearing Capacity of Tower Considering the Influence of Joint Axis Stiffness

The tower connection joint can be equivalent to a spring element with elastic modulus $E$ and cross-sectional area $A_j$. The coefficient of the length of the joint in the general name of the members is $k (0 \sim 1.0)$. According to the energy method, the energy of a single joint under axial force can be expressed as following. Simplified calculation model of axial stiffness of double-limb double-plate is shown in Figure 11.

![Figure 11. Simplified calculation model of axial stiffness of DLDPJ.](image-url)
The strain energy of DLDPJ is shown in Equation (16).

$$U = \sum \frac{F_N k b \cos \theta}{2 E A_j}$$  

(16)

In Equation (16), $k$ is the ratio of the length of the connecting joint to the total length of the member, referred to as the joint length coefficient; $A_j$ is the cross-sectional area of the equivalent spring joint.

After substituting Equations (4) and (5) into Equation (16), the following Equation (17) is obtained.

$$U = \frac{1}{2E} \left( 4n \int_0^l \left( \cos \frac{\pi x}{2l} \right)^2 dx \approx 8 \int_0^l \left( \cos \frac{\pi x}{2l} \right)^2 dx = 4l \right)$$  

(17)

The transformation formula of joint energy part is shown in Equation (18).

$$\sum_{i=1}^{8n} \left( \cos \frac{\pi x}{2l} \right)^2 d \approx 8 \int_0^l \left( \cos \frac{\pi x}{2l} \right)^2 dx = 4l$$  

(18)

By transforming Equations (9), (10) and (18). Equation (19) is obtained as following.

$$U = \frac{F_P^2 a l^2}{4E} \left[ \frac{1}{A_1 b^2} + \frac{\pi^2}{32 l^2 \tan(\theta)} \cdot \frac{1 - k}{A_2 \cos^2 \theta} + \frac{\pi^2}{32 l^2 \tan(\theta)} \cdot \frac{k}{A_j \cos^3 \theta} \right]$$  

(19)

From the Equation (13), the expression of the critical load can be obtained as following.

$$F_{Pcr} = \frac{\pi^2 E I}{l^2} \cdot \frac{1}{4 + \frac{\pi^2}{8} \left( \frac{b}{l} \right)^2 + \frac{(1-k) A_1}{A_2 \sin \theta \cos^2 \theta} + \frac{\pi^2 k}{A_j \sin \theta \cos^3 \theta}}$$  

(20)

3.3. Influence Coefficient of Axial Stiffness of Joints

In order to obtain the influence law of the axial stiffness of the joint on the stable bearing capacity of the tower, take the section sizes as $A_1 = A$, $A_2 = A/5$, $A_j = \zeta A$, respectively, the tower root opening and the height of the joints are the same, and take the height of the joints as $h$, that is $b = h$, take the two-layer tower cells as the research object, that is $l = nh$, $n = 2$, take the length coefficient of the connecting joint is $k = 0.3$, combined with Equation (14) and Equation (20), The calculation expression of the stable bearing capacity of the tower with and without the axial stiffness of the joint is obtained. The ratio coefficient of the two is taken as $\chi$, and the expression obtained by calculation is shown in Equation (21).

$$\chi = \frac{\left(32\sqrt{2} + 5\pi^2\right) \zeta}{32\sqrt{2} \zeta + 3.5\pi^2 \zeta + 0.3\pi^2}$$  

(21)

In Equation (21), $\chi$ is the ratio coefficient of the tower bearing capacity considering and not considering the axial stiffness of the joint, and $\zeta$ is the axial stiffness variation coefficient of the equivalent spring connected joint.

With the increase of axial stiffness variation coefficient, the bearing capacity ratio coefficient $\chi$ increases first and then tends to be stable. The variation curve is shown in Figure 12. According to the curve change range, the proportion coefficient $\chi$ of bearing capacity changes significantly in the range of axial stiffness change coefficient 0-0.4, then the change of proportion coefficient $\chi$ tends to be gentle, and finally the value of $\chi$ is stable at about 1.16.
The relationship curve between the ratio coefficient and the axial stiffness variation is shown in Figure 12.

According to Equation (18), take $\zeta = 1/5$ to calculate the tower bearing capacity as the reference bearing capacity, and $\zeta = 1/5 \sim 3$ to calculate the stable bearing capacity of the tower, respectively, and take the ratio coefficient to the benchmark bearing capacity as $\chi_1$. The relationship curve between the ratio coefficient $\chi_1$ and the axial stiffness change coefficient $\zeta$ is obtained, as shown in Figure 13.

In order to obtain the influence law of the axial stiffness of the joint on the stable bearing capacity ratio coefficient, the bearing capacity ratio coefficient $\chi$ and the axial stiffness coefficient $\zeta$ are considered.

According to Section 3.2, the tower connection joint can be equivalent to a spring element with a cross-sectional area of $A_j$. According to the elastic deformation calculation formula of the joint, the calculation expression of the cross-sectional area of the equivalent spring joint $A_j$ can be obtained as following.

$$A_j = \frac{Fl}{E\Delta l}$$  \(22\)

According to Equation (22), we can acquire Equation (23).

$$\frac{A_{js}}{A_{jd}} = \frac{\kappa_s l_s}{\kappa_d l_d}$$  \(23\)

In Equation (23), $A_{js}$ and $A_{jd}$ are the equivalent cross-sectional areas of equivalent spring of SLSPJ and DLDPJ, respectively; $\kappa_s$ and $\kappa_d$ are the equivalent elastic stiffness of SLSPJ and DLDPJ, respectively, which are the tangent stiffness of the elastic deformation.
part in the test of SLSPJ and DLDPJ, respectively; \( l_s \) and \( l_d \) represent the connection length of SLSPJ and DLDPJ, respectively.

Combined with the bearing capacity test results of SLSPJ and DLDPJ, under the actual working state, most of the gusset plates are in the elastic state, according to the elastic part data of the load displacement curve in Figure 9. The following Equation is obtained through calculation.

\[
\kappa_s = 35.6 \frac{\text{kN}}{\text{mm}} \quad \kappa_d = 11.4 \frac{\text{kN}}{\text{mm}} \quad (24)
\]

In the actual model, \( l_s = l_d \), which can be obtained by calculation as following.

\[
A_{js}/A_{jd} = 3.12 \quad (25)
\]

Take the section sizes as shown in Equation (26).

\[
A_1 = A \quad A_2 = A/5 \quad A_{jd} = 0.15A \quad A_{js} = 0.47A \quad (26)
\]

Take the tower root opening and the height of the joints as \( h \), take the two-layer tower cells, i.e., \( l = nh \), \( n = 2 \), and the length coefficient of the connecting joint is \( k = 0.3 \). According to Equation (20), the calculation shows that the stable bearing capacity of quadrilateral tower can be improved by 15.6%.

4. Comparative Analysis of Theory and Finite Element Numerical Simulation

In order to consider the influence of the connection joint of the tower on the stable bearing capacity of the tower, the numerical model of the quadrilateral two-layer transmission tower cell is established by using ANSYS 17.0. The root opening of the tower is 4 m and the height is 8 m. The test model of single-limb single-plate and double-limb double-plate connection joints is applied to the finite element model of the quadrilateral tower, and the leg member is circular steel pipe \( \phi 140 \times 8 \text{ mm} \), angle steel \( L50 \times 5 \text{ mm} \) is adopted for the diagonal member, the thickness of a single plate connection is 8 mm, and the thickness of the double plate connection is 4 mm. According to the processing drawing of single-limb single-plate and double-limb double-plate connection (as shown in Figures 2–4), the geometric model of quadrilateral transmission tower is established, as shown in Figure 14.

![Figure 14](https://via.placeholder.com/150)

**Figure 14.** (a) Geometric dimensions of geometric model of quadrilateral tower; (b) three-dimensional drawing of geometric model of quadrilateral tower.
4.1. Finite Element Model

According to Figures 2, 3 and 14, a quadrilateral tower model including single plate connection joint and double plate connection joint is established, as shown in Figure 15.

The elastic modulus of the tower model material is 210 GPa, Poisson’s ratio is 0.3, and the density is 7850 kg/m³. The SHELL63 shell element is used as the element type. In the calculation of the tower model, the gusset plate, connecting angle steel and the bolt hole of angle steel are coupled. The effect of bolt connection is simulated in the form of coupling connection joints to form a finite element model, as shown in Figure 16.

Figure 15. (a) Geometric model of quadrilateral tower with SLSPJ; (b) geometric model of quadrilateral tower with DLDPJ.

The elastic modulus of the tower model material is 210 GPa, Poisson’s ratio is 0.3, and the density is 7850 kg/m³. The SHELL63 shell element is used as the element type. In the calculation of the tower model, the gusset plate, connecting angle steel and the bolt hole of angle steel are coupled. The effect of bolt connection is simulated in the form of coupling connection joints to form a finite element model, as shown in Figure 16.
Figure 16. (a) Finite element model of quadrilateral tower with SLSPJ; (b) finite element model of quadrilateral tower with DLDPJ.

4.2. Finite Element Results Analysis

The tower structure is calculated according to the finite element method (load displacement whole process analysis) considering the geometric deviation of the structure shape. In the analysis, it assumes that the material remains elastic. During the nonlinear whole process analysis of the tower, considering the influence of the installation deviation of the initial member position, the lowest order buckling mode (first order buckling mode) of the structure is taken as the initial geometric defect distribution mode, the maximum calculated value of the defect can be taken as 1/300 of the tower height [31]. The entire process of nonlinear buckling of the tower is analyzed, and the load value at the first critical point is obtained as the nonlinear ultimate bearing capacity of the tower.

Through the nonlinear analysis of the tower structure, the vertical displacement of the quadrilateral tower connected by SLSPJ and DLDPJ is obtained, as shown in Figure 17.
Figure 17. (a) Load bearing capacity and horizontal Z displacement of SLSPJ; (b) load bearing capacity and horizontal Z displacement of DLDPJ.

Through the finite element analysis, the load displacement relationship curve of the stable bearing capacity of the quadrilateral tower with SLSPJ and DLDPJ is shown in Figure 18.

According to the load displacement curve and considering the influence of geometric nonlinearity of tower and connecting joint, the nonlinear stability bearing capacity of quadrilateral tower with double-limb double-plate connecting joint is higher. Using double-limb double-plate connecting joint can improve the nonlinear stability bearing capacity of a transmission tower by 14.9%.
Figure 18. Load displacement curve of tower bearing capacity.

5. Conclusions

In this paper, through the tensile bearing capacity test of single-limb single-plate and double-limb double-plate connection joints, the mechanical characteristics of the connection joints are obtained. The theoretical calculation expression of the stable bearing capacity of quadrilateral tower considering the influence of the axial stiffness of the joints is further derived by the energy method. Through numerical simulation, the influence of double-limb double-plate connection joint on the nonlinear stability bearing capacity of quadrilateral tower is further analyzed, and the following conclusions are obtained.

- The scale test results of single-limb single-plate and double-limb double-plate connection joints show that the bearing capacity of double-limb double-plate connection joint can be increased by 26.1%. The load displacement curve of the joint shows that the elastic stiffness of double-limb double-plate connection joint is 3.12 times that of single-limb single-plate connection joint.

- The influence of the axial stiffness of the joint on the stable bearing capacity of the quadrilateral tower is studied by the energy method. The axial stiffness coefficient is introduced to express the influence of the change of the joint stiffness on the stable bearing capacity of the tower. The results show that with the increase of the axial stiffness coefficient, the stable bearing capacity of the tower first increases and then tends to be stable, and the axial stiffness coefficient has the greatest influence in the range of 0–0.5.

- Combined with the joint test and theoretical analysis, it is found that the DLDPJ can improve the stable bearing capacity of quadrilateral tower by 15.6%.

- The quadrilateral tower numerical model of the test model joint is established, and the whole process of nonlinear buckling of the tower is analyzed. Considering the influence of geometric nonlinearity of the tower and connection joints, it is found that the double-limb double-plate connection joint can improve the nonlinear stability bearing capacity of the transmission tower by 14.9%.
In this paper, the theoretical calculation expression of the stable bearing capacity of the quadrilateral tower considering the axial stiffness of the joint is proposed. The bearing capacity of the double-limb double-plate connection joint is studied from three aspects of the model test, theoretical analysis and numerical simulation, and the research results have been verified in the tower cell and have good engineering application value. However, this paper does not consider the influence of the number of real tower floors and tower inclination. Based on this, the next step will be to carry out the application research of double-limb double-plate connection joints in the full tower.

**Author Contributions:** Conceptualization, T.Z. and H.Y.; methodology, T.Z. and H.Y.; software, T.Z.; validation, T.Z., H.Y. and M.H.; formal analysis, T.Z. and H.Y.; investigation, L.Z. and P.H.; resources, L.Z., Z.L. and M.H.; data curation, Z.L. and M.H.; writing—original draft preparation, T.Z. and H.Y.; writing—review and editing, T.Z. and H.Y.; visualization, T.Z.; supervision, Z.L. and M.H.; project administration, Z.L. and M.H.; funding acquisition, M.H. All authors have read and agreed to the published version of the manuscript.

**Funding:** The research work was funded by the National Natural Science Foundation of China (Project nos. 11572147 and 51568046).

**Institutional Review Board Statement:** Not applicable.

**Informed Consent Statement:** Not applicable.

**Data Availability Statement:** The data presented in this study are available on request from the corresponding author.

**Conflicts of Interest:** The authors declare no conflict of interest.

**References**

1. Murakawa, H.; Deng, D.; Ma, N.; Wang, J. Applications of inherent strain and interface element to simulation of welding deformation in thin plate structures. *Comput. Mater. Sci.* 2012, 51, 43–52. [CrossRef]
2. Zhang, C.; Li, S.; Sun, J.; Wang, Y.; Deng, D. Controlling angular distortion in high strength low alloy steel thick-plate T-joints. *J. Mater. Process. Technol.* 2019, 267, 257–267. [CrossRef]
3. Fu, X.; Du, W.; Li, H.; Li, G.; Dong, Z.; Yang, L. Stress state and failure path of a tension tower in a transmission line under multiple loading conditions. *Thin-Walled Struct.* 2020, 157, 107012. [CrossRef]
4. Xie, Q.; Zhang, J. Experimental study on failure modes and retrofitting method of latticed transmission tower. *Eng. Struct.* 2021, 226, 111365. [CrossRef]
5. Xue, J.; Xiang, Z.; Ou, G. Predicting single freestanding transmission tower time history response during complex wind input through a convolutional neural network based surrogate model. *Eng. Struct.* 2021, 233, 111859. [CrossRef]
6. Roy, S.; Kundu, C.K. State of the art review of wind induced vibration and its control on transmission towers. *Structures* 2021, 29, 254–264. [CrossRef]
7. Ma, H.; Zhao, Y.; Fan, F.; Yu, Z.; Zhi, X. Experimental and numerical study of new connection systems for a large-span hyperbolic steel cooling tower. *Eng. Struct.* 2019, 195, 452–468. [CrossRef]
8. Ma, H.; Yu, Z.; Zhao, Y.; Fan, F. Behavior of HCR semi-rigid joints under complex loads and its effect on stability of steel cooling towers. *Eng. Struct.* 2020, 222, 11062. [CrossRef]
9. Ma, H.; Zhao, Y.; Fan, F.; Xie, P. Nonlinear stability of steel cooling towers with semirigid connections. *Thin-Walled Struct.* 2021, 159, 107164. [CrossRef]
10. Martins, D.; Gonilha, J.; Correia, J.R.; Silvestre, N. Monotonic and cyclic behaviour of a stainless steel cuff system for beam-to-column connections between pultruded I-section GFRP profiles. *Eng. Struct.* 2021, 249, 113294. [CrossRef]
11. Zhang, A.; Xie, Z.; Zhang, Y.; Lin, H. Shaking table test of a prefabricated steel frame structure with all-bolted connections. *Eng. Struct.* 2021, 248, 113273. [CrossRef]
12. Li, C.; Deng, H.; Gao, Y.; Song, X.; Hu, X. Compression analysis of external double-layered flange connection in transmission tower. *Structures* 2021, 33, 3002–3016. [CrossRef]
13. Song, S.; Chen, J.; Xu, F. Mechanical behaviour and design of concrete-filled K and KK CHS connections. *J. Constr. Steel Res.* 2022, 188, 107000. [CrossRef]
14. Tian, L.; Liu, J.; Chen, C.; Guo, L.; Wang, M.; Wang, Z. Experimental and numerical analysis of a novel tubular joint for transmission tower. *J. Constr. Steel Res.* 2020, 164, 105780. [CrossRef]
15. Li, F.; Deng, H.; Hu, X. Experimental and numerical investigation into ultimate capacity of longitudinal plate-to-circular hollow section K- and DK-joints in transmission towers. *Thin-Walled Struct.* 2019, 143, 106240. [CrossRef]
16. Li, F.; Deng, H.; Cai, Q.; Dong, J.; Fu, P. Experiment and design investigation of a multi-planar joint in a transmission tower. *J. Constr. Steel Res.* 2018, 149, 78–94. [CrossRef]
17. Liu, H.; Han, J.; Sun, Z.; Yang, J.; Yang, F.; Xu, X. Efficient method to include joint zones of chord members in finite element model of tubular transmission tower at linear elastic stage. *Int. J. Steel Struct.* 2015, 15, 973–988. [CrossRef]

18. He, X.; Chan, T.; Chung, K. Effect of inter-module connections on progressive collapse behaviour of MiC structures. *J. Constr. Steel Res.* 2021, 185, 106823. [CrossRef]

19. Ma, R.; Yu, L.; Zhang, H.; Tan, L.; Ahmad, B.H.K.; Feng, J.; Cai, J. Experimental and numerical appraisal of steel joints integrated with single- and double-angles for transmission line towers. *Thin-Walled Struct.* 2021, 164, 107833. [CrossRef]

20. Abdelrahman, A.H.A.; Liu, Y.; Chan, S. Advanced joint slip model for single-angle bolted connections considering various effects. *Adv. Struct. Eng.* 2020, 23, 2121–2135. [CrossRef]

21. Gan, Y.; Deng, H.; Li, C. Simplified joint-slippage model of bolted joint in lattice transmission tower. *Structures* 2021, 32, 1192–1206. [CrossRef]

22. An, L.; Wu, J.; Jiang, W. Experimental and numerical study of the axial stiffness of bolted joints in steel lattice transmission tower legs. *Eng. Struct.* 2019, 187, 490–503. [CrossRef]

23. Balagopal, R.; Ramaswamy, A.; Palani, G.S.; Rao, N.P. Simplified bolted connection model for analysis of transmission line towers. *Structures* 2020, 27, 2114–2125. [CrossRef]

24. Szafran, J.; Juszczyk, K.; Kamiński, M. Experiment-based reliability analysis of structural joints in a steel lattice tower. *J. Constr. Steel Res.* 2019, 154, 278–292. [CrossRef]

25. De Souza, R.R.; Miguel, L.F.F.; McClure, G.; Alminhana, F.; Kamiński, J. Reliability assessment of existing transmission line towers considering mechanical model uncertainties. *Eng. Struct.* 2021, 237, 112016. [CrossRef]

26. Yang, F.; Zhu, B.; Li, Z. Numerical analysis and full-scale experiment on K-joint deformations in the crank arms of lattice transmission towers. *Struct. Des. Tall Spec. Build.* 2018, 27, e1448. [CrossRef]

27. Ahmed, K.I.E.; Rajapakse, R.K.N.D.; Gadala, M.S. Influence of Bolted-Joint Slippage on the Response of Transmission Towers Subjected to Frost-Heave. *Adv. Struct. Eng.* 2009, 12, 1–17. [CrossRef]

28. Yan, H.; Nie, X.; Zhang, L.; Yang, F.; Huang, M.; Zhao, T. Test and Finite Element Analysis of a New Type of Double-Limb Double-Plate Connection Joint in Narrow Base Tower. *Materials* 2021, 14, 5936. [CrossRef]

29. Ugural, A.C.; Fenster, S.K. *Advanced Strength and Applied Elasticity*; Prentice Hall: Upper Saddle River, NJ, USA, 1975; pp. 382–394.

30. Huang, M.; Zhang, L.; Chen, Y.; Zhao, T. Oscillation periods of electric transmission lines with and without effect of bending deformation energy. *J. Eng. Math.* 2019, 119, 241–254. [CrossRef]

31. Xu, K. *Analysis and Application of ANSYS Building Structure*; China Construction Industry Press: Beijing, China, 2013.