Dynamic Analysis of 110 kv Double-circuit Compound High-strength Hollow Mezzanine Concrete-filled Steel Tube Pole

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Abstract. Combined with the simplified mechanical model of the pole-line system, the dynamic stability of 110kV hollow mezzanine concrete pole is analyzed and its dynamic equation is established. The main resonance mode in the plane is analyzed by the approximate analytical method, and the stability is determined and solved by the stability theory. The characteristics of wind-induced vibration are revealed from the perspective of nonlinear dynamics. Based on Hamilton principle, the partial differential equation of motion of the simplified dynamic analysis model of the tower column is established, and it is discretized into ordinary differential equation by Galerkin method. Finally, the critical frequency equation is established according to the method proposed by Bolotin, so as to obtain the dynamic stability region and instability region of the tower column. Based on the B-R motion criterion, the nonlinear dynamic stability of the pole-line system is studied, and the initial pretension, wind direction angle and span of the cable are selected for parameter analysis. The influence of these parameters on the nonlinear dynamic stability critical load of the hollow mezzanine concrete pole line system is investigated.

1. Introduction

The complexity of structural dynamic stability and the diversity of dynamic loads and structure types determine the non-uniqueness of the criteria for determining structural dynamic stability. Since the late 1960s, many scholars at domestic and foreign have conducted in-depth studies on structural dynamic stability and achieved a series of results. The Russian scholar Bolotin made a comprehensive analysis and discussion on the problem of dynamic stability under periodic loads in his book "Dynamic Stability of Elastic System" [1]. His idea is to use the Fourier class to represent unknown displacements. The numerical representation is incorporated into the differential equation of motion. And the critical frequency equation is established according to the existence conditions of the solution to obtain the dynamic instability region of the structure. Regarding the dynamic stability of structures under impact loads, many scholars have conducted in-depth research and established corresponding criteria for determining dynamic stability. Budiansky and Roth [2] proposed the B-R motion criterion in 1962 when studying the dynamic stability of the spherical shell. The idea is derived from the instability of extreme points in static stability. Because the criterion is clear in concept, easy to program, and suitable for both conservative and non-conservative systems, etc., it has been widely used in practical engineering. But the criterion also has disadvantages such as large amount of calculation. Hsu CS et al. [3] proposed in 1966 to study the dynamic stability of the structure through the phase plane. The so-called phase plane
refers to a certain relationship graph between displacement and speed. According to the change law of the graph, it is judged whether the structure has dynamic instability. And the upper and lower limits of the dynamic stability conditions are obtained. Since the essence of the phase plane change law reflects the change of the system energy, this criterion is called the Hsu CS energy criterion. However, the criterion is difficult to be used in complex structural systems and can not be used to study the stability of the post-buckling path. Simitess [4] believes that the critical condition of dynamic instability is related to the characteristics of the total potential energy change of the system. In actual engineering, the loads acting on the structure are usually arbitrary loads such as wind loads and earthquakes. This makes such type of dynamic stability problems more complicated. It is difficult to establish relevant criteria from a mathematical view. It is the only way to establish relevant criteria to determine whether the structure has dynamic instability based on the dynamic response. However, some scholars later believed that the introduction of time parameters made the dynamic stability problem far more complicated than the static stability problem, and the structures’ dynamic response is reciprocating, so there has not yet been an applicable and efficient criterion for the dynamic stability problem [5].

Because the hollow mezzanine concrete pole is a wind-sensitive structure, the problem of dynamic stability is directly related to the safety of the entire transmission line. In order to deeply understand the wind-induced vibration characteristics of the system, theoretically analyze the dynamics and reveal its internal vibration mechanism, it is necessary to use a suitable theoretical model as the cornerstone. Therefore, this paper studies the wind-induced vibration of the pole-line simplified mechanical model from the perspective of nonlinear dynamics, so as to have a theoretical grasp and understanding of its dynamic characteristics, and provide a certain theoretical basis and method for further transmission line design.

2. Simplified mechanical model of hollow mezzanine concrete pole

The wind load on the hollow mezzanine concrete pole is very small, which is negligible compared with the top axial pressure. Therefore, the concrete pole can be simplified as an elastic beam hinged at both ends and subjected to axial pressure. For the convenience of analysis, according to the principle of equivalent stiffness, this elastic beam can be regarded as a straight beam with constant cross-section for research. The simplified model of in-plane vibration is shown in Figure 1.
3. Kinetic equation modeling

Hamilton’s principle states that within any time interval \( t_1 \) to \( t_2 \), the variation of kinetic energy and potential energy is equal to zero. Considering that the variation of work done by non-conservative forces is also equal to zero, the expression is:

\[
\int_{t_1}^{t_2} \delta(T - U)dt + \int_{t_1}^{t_2} \delta W_{nc}dt = 0 \quad (1)
\]

In the formula:
- \( T \) is kinetic energy of the system;
- \( U \) is the potential energy of the system, including strain energy and potential energy of conservative external forces;
- \( W_{nc} \) is the work done by non-conservative forces acting on the system;
- \( \delta \) is variation.

The Hamilton principle uses the variational terms of kinetic energy and potential energy to replace inertial force and elastic force, so the equation does not include vector, but only related to scalar, that is, energy.

3.1 Modeling based on Hamilton principle

The variation of system kinetic energy:

Since the variation in the two-time integration limits is zero, the following formula holds:

\[
\delta y(t_1) = \delta y(t_2) = 0 \quad (2)
\]

Substitute the above formula to get:

\[
\int_{t_1}^{t_2} \delta T dt = - \int_{t_1}^{t_2} \int_0^1 [m(x) \frac{\partial^2 y}{\partial t^2} \delta y] dx dt \quad (3)
\]

Since the variational \( \delta y(0), \delta y(l), \delta y'(0), \delta y'(l) \) etc. on the boundary are equal to zero for the displacement boundary condition, and they are arbitrary for the force boundary condition. Therefore, the second and third terms on the right end of the equal sign in the above formula are zero.

So:

\[
\int_{t_1}^{t_2} \delta U_1 dt = \int_{t_1}^{t_2} [\int_0^1 EI(x) \frac{\partial^4 y}{\partial x^4} \delta y] dx dt \quad (4)
\]

Variations of non-conservative external force and damping force of the system are:

\[
\int_{t_1}^{t_2} \delta W_{nc} dt = - \int_{t_1}^{t_2} \int_0^1 [p(t) \frac{\partial^2 y}{\partial x^2} \delta y] dx dt - \int_{t_1}^{t_2} \int_0^1 [c(x) \frac{\partial y}{\partial x} \delta y] dx dt \quad (5)
\]

Equation (7) is the vibration partial differential equation of the system. In order to further solve this equation, the following uses the Galerkin method to discretize the above partial differential equations into ordinary differential equations, and then further uses nonlinear vibration theory to solve the ordinary differential equations.

Take the lateral displacement of the system as:

\[
y(x, t) = \phi(x)f(t) \quad (8)
\]

In the formula, \( \phi(x) \) is the mode function, \( f(t) \) is the modal coordinate.

3.2 Determination of dynamic instability region

The external excitation in equation is not on the inhomogeneous term of the dynamic equation, but on its coefficient term. Parametric vibration is another form of vibration besides free vibration, forced vibration and self-excited vibration. The system that produces parametric vibration is called parametric system [6]. Parametric vibration is a typical nonlinear vibration. In addition to the numerical calculation method, it can also be solved by using the small parameter method and directly establishing the
frequency equation. Small parameter methods mainly include harmonic balance method, multi-scale method, and average method [7] etc. In this paper, the above dynamic equation is solved by directly establishing the frequency equation.

Under the action of dynamic wind load on the tower-line system, the dynamic load \( p(t) \) generated on the top of the tower column by the cables, suspension cables, etc. is a random load, so the Fourier transform is used to convert \( p(t) \) into the superposition of trigonometric functions. That is:

\[
p(t) = p_0 + \Phi(t) \quad (9)
\]

Ignore the numbers much less than 1, so only the elements on the main diagonal in equation are retained, and the approximate boundary of the main unstable region can be obtained approximately, that is:

\[
\frac{\theta}{\omega} = 2\sqrt{1 + \mu P_1} \quad (10)
\]

The relationship of equation (10) can be expressed in Figure 2.

![Fig.2 Relation curve of \( \theta/\omega - \mu P_1 \)](image)

4. Nonlinear dynamic analysis of hollow mezzanine high-strength concrete-filled steel tube pole

Design wind speed of 110kV hollow mezzanine concrete pole is 33m/s. The boundary conditions of the upper end of the windward side pole and the leeward side pole in the vertical direction of the hollow mezzanine steel tube concrete pole line are equivalent to an articulated support, that is, \( kd \) tends to infinity. At this time, the problem is reduced to the problem of parametric resonance of simply supported compression rods. Literature [10] conducted a more systematic theoretical study on this problem.

The cross-sectional area of the conductor is selected according to the 110KV double-circuit concrete-filled steel tube specification, the average tension of the conductor is 85.6kN, and the damping ratio is 0.05.

4.1 The parameters and the form of structure

The main structure of the system is two lines and one pole. These are the hollow mezzanine concrete-filled steel tube pole and the conductors on both sides. The height of the tower is 50m, and there is no height difference on both sides of the wire. Its structure diagram is shown in Figure 3.
We use the large-scale general finite element software ANSYS. Because the wire is a cable structure and can only be pulled, the wire uses Link180 unit. The tower is mainly subjected to compression and bending, so Beam188 unit is used. Modeling the wire and the transmission tower as a whole, the finite element model diagram is shown in Figure 4.

The transmission tower is a hollow mezzanine concrete-filled steel tube pole section, which is composed of outer steel pipe-concrete-inner steel pipe. The cross-sectional form is shown in Figure 5. At the bottom of the tower, the outer steel pipe diameter is 750mm, and the inner steel pipe is 600mm. The diameter of the steel pipe changes uniformly along the height of the tower. The taper of the outer steel pipe is 1/100 and the ratio of diameter to thickness is 50; the taper of the inner steel pipe is 9/1000 and the ratio of diameter to thickness is 40. The hollow ratio of the steel pipe rod: 0.47-0.64.
Table 1 Pole and tower material properties

| material          | Q420     | C80      |
|-------------------|----------|----------|
| Elastic Modulus (Pa) | 3.50E+10 | 2.10E+11 |
| density (kg·m⁻³) | 2600     | 7850     |
| Poisson's ratio   | 0.30     | 0.24     |

The material properties of the wire material are shown in Table 2

Table 2 Wire material properties

| Wire parameters       | unit     | Value    |
|-----------------------|----------|----------|
| section (mm²)         |          | 225.24   |
| diameter (mm)         |          | 15.01    |
| Single (kg/m)         |          | 0.682    |
| Coefficient of elasticity (N/mm²) | | 65000    |
| Linear expansion coefficient (10⁻⁶/K) | | 20.5     |

4.2 Wind speed simulation

The synthesis of fluctuating wind speed is based on the cross-power spectrum density matrix, so a reasonable wind speed spectrum and coherence function must be selected before wind speed synthesis. The wind speed spectrum, cross power spectrum and coherence function are respectively:

\[
\frac{\omega S_\omega(\omega)}{\nu_\omega} = \frac{4k \omega^2}{(1 + x^2)^{4/3}}
\]

(11)

In the formula, \( \omega \) is the circular frequency; \( k \) is the ground roughness coefficient; \( \bar{v}_{10} \) is the average wind speed at a height of 10m; \( U(z_i) \) and \( U(z_j) \) are the average wind speed at heights \( z_i \) and \( z_j \) respectively; \( C_r \) is \( x, y, \) the attenuation coefficients in the three directions of \( z; x = 1200 \omega / \bar{v}_{10}. \)

In the formula, \( \omega \) is the circular frequency; \( k \) is the ground roughness coefficient; \( \bar{v}_{10} \) is the average wind speed at a height of 10m; \( U(z_i) \) and \( U(z_j) \) are the average wind speed at heights \( z_i \) and \( z_j; C_r \) is the attenuation coefficient of \( x, y, z \) three directions; \( x = 1200 \omega / \bar{v}_{10}. \)

Table 3 Wind speed simulation parameters

| parameter               | value | parameter               | value |
|-------------------------|-------|-------------------------|-------|
| Cutoff frequency (rad/s) | 4π    | Lateral attenuation coefficient \( C_y \) | 16    |
| Frequency equal fraction | 2048  | Vertical attenuation coefficient \( C_z \) | 10    |
| Simulation time interval (s) | 0.125 | Analog points | 154   |
| Longitudinal attenuation coefficient \( C_x \) | 6     | Sample time (s) | 128   |

Wind load calculation

The wind direction is perpendicular to the pitch direction. The static wind load on the wire and pole tower can be calculated as follow:

\[
F_g = \frac{1}{2} \rho U_g^2 C_D A_n \quad (12)
\]

In the formula, \( F_g \) is the wind load of per unit length of the member (N/m); \( \rho \) is air density (1.25 kg/m³); \( U_g \) is equivalent static gust wind speed at the reference height of the component (m/s); \( C_D \) is the damping coefficient. For a round section with a smooth surface, we take 0.6; \( A_n \) is the projected area of per unit length component in the downwind direction (m²/m).
4.3 Dynamic response of structure under disconnection load + wind load

Considering that the wind speed is 60m/s, we analyze the dynamic response of the pole after one side of the wire is broken at the wire and the pole tower. After considering the disconnection and wind load, we analyze the dynamic response of the structure to obtain the displacement time history of the entire structure, as shown in Figures 6 to 8. Extract the maximum displacement of the top of the rod, the maximum bending moment of the bottom of the rod, and the maximum Mises stress from the results. The related curve of gear span is shown in Figure 6 ~ Figure 8.

![Displacement time history of the tower under different gears](image1)

a) top displacement along the wire span direction  

b) displacement in the direction of vertical span

Figure 6 Displacement time history of the tower under different gears

![Time history diagram of bottom bending moment under different gears](image2)

a) Bending moment along the wire pitch direction  

b) bending moment in vertical wire direction

Fig. 7 Time history diagram of bottom bending moment under different gears
Fig. 8 Mises stress time history diagram of the sole

Numerical simulation results after wire breakage show that, under different spans, the bending moment in each direction of the tower bottom increases with the increase of the span, and the maximum Mises stress at the bottom of the tower also increases with the span.

5. Conclusion

The dynamic equation established by the simplified mechanical model belongs to the category of parametric vibration, which is a typical nonlinear vibration, and the main resonance mode of the system is likely to appear in the plane. The main conclusion is:

1. The dynamic equation established based on the simplified mechanical model of the hollow sandwich steel tube concrete pole belongs to the category of parametric vibration and is a typical nonlinear vibration. The main resonance mode of the system is more likely to appear in the plane. The appearance of the main resonance will make the system unstable and eventually lead to the destruction of the structure. The dynamic equation is solved by the approximate analytical method, and the ratio between the frequency of the external load resonant force and the frequency of the system itself is given through the Lyapunov first-order approximation theory to meet the conditions to keep the system in a certain stability range.

2. We establish the partial differential motion equation of the simplified model of the tower based on the Hamilton principle, and use the Galerkin method to discretize it into ordinary differential equations. Based on the method proposed by Bolotin, the critical frequency equation of the simplified model of the pole is established, and the dynamic stability and unstable regions of the pole are obtained.

3. The B-R motion criterion is used to study the nonlinear dynamic stability of the pole system, and select the initial pretension, wind direction angle and span of the wire for parameter analysis, the influence of parameters on the nonlinear dynamic response of the rod-line system is investigated.

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