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Coupling characteristics of creep fracture of rock foundation on wind turbine under wind induced vibration

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Abstract: This paper presents the elastoplastic creep fracture characteristics and bearing stability of rock foundation for wind turbine based on the influence of wind load. Combined with the superstructure-concrete foundation-rock foundation lay load and boundary conditions, the wind load standard value and wind turbine system composition were obtained under normal wind condition, and then the forced vibration model of viscous damping two-degree-of-freedom system was established. Furthermore, the mode characteristics, frequency characteristic equations and the relations of the first and second order natural frequencies were obtained. According to the plastic yield characteristics of power hard rock base material, the analytical expressions of displacement, plastic zone and plastic state of I-II composite crack were obtained by coupling Mohr-Coulomb plastic yield condition and creep fracture characteristics. Simultaneously, the improved nonlinear creep model equation and accelerated creep fracture time were also obtained based on improved Kelvin nonlinear accelerated creep model. Combined with the example, it is verified that the accelerated creep displacement and crack propagation of rock foundation are obvious, considering the wind bracing and creep characteristics of rock foundation. Final, the failure modes of rock-foundation are compressive shear, local shear and bending-shear. It is necessary to timely reinforce the interface of foundation rock foundation.

Key words: wind turbine; wind load; mechanical condition of foundation-soil structure; vibration equation; coupling characteristics of creep fracture

1. Introduction

Wind power generation is mainly to convert wind energy into electricity for living and production. Wind power generation is one of the new renewable energy, which has been widely used in the development of low-carbon economy and green social economy. It is clear that wind turbine is mainly composed of engine room, tower cylinder, foundation and rock and soil bearing layer and it is a towering structure. In addition, its overall stability is mainly controlled by the bearing capacity of subsoil foundation. However, once the subsoil foundation is unstable and destroyed, the tower frame will collapse, and the result will be disastrous. A large number of tower frame collapse accidents show that the direct causes of disasters are bad climatic conditions and bad geological environment, such as strong wind and showers, thunderstorms, freezing, uneven settlement of foundation, failure of foundation bearing capacity and other inducing factors. Especially, under the strong wind load, it often causes the wind-induced fracture of tower frame, the bending and shear failure of the interface between tower frame, foundation and ground rock and soil, the abrupt displacement of the supporting rock layer, and the connection of cracks and fractures. Even accompanied by sick operation of wind turbines, it has posed a major threat to the
safety of wind turbines, which has aroused widespread concern of many scientific and technological workers (Vaughan, 1987; Veletsos, & Verbic, 1973). Currently, according to wind-induced disaster research illustrated tall structures and high-rise buildings, mainly concentration in the transmission lines is sensitive to wind vibration engineering (Rogers et al., 2002; Kim, & Tamura, 2013), which has been in the wind load response theory research, the wind vibration control technology, the wind tunnel experiment, simulation experiment, specifications and intellectual property rights, and so on. We have already accumulated the rich practical experience and theoretical fruits. Yang et al. (2015) have conducted a lot of work on the wind-load time-history numerical simulation and wind tunnel experiment of the single-column transmission cable tower system, and obtained the research results of the internal force distribution characteristics of each component of the high-voltage transmission tower, the optimization and selection of experimental materials, and the wind resistance setting of viscoelastic dampers. Fang et al. (2004) have proposed numerous theoretical achievements in solving vibration mode coefficients under wind loads, calculating equivalent static wind loads, time-history analysis of high-order vibration modes, and seismic design of high-rise buildings, which have been widely applied in engineering practice. Hatada et al. (2015) carried out structural dynamic analysis and stability evaluation of communication pylons by applying wind vibration responses of multiple wind speed spectra, summarized the time-history distribution characteristics of stress and displacement of pylons under strong winds, and verified the universality of the standard formula to obtain fluctuating wind load. The study is just around the upper structure wind load response of dynamic mechanical characteristics, but seldom consider the lower inherent mechanical properties of rock mass structural materials and its stability affected by wind load counter attack tall structures. However, the research of high structure stability evaluation of the wind vibration and wind resistance design has its limitations (Ken et al., 2015; Osiptsov et al., 2020).

Subsoil foundation is the main carrier of wind turbine. Compared with the superstructure, the rock and soil materials of the substructure have the characteristics of lower strength, more complex failure modes and greater difference in mechanical properties. These characteristics have a great influence on the stable state of wind turbine. Under fluctuating wind load, the additional stress and the mechanical properties inherent in creep of the fractured strata show a more complex and nonlinear process of accelerating failure (Hamza, & Stace, 2018). At the same time, the research literature is rare, and even wind vibration response is foundation-rock structure viscous damping two degree of freedom forced vibration model, the failure modes of fluctuating wind load foundation and nonlinear acceleration creep property of many problems unsolved, such as the influence of its stability. In this paper, based on the instability mechanism of wind-induced foundation, the vertical displacement characteristics and mechanical conditions of foundation rock load were analyzed theoretically and numerically, which can provide reference and technical support for wind vibration control and design parameters of foundation reinforcement.

2. Mechanical analysis of rock foundation under wind load

2.1 Wind load

This paper takes the onshore wind power generation system as the object. According to the composition of wind turbine, the engine room and tower frame are the superstructure, and the
foundation and rock bearing layer are the substructure. The superstructure mainly bears the wind load directly, and the substructure is also seriously affected by the natural environment and construction condition under the condition of the wind load transmitted, the additional stress of the base and the upper self-weight. These influencing factors are the important root cause of wind-induced disaster unit collapse (Li, & Li, 2016). It is obvious that wind load is one of the most important loads acting on land wind turbine. In addition, its standard value is mainly composed of average wind pressure and fluctuating wind pressure. Meanwhile, the wind turbine is a towering structure, and the superstructure is mainly affected by downwind gradient wind and wind shear. Wind-induced response mainly causes the tower tube to be destroyed by excessive deformation in the form of bending, shear and torsion. The lower foundation structure is mainly affected by wind load transfer and foundation bearing capacity. These mechanical properties have important influence on foundation stability. At the same time, the bending moment and shear force caused by the upper wind load also have an important influence on the mechanical properties and deformation of rock mass. Furthermore, promote the shear and punching failure of the foundation. Therefore, the mechanical effect of wind load on the whole wind turbine and the deformation of foundation can not be ignored.

According to literature (Zhang et al., 2008), under normal wind conditions, the standard wind load value is mainly composed of average wind and fluctuating wind to determine the standard wind load value. At present, the pulse wind speed spectrum mainly includes as following Davenport spectrum, Kaimal spectrum and Simiu spectrum. Considering the energy of onshore fluctuating wind, it is mainly concentrated in the low frequency band. Moreover, the wind vibration response of the instability failure of the towering structure is present in the middle and lower part of the structure. In addition, the height variation characteristics of the wind speed spectrum are not considered temporarily in the wind load calculation of onshore wind farm. Therefore, Kaimal fluctuating wind speed spectrum is adopted in the study of wind-induced disasters of tower drum, and the power spectrum of its downwind horizontal fluctuating wind speed is expressed as (Zhang et al., 2008):

(1) Follow the direction of the wind

\[
\frac{nS_u(n)}{v_s^2} = \frac{105nz}{U_z(1+\frac{33nz}{U_z})^{5/3}}
\]  

\[ (1) \]

(2) Vertical direction

\[
\frac{nS_w(n)}{v_s^2} = \frac{2.1nz}{U_z(1+\frac{5.3nz}{U_z})^{5/3}}
\]  

\[ (2) \]

Where, \( S_u(n) \), \( S_w(n) \) are the self-spectral density functions of downwind and vertical fluctuating wind speed processes, respectively, \( n \) is frequency, \( v_s \) is shear wave velocity of wind load, \( z \) is height above ground, and \( U_z \) is average wave velocity at height of \( z \).
Under normal wind conditions, the distribution of average wind speed at the height of wind turbine within 10min is determined according to Rayleigh distribution, which can be obtained from the following equation:

\[ v_k = 1 - \exp\left(-\pi (v_h / 2v)\right) \]  

(3)

The standard deviation of wind speed in normal turbulence model was determined by 90% quantile of wind speed at given hub height. It can be expressed as:

\[ \sigma_v = P_f (0.75v_h + b) \]  

(4)

Where, \( P_f \) is the turbulence intensity at the hub height, and \( b \) is the calculated parameter.

Meanwhile, the fluctuating wind load acting on the tower tube structure can be calculated by the following Eq.(5):

\[ P_d = \frac{1}{2} A_t \mu_s \rho_a (v_{10} + v_f)^2 \]  

(5)

Where, \( v_{10} \) is the average value of the basic wind speed, \( \rho_a \) is the density of the air, \( A_t \) is the windscreen area of the tower, \( \mu_s \) is the wind load shape coefficient, and \( v_f \) is the fluctuating wind speed.

2.2 wind vibration model

As a random vibration load, wind load has a continuous mechanical effect on the stability of high-rise structures. In particular, wind load has the duality of driving impeller and wind vibration response to wind turbine. Therefore, the negative effects caused by wind load should be strictly controlled. According to the inherent mechanical properties and strength differences of geotechnical materials of wind turbine substructure foundation, the foundation is regarded as forced vibration of viscous damping two-degree-of-freedom system in wind vibration study. The specific vibration model is shown in Fig.1.

Fig.1 Forced vibration of foundation-soil

In Fig.1, \( k_1, k_2, k_3 \) are the stiffness coefficients of tower tube foundation interface, foundation interface, soil and stable soil layer, respectively, and \( c_1, c_2, c_3 \) are the damping of the above structures, respectively.

According to the mechanical equilibrium of the system, the vibration equation can be derived from the following equation:

\[
\begin{align*}
M_{11} \ddot{x}_1 + C_{11} \dot{x}_1 + C_{12} \dot{x}_2 + K_{11} x_1 + K_{12} x_2 &= F_{1e}(t) \\
M_{22} \ddot{x}_2 + C_{21} \dot{x}_1 + C_{22} \dot{x}_2 + K_{21} x_1 + K_{22} x_2 &= F_{2e}(t)
\end{align*}
\]  

(6)

Where, \( M_{11} = m_1 \), \( M_{22} = m_2 \), \( C_{11} = c_1 + c_2 \), \( C_{12} = C_{21} = c_2 \), \( C_{22} = c_2 + c_3 \), \( K_{11} = k_1 + k_2 \), \( K_{12} = k_3 \).
Combined with resonance characteristics, the solution of Eq. (6) of inhomogeneous vibration has the following form as:

\[
x_1 = A_1 \cos(ft) + B_1 \sin(ft) \\
x_2 = A_2 \cos(ft) + B_2 \sin(ft)
\]  

(7)

The first and second derivatives of Eq. (7) are derived into Eq.(6), and it can be obtained as follows:

\[
\begin{align*}
x_1' &= -A_1 f^2 \cos(ft) - B_1 f^2 \sin(ft) \\
x_2' &= -A_2 f^2 \cos(ft) - B_2 f^2 \sin(ft)
\end{align*}
\]  

(8)

Then, substituting Eqs. (7) - (8) into Eq.(6), further, it can be proposed as follows:

\[
\begin{bmatrix}
C_{11}f - C_{11}f & K_{14} - K_{21} - M_{12}f^2 \\
-C_{12}f & K_{12} + M_{22}f^2 - C_{22} - K_{22}
\end{bmatrix}
\begin{bmatrix}
\sin(ft) \\
\cos(ft)
\end{bmatrix} = \begin{bmatrix} 0 \\ 0 \end{bmatrix}
\]  

(9)

Taking \( \sin(ft) \) and \( \cos(ft) \) as unknowns and considering that the coefficient of the determinant is zero, the frequency characteristic equation of wind turbine foundation and rock foundation can be obtained as follows:

\[af^4 + bf^2 + c = 0\]  

(10)

By solving the eigenvalues of Eq.(10): \( f_1^2 \) and \( f_2^2 \), the first-order natural frequency and the second-order natural frequency of foundation-rock foundation can be obtained.

2.3 load calculation of rock foundation

Wind turbine lower foundation usually adopts foundation ring and anchor bolt structure. Because of its high bearing capacity, simple design and complex geological and topographical conditions. In addition, the foundation ring has been widely used in onshore wind farms. According to the load condition and mode analysis of wind turbine, it is mainly considered that the tower cylinder is prone to produce the rotational deformation in the downwind direction under the horizontal wind load based on the fluctuating wind speed spectrum, thus forming the rotational bending moment and shear force. Considering foundation stiffness and displacement constraints, the vibration mode of the fan system is bending-shear type with up-bend and down-shear. In addition, the dynamic interaction between tower tube and foundation will produce shear force \( Q(t) \) and bending moment \( M(t) \) on the interface between them. As the soil layer covered by fan foundation is thin, the influence of soil pressure inside the buried depth range of foundation is not considered(Hutchins et al., 2012). Fig.2 shows the load distribution of the base-rock matrix structure.
Fig. 2 Distribution of load on foundation-rock foundation structure

Combined with the composition of average wind and fluctuating wind to determine the calculation method of wind load standard value, the wind-induced shear force at the top of foundation ring is calculated as follows:

\[ Q(f) = \int_0^μ \pi D^2 A_1 \mu_1 \rho_2 \left( v_{10} + v_i \right)^2 \frac{dz}{8 \cos θ} \]  

The wind-induced bending moment at the top of the foundation ring is calculated as follows:

\[ M(f) = \int_0^μ \pi D^2 A_1 \mu_1 \rho_2 \left( v_{10} + v_i \right)^2 \frac{dz}{8 \cos θ} \cdot z \cdot dz + \frac{1}{2} H \cdot G \cdot \tan θ \]

where

\[ θ = \frac{(1 + ν) A_1 \mu_1 \rho_2 (v_{10} + v_i)^2}{E} \]

Where, \( H \) is the height from the hub to the top surface of the foundation, \( D \) is the diameter of the tower cylinder, \( G \) is the dead weight of the superstructure, and \( E, ν \) is the elastic parameter of the superstructure.

3. fracture characteristics of fractured rock foundation

The surface load of wind turbine is transmitted to the rock foundation through the foundation ring, so the bearing capacity of the rock foundation determines the stable working condition of the fan. Furthermore, the stress condition, deformation characteristics and crack propagation control of the foundation are very important. Usually, under the action of excessive load, the deformation characteristics such as punching, shear and uneven settlement are produced, which easily induce the crack propagation and coalescing of fractured rock and accelerate the fracture failure of rock foundation. The crack propagation deformation is shown in Fig. 3.

Fig. 3 Crack expands and deforms under load

According to the load distribution of foundation-rock mass matrix structure, the main load distribution of rock foundation cracks is compressive shear load, under the action of compressive shear load, rock foundation cracks produce composite shear deformation, namely I-II type composite crack. Fracture failure of fissured rock base usually occurs when the stress at the crack front reaches the yield condition and produces a plastic zone, which further expands to produce fracture failure.

According to the yield failure characteristics of rock, the Mohr-Coulomb plastic yield condition was adopted in this paper under positive pressure and negative tension conditions:
\[ f = \frac{1}{3} I_1 \sin \phi - (\cos \theta_x + \frac{1}{\sqrt{3}} \sin \theta_x \sin \phi) \sqrt{J_2} + c \cos \varphi \]  

(13)

where

\[ \theta_x = \frac{1}{3} \sin^{-1} \left[ -\frac{3\sqrt{3}}{2} \frac{J_3}{(\sqrt{J_2})^3} \right] \]

Where, \( I_1 \) is the first invariant of stress tensor, \( J_2, J_3 \) is the second and three invariants of stress skew, respectively.

Combined with the stress distribution characteristics of the crack surface, the normal stress and shear stress in the crack direction are expressed as follows:

\[ \sigma_n = \frac{1}{2\sqrt{2\pi r}} \left[ K_1 \cos \frac{\theta}{2} \sin \theta (1 + \cos \theta) - 3K_\eta \cos \frac{\theta}{2} \sin^2 \theta \right] \]  

(14)

\[ \tau_n = \frac{1}{2\sqrt{2\pi r}} \left[ K_1 \cos \frac{\theta}{2} (\cos \theta + \cos^2 \theta) - \frac{3}{2} K_\eta \cos \frac{\theta}{2} \sin \theta \right] \]  

(15)

According to the Mohr-Coulomb theory, the principal stress distribution of I-II type composite crack under plane strain condition is proposed as follows(Zhang et al., 2012):

\[ \sigma_1 = \sigma_n + \frac{\tau_n}{\cos \varphi} (1 + \sin \varphi) = \frac{1}{2\sqrt{2\pi r}} \left[ K_1 (A + \frac{1+\sin \varphi}{\cos \varphi} C) - K_\eta (B + \frac{1+\sin \varphi}{\cos \varphi} D) \right] \]  

(16)

\[ \sigma_2 = 0 \]  

(17)

\[ \sigma_3 = \tau_n \tan \varphi + \sigma_n = \frac{1}{2\sqrt{2\pi r}} \left[ K_1 (C \tan \varphi + A) - K_\eta (D \tan \varphi + B) \right] \]  

(18)

where

\[ A = \cos \frac{\theta}{2} \sin \theta (1 + \cos \theta) \]

\[ B = 3 \cos \frac{\theta}{2} \sin^2 \theta \]

\[ C = \cos \frac{\theta}{2} (\cos \theta + \cos^2 \theta) \]

\[ D = \frac{3}{2} \cos \frac{\theta}{2} \sin \theta \]

Furthermore, the Mohr-Coulomb plastic yield conditions were obtained:

\[ f = \frac{1}{3} I_1 \sin \phi - (\cos \theta_x + \frac{1}{\sqrt{3}} \sin \theta_x \sin \phi) \sqrt{J_2} + c \cos \varphi \]  

(19)

For rock-base power hardening materials, we can obtain:

\[ \sigma = m \varepsilon^n \]  

(20)
According to the Mohr-Coulomb plastic yield relation and combined with the crack distribution, the upper limit of the ultimate load on the base is:

\[ P = b \sigma_n \frac{B}{S} (W-a) \]  

(21)

Where, \( P \) is the upper limit of ultimate load of the base, \( \bar{a} \) is the crack length, \( (W-a) \) is the thickness of rock bridge, and \( B \) is the width of rock foundation.

Combined with the relation between COD and J integral under elastic-plastic condition (Landers, & Begley, 1974), the integral of crack \( (J_P) \) under plastic yield is obtained as follows:

\[ J_P = \frac{\sigma_s COD}{2r} \]  

(22)

Where, \( \sigma_s \) is the yield stress, \( r \) is the plastic zone radius, and \( COD \) is the crack tip opening displacement.

According to Dugdale model, crack tip opening displacement is written as follows:

\[ COD = \frac{2.55 \sigma_s}{E} \frac{a \ln \sec \left( \frac{\pi P}{2 \sigma_s} \right)}{a \ln \sec \left( \frac{\pi P}{2 \sigma_s} \right)} \]  

(23)

Combining Eqs. (22)- (23), the integral of crack \( (J_P) \) is obtained as follows:

\[ J_P = \frac{1.275 a \sigma_s^2}{Er} \ln \sec \left( \frac{\pi P}{2 \sigma_s} \right) \]  

(24)

4. Coupling characteristics of creep fracture

4.1 Nonlinear creep analysis

It is clear that creep is an inherent mechanical property of rock mass material. It is the deformation under external load that develops over time. When the creep load exceeds the long-term strength of rock mass, the accelerated deformation will occur. At the same time, the engineering rock mass is in a complicated underground environment for a long time. Hence, the long-term strength determines the aging deformation and stability of rock mass structure.

At the initial stage of reinforcement, the resin rock mass structure presents instantaneous elastic-viscous deformation, which gradually recovers with the development of time and the bonding effect increases. With the further increase of the load, the structural stress of resin rock mass is adjusted and the creep expansion of cracks occurs. Namely, the plastic shear yield condition is reached. According to the deformation composition of resin rock mass structure and shear failure criterion of fractured rock mass, combined with Mohr-Coulomb plastic yield condition, M-C element was used as nonlinear creep element and was connected in series with classical Kelvin element to form a generalized Kelvin nonlinear creep element model. Moreover, the improved creep model has simple structure and few components, and the model parameters can be easily identified through experiments, integral transformation, polynomial fitting and other
methods. Meanwhile, the improved model can also effectively characterize the three-stage creep characteristics of the resin rock mass structure. The improved nonlinear creep model is shown in Fig.4.

**Fig.4** Nonlinearity creep model of modified Kelvin model

According to the nonlinear element model theory, creep formulas of linear Kelvin body and nonlinear M-C body are obtained, and the nonlinear element is mainly reflected by nonlinear viscosity coefficient.

As linear Kelvin body, the creep equations can be written as follows:

\[ S_{ij} = 2\eta \delta_{ij} + 2Ge_{ij} \]  

\[ \varepsilon = \frac{\sigma}{\eta(t, \sigma)} \left( \frac{\tau}{\tau_f} \right)^n \]  

1) When \( \sigma \) is less than \( \sigma_g \), the nonlinear coefficient of viscosity is expressed as follow:

\[ < \eta(t, \sigma) > = \infty \]

2) When \( \sigma \) is greater than or equal to \( \sigma_g \), the nonlinear coefficient of viscosity is expressed as follow:

\[ < \eta(t, \sigma) > = At(ln\frac{t_0}{t} + 1) + B[\arctan(\frac{\sigma}{\sigma_r}) + \frac{\pi}{2}] \]

Nonlinear M-C body is depicted as follows:

\[ \delta_{ij} = \frac{\lambda}{\eta} \frac{\partial g}{\partial \sigma_{ij}} - \frac{1}{3} \delta_{ij} \]  

\[ e_\sigma^\rho = s_j \left( \frac{1}{2G^\rho} + \frac{1}{k} \int d\lambda \right) \]

\[ \delta_{\mu} = \lambda \left[ \frac{\partial g}{\partial \sigma_{11}} + \frac{\partial g}{\partial \sigma_{22}} + \frac{\partial g}{\partial \sigma_{33}} \right] \]

\[ \lambda = \frac{G}{\eta} \]

\[ s_j = ks_j^0 \]

Where, \( k \) is the creep experimental parameters, \( A, B \) is the creep experimental constant, and
\( S_{ij}, e_{ij} \) is the partial stress and strain of toughened resin rock mass structure, respectively, \( \eta \) is the viscosity coefficient, \( G \) is the shear modulus, \( g \) is the plastic potential function, \( \delta_{ij} \) is the Kirschner symbol, \( \sigma_{11}, \sigma_{33} \) are the maximum and minimum principal stress, respectively, \( \sigma \) is the plastic yield limit of M-C body, \( \sigma_i \) is the tensile strength.

According to the Mohr-Coulomb plastic yield condition, the improved nonlinear creep model equation is obtained by applying the principle of strain addition:

\[
e_y = \frac{S_y}{2G} [1 - \exp(-\frac{G}{\eta})] + s_y (\frac{1}{2G^p} + \frac{1}{k} \int d\lambda)
\]

(29)

Considering the plastic yield failure condition of rock mass, the accelerated creep failure time \( t_F \) of foundation rock mass can be further obtained as follows:

\[
t_F = t_0 + \frac{1}{\int_0^{b} k e_0 \exp(\pi dz) - \tau_f}
\]

(30)

Where, \( t_0 \) is the initial yield time of bedrock body, \( \tau_f \) is the initial yield strength of rock mass structure, and \( e_0 \) is the initial strain of accelerated creep.

In the parameter identification of the improved nonlinear creep model, the model calculation parameters and long-term strength are usually obtained according to the creep curves and creep isochronic curves of different loads, that is, the long-term strength under \( F(\infty) \) certain creep load is \( t = \infty \), where the creep time is \( t = \infty \). In fact, when the long-term strength is reached, accelerated failure deformation occurs.

4. 2 Coupling characteristics of creep fracture

Under the action of superstructure and foundation load, rock foundation will produce creep deformation. With the increase of load, the rock foundation will be destroyed by accelerated creep deformation when the yield limit is reached. At the same time, the increase of load will induce the formation and propagation of main cracks. When the crack expands, the strength of bedrock decreases and the stress state changes constantly, which makes the creep stress reach the yield limit of bedrock body and accelerates the creep failure.

According to creep characteristics of self-built nonlinear M-C body, when creep load reaches the yield limit of bedrock body, M-C body will produce accelerated plastic deformation. Furthermore, the fracture creep equation of creep effect bedrock body can be obtained as follows:

\[
e_i = \frac{2\sigma_i - \sigma_\tau}{6G} [1 - \exp(-\frac{G}{\eta(t, \sigma)})] + \frac{2\sigma_i - \sigma_\tau}{3} (\frac{1}{2G^p} + \frac{1}{k} \int d\lambda)
\]

(31)
Eqs. (16) - (18) are substituted into Eqs. (30) - (31) respectively to obtain the principal stress at the crack tip.

Furthermore, the time of accelerated creep failure \( t_F \) of foundation rock mass can be further obtained as follows:

\[
t_F = t_0 + \frac{1}{\int_0^k k e_{0r} \exp(\pi dz) - \frac{1}{2\sqrt{2\pi} r_p} [K_t \cos \theta_0 (\cos \theta_0 + \cos^2 \theta_0) - \frac{3}{2} K_n \cos \theta_0 \sin 2\theta_0]}
\]

According to the maximum normal stress theory, the direction angle of crack propagation \( \theta_0 \), the plastic zone of crack tip and the time of accelerated creep failure can be obtained.

5. Example analysis

In order to verify the stability influence of rock foundation creep fracture on wind turbine, according to the shear failure characteristics of rock and taking shear inclined crack power hardening rock foundation as the research object, the influence of tower wind vibration effect of base pressure on the creep fracture of rock foundation was analyzed. Considering the wind-induced vibration response, the additional stress of foundation is 4 times the depth and 2 times the width of foundation. The calculation range is characterized by width of 38m, and thickness of 10m. The foundation structure adopts the foundation ring expanded slab concrete structure, the design strength is set to 310MPa with concrete strength of C30. The foundation holding layer is the rock layer, calculate parameters adopted in material are: unit weight 26.5kN/m3, poisson's ratio 0.25, internal friction angle 44°, bonding force is 1100kPa. The instantaneous elastic modulus is set to 22 GPa, viscoelastic modulus is 2.5 GPa, viscoelastic coefficient is 7.6×10³ MPaˑD, damping ratio is 0.03, wind load shape coefficient is 1.466 and basic wind pressure is 0.5 kPa. Tower frame height is 70m, foundation buried depth is 2.5m. In the simulation, the boundary conditions for the lower is fixed to constraint surrounding the foundation soil, however the upper is freedom. In order to eliminate the boundary effect, viscous damping is set in the lower part and left and right sides of the calculation model, respectively.

Through the improved Kelvin nonlinear accelerated creep model, the finite difference calculation program is developed for coupling characteristics of creep fracture of rock foundation on wind turbine under wind induced vibration. On the wind turbine foundation-batholith structure, the vertical displacement, plastic zone, the acceleration of crack extension, wind-induced disaster creep failure time and failure state were obtained in the four cases as following: (a) without considering wind load, creep and fracture, (b) considering wind load without creep and fracture, (c) considering wind load creep without fracture and (d) considering creep fracture under wind load. The calculation model is divided into a total of 16884 nodes and 84141 tetrahedron unit. In the simulation, boundary conditions for the lower is fixed to constraint surrounding the foundation soil, however the upper is freedom. In order to eliminate the boundary effect, in the lower part of the
calculation model, the left and right sides, respectively, is set up the viscous damping. Its
calculation model, boundary conditions and mesh division are shown in Figs.5-6.

Fig.5 Sketch of calculation and boundary (Unit:m).

Fig.6 Calculated meshing

Fig.7 Vertical displacement distribution under various working conditions: (a) Without considering wind
load, creep and fracture, (b) Considering wind load without creep and fracture, (c) Consider wind load
creep without fracture \( (t=18h) \), (d) Considering creep fracture under wind load \( (t_0=26.2h) \), and (e)

Consider creep fracture under wind load \( (t_f=27.1h) \)

Fig.7 illustrates the vertical displacement distribution. On the wind turbine concrete
foundation-subsoil foundation frame, vertical displacement is mainly negative displacement ,
which existed in bottom. Without considering wind load, creep and fracture, in view of foundation
and rock foundation lay, the maximum vertical displacement is 9.7 mm, mainly distributing at the
top of the base ring. However, the vertical displacement of concrete foundation and deep rock
foundation lay is very small. Meanwhile, wind turbine appears stable state. Considering wind load
without creep and fracture, the instantaneous maximum vertical displacement of rock foundation is
1.5cm, which is mainly distributed in the downwind foundation ring and the right side of concrete
foundation. Considering wind load creep without fracture \( t=18h \), the maximum vertical displacement of
rock foundation is 2.8cm, which develops horizontally and deeply from foundation circumferential
to rock base. After wind load is applied for 18h, the vertical displacement of rock foundation also
appears 4mm. Considering creep fracture under wind load, the vertical displacement of concrete
foundation and rock foundation continues to increase. After wind load continues 26.2 h , the
vertical displacement of rock foundation is more than 3.3 cm. Meanwhile, the maximal
displacement are mainly distributed in the leeward based ring to interface with rock base.
Compared with the creep without fracture, the vertical displacement increased by 120%
, hence,displacement rate is significantly. At the same time, there is also a vertical displacement of
1cm at the position of foundation cone slope, and the foundation has obvious deformation, which
has seriously affected the stability of concrete foundation and rock foundation. Furthermore, it is
shown from the failure state of the wind-induced disaster concrete foundation and rock foundation.
Meanwhile, the wind load is 27.1 h, the maximum vertical displacement is to 13 cm, which appears
accelerating deformation status, base bending shear, compressive shear destruction, differential
settlement of rock at the grass-roots level and the shear destruction. Evidently, the former mainly is
caused by wind load transferring the bending moment and shear force at the bottom of the tower
drum. The differential settlement of rock foundation is caused by eccentric additional stress of
foundation and appears shear instability.

Fig.8 Distribution of plastic zone under various working conditions: (a) Without considering wind load, creep and fracture, (b) Considering wind load without creep and fracture, (c) Consider wind load creep without fracture \( t=18h \), (d) Considering creep fracture under wind load \( t_0=26.2h \), (e) Considering creep fracture under wind load \( t_0=27.1h \), and (f) The creep fracture failure modes under wind load are considered \( t_f=27.1h \).

Fig.8 illustrates the distribution of plastic zone under various cases. The plastic zone are mainly shear without considering wind load, creep and fracture, and the plastic zone are mainly distributed at the bottom of the base ring. Moreover, with the strength of the play, the plastic zone decreases and through plastic zone has been not formed. Meanwhile, wind turbine appears stable state. Considering wind load without creep and fracture, the plastic zone is mainly distributed in the downwind foundation ring and the shallow area on the right side of concrete foundation, and the plastic zone increases and gradually breaks through. Consider wind load creep without fracture, the plastic zone of foundation ring and shallow layer on the right side of concrete foundation appears to increase in the downwind direction, and develops from the deep of foundation ring to the deep of rock base. After 18h of wind load, the plastic zone of rock foundation increases in the deep. When the creep fracture under wind load is considered, the plastic zone of the foundation and rock base continues to increase. After 26.2h wind load, the plastic zone at the bottom of the foundation ring appears and continues to expand, which has seriously affected the stability of the foundation and rock base. After 27.1h of wind load, the plastic zone at the bottom of the foundation ring was connected, and a shear zone was formed in the crack area of the rock base. Meanwhile, the rock foundation showed obvious shear failure.

6. Conclusion

Theoretical research and engineering application of wind load response of wind turbine foundation rock structure are carried out in terms of load distribution, vibration characteristics, creep fracture effect of rock mass, additional stress of base, plastic zone, crack growth and accelerated creep failure time. Meanwhile, the following conclusions are obtained.

According to the composition of the wind turbine, the wind turbine is divided into two parts including the upper part and the lower part by the interface between the tower frame and foundation. According to the material physical and mechanical properties, the system composition and boundary conditions meet the viscous damping forced vibration model of two degree of freedom system, further vibration equation of concrete foundation and rock foundation is obtained, and the first order natural frequency and the second order natural frequency of concrete foundation and rock foundation are obtained.

According to the wind vibration response and the deformation characteristics of rock foundation in wind power system, the nonlinear creep model, the nonlinear characteristics of viscosity coefficient, the time of accelerated creep failure, the analytic formula of maximum additional
stress of basement and the motion equation of rock mass are also established. Combined with the calculation examples, it is verified that the vertical displacement change rate and the plastic zone of the foundation structure are significant considering creep fracture effect of the foundation under wind load. The resulting differential settlement and the penetration of the plastic zone pose a serious threat to the stability of the wind turbine. Therefore, improving the strength of the interface between concrete foundation and rock foundation on shallow surface rock mass in the bearing layer, reducing the propagation of cracks, and enhancing the shear capacity of the concrete in the main wind direction at the top of the foundation and the cone slope are important ways to reduce instability of the subsoil foundation structure caused by wind disasters.

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References

Fang, C.X., Chen, R. and Xue, S.T. (2004). Effect of pile-soil-structure interaction on downwind response of highrise building to wind-induced vibration, *Chinese Journal of rock mechanics and engineering*, 23, 2078-2084. doi:1000-6915(2004)12-2078-07

Guo, Y., Sun, B.N., Ye, Y., Lou, W.J. and Shen, G.H. (2009). Frequency-domain analysis on wind-induced dynamic response and vibration control of long span transmission line system. *ACTA AERODYNAMICA SINICA*, 27, 288-295. doi:025821825(2019)0320288208

Hamza, O. and Stace, R. (2018). Creep properties of intact and fractured muddy siltstone. *International Journal of Rock Mechanics & Mining Sciences*, 106, 109-116. doi: 10.1016/j.ijrmms.2018.03.006

Hatada, T., Kobori, T., Ishida, M., and Niwa, N. (2015). Dynamic analysis of structures with maxwell model. *Earthquake Engineering & Structural Dynamics*, 29(2), 159-176. doi:10.1002/(SICI)1096-9845(200002)29:2<159::AID-EQE895>3.0.CO;2-1

Hutchins, N., Chauhan, K., Marusic, I., Monty, J. and Klewicki, J. (2012). Towards reconciling the large-scale structure of turbulent boundary layers in the atmosphere and laboratory. *Boundary-Layer Meteorology*, 145, 273-306. doi:10. 1007/s10546-012-9735-4

Ken, G., Gholamreza, N., and Hadi, M. (2015). Creep deformation of fracture surfaces analysis in a hydraulically fractured reservoir using the finite element method. *Journal of Petroleum and Gas Engineering*, 6(6), 62-73. doi:10.5897/JPGE 2015.0217

Kim, Y.C. and Tamura, Y. (2013). Effects of surrounding buildings on quasi-static wind load combinations for a low-rise building. *Journal of Structural & Construction
Landers, I. D., and Begley, J. A. (1974). Test results from J-integral studies: an attempt to establish a \( J_{IC} \) testing procedure. *ASTM STP, 560*, 170-186.

doi:10.1520/STP33140S

Li, Y. G., and Li, Q. S. (2016). Influence of fundamental mode shapes on equivalent static wind loads of tall buildings. *Earthquake engineering and engineering dynamics*, 36, 38-44.
doi:10.13197/j.eceev.2016.06.38.liyg.005

Osiptsov, A., Garaga Sh, I. A., Boronin, S. A., Tolmacheva, K. I., and Paderin, G. (2020). Impact of flowback dynamics on fracture conductivity. *Journal of Petroleum Science and Engineering*, 188, 106822.
doi:10.1016/j.petrol.2019.106822

Rogers, A. L., Manwell, J. F. and Megowan, J. G. (2002). Design requirements for medium-sized wind turbines for remote and hybrid power systems. *Renewable Energy*, 26(2), 157-168.
doi:10.1016/S0960-1481(01)00126-4

Veletsos, A. S. and Verbic, B. (1973). Vibration of Viscoelastic foundation. *Earthquake Engineering and Structural Dynamics*, 2, 87-102.
doi:10.1002/eqe.4290020108

Yang, W. G., Wang, Z. Q., Zhu, B. W. and Qi, L. Z. (2015). Time history analysis on wind-induced response of UHV guyed single-mast transmission-line system. *Proceedings of the CSCE*, 3, 3182-3191.
doi:10.13334/j.0258-8013.pcsee.2015.12.032

Zhang, X. M., Wang, L., Yan, B. and Zhang, P. Y. (2012). Fracture mechanics. *Tsinghua University Press*, BeiJing, China.

Zhang, W. F. and Ma, C. H. (2008). Notes on power spectrum density and wind-velocity PSD. *Chinese journal of computational mechanics*, 25, 474-477.
doi:1007-4708(2008)04-0474-04
Figures:

**Fig. 1** Forced vibration of foundation-soil

**Fig. 2** Distribution of load on foundation-rock foundation structure

**Fig. 3** Crack expands and deforms under load
Fig. 4 Nonlinearity creep model of modified Kelvin model.

Fig. 5 Sketch of calculation and boundary (Unit: m).

Fig. 6 Calculated meshing.
Fig. 7 Vertical displacement distribution under various working conditions: (a) Without considering wind load, creep and fracture, (b) Considering wind load without creep and fracture, (c) Consider wind load creep without fracture \( (t = 18\text{h})\), (d) Considering creep fracture under wind load \( (t_0 = 26.2\text{h})\), and (e) Consider creep fracture under wind load \( (t_F = 27.1\text{h})\)
Figure 8 Distribution of plastic zone under various working conditions: (a) Without considering wind load, creep and fracture, (b) Considering wind load without creep and fracture, (c) Consider wind load creep without fracture \( (t = 18 \text{h}) \), (d) Considering creep fracture under wind load \( (t_0 = 26.2 \text{h}) \), (e) Considering creep fracture under wind load \( (t = 27.1 \text{h}) \), and (f) The creep fracture failure modes under wind load are considered \( (t_F = 27.1 \text{h}) \).