Analysis of wind time-varying performance of reinforced concrete electric pole under Marine atmospheric environment

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Abstract. Bearing capacity calculation model of concrete electric pole considering concrete long-term performance degradation and steel corrosion is established. The horizontal bearing capacity and wind resistance of typical concrete electric pole under Marine atmospheric environment are analyzed with service time. The effects of concrete protective layer thickness, stirrup diameter and strength coefficient on the long-term wind resistance of electric pole are discussed. It is found that with the increase of service time, the shear strength of the concrete pole deteriorates faster than that of the flexural strength, and the failure mode changes from bending failure to shear failure. The wind resistance of the pole after long service can be improved by increasing the thickness of the protective layer, adopting larger diameter stirrup and higher strength coefficient. The bearing capacity calculation model of concrete electric pole considering concrete long-term performance degradation and steel corrosion is established. The horizontal bearing capacity and wind resistance of typical concrete electric pole under Marine atmospheric environment are analyzed with service time. The effects of concrete protective layer thickness, stirrup diameter and strength coefficient on the long-term wind resistance of electric pole are discussed. It is found that with the increase of service time, the shear strength of the concrete pole deteriorates faster than that of the flexural strength, and the failure mode changes from bending failure to shear failure. The wind resistance of the pole after long service can be improved by increasing the thickness of the protective layer, adopting larger diameter stirrup and higher strength coefficient.

Keywords: concrete pole, naval air environment, wind-resistant performance, Time-varying performance, failure mode.

1. Introduction

Ring-section reinforced concrete pole has the advantages of beautiful appearance, reasonable force, good durability, etc., and is widely used in 220kV and below transmission and distribution lines. However, in recent years, the disaster data of the lines in the coastal areas of our country show that a large number of reinforced concrete poles are broken during typhoon disasters. For example, in 2014, under the influence of Typhoon Rammasun, the 10 kV lines in Hainan Province broke 9020 bases, the 10 kV lines in Guangdong province fell, broke and tilted poles and towers totalled more than 9000 bases, and the 111859 lines fell, broke and tilted poles in Guangxi Province [1]. In 2016, Typhoon Meranti caused more than 4000 foundations of 10 kV line downed in Fujian [2]; In 2020, Typhoon Mikla landed in Fujian, causing a large number of power poles in Zhangzhou, Xiamen and other places to be broken, and nearly 800,000 households were affected by electricity [3].

The main reasons for the fracture of reinforced concrete electric poles in typhoon are low design wind speed [1,3] and insufficient bearing capacity of electric poles [1,4,5,6,7]. In addition to insufficient wind bearing capacity of poles due to low design wind speed [1,7], unqualified performance of raw materials [6,7], improper manufacturing process control [6,7] and unsatisfactory construction quality [1,6] will also cause insufficient bearing capacity of poles. In addition, long-term environmental erosion will cause degradation damage to the performance of concrete materials and corrosion of steel bars, which will degrade the bearing capacity of electric poles [4,5,7,8]. Literature

[8] investigated the current situation of concrete electric poles in North China and found that the poles in mountainous areas and plains were basically intact after decades of use, while those in coastal areas had obvious cracks and longitudinal cracks, and steel bars were leaking and corroded seriously. The performance degradation of concrete electric pole under long-term Marine atmospheric environment needs to be further studied. Literature [9] tested the state of the concrete electric pole that has been in operation for nearly 50 years, and found that the concrete was basically completely carbonized, the steel bar had been corroded, and the bending failure mode during the test was typical steel bar tensile failure. In reference [4], based on the experimental research and theoretical analysis, the evaluation method of the safety grade of electric poles was proposed based on the two factors of concrete defects and steel bar rust. At present, there are no reports on the load bearing and wind resistance of concrete pole with the consideration of concrete long-term performance degradation and steel corrosion.

In this paper, the bearing capacity calculation model of concrete electric pole is established considering the long-term performance degradation of concrete and steel corrosion, and then the horizontal bearing performance and wind resistance of typical concrete electric pole are analyzed with the service time, and then the influence of concrete protective layer thickness, stirrup diameter and strength coefficient on the long-term wind resistance of electric pole is discussed. Finally, the corresponding design suggestions are given.

2. Mechanical time-varying properties of materials

2.1 Rebar

1) Rusting time

The rusting time of steel bars in reinforced concrete electric poles under Marine atmospheric environment can be calculated by formula (1) [10].

$$t_1 = (c/K_{Cl})^2 \times 10^{-6} \tag{1}$$

Where, the rusting time of steel bar at position t1, a; c is the thickness of concrete protective layer, mm; K_{Cl} is the chloride erosion coefficient, calculated according to equation (2).

$$K_{Cl} = 2\sqrt{D_{Cl}}erf^{-1}(1 - M_{cr}/M_s) \tag{2}$$

Where, D_{Cl} is chloride ion diffusion coefficient, m^2/a; erf is the error function; M_{cr} is the critical chloride ion concentration of steel bar corrosion, kg/m3, which can be taken as shown in Table 1. M_s is the chloride ion concentration on the concrete surface, kg/m^3, When no measured data is available, the value can be taken according to Table 2, where the value is the chloride ion concentration on the concrete surface 0.1km away from the coast, and other locations can be corrected according to Table 3.

Table 1. Critical chlorideion concentration for steel bar corrosion

$f_{cu,k}$ /MPa	40	30	≤ 25
M_{cr} /(kg/m^3)	1.4	1.3	1.2

a. $f_{cu,k}$ is the standard compressive strength of concrete cube; When the concrete strength is higher than C40, the critical chloride ion concentration increases by $0.1kg/m^3$ for every 10MPa increase in strength.

Table 2. Chlorideion concentration on concrete surface

$f_{cu,k}$ /MPa	40	30	25	≤ 25
M_{cr} /(kg/m^3)	3.2	4.0	4.6	5.2

Table 3. Correction coefficient of chloride ion concentration on concrete surface

S /km	0	0.1	0.25	0.50	1.00
ξ	1.96	1.00	0.66	0.44	0.33

b. S is the distance of the structure from the coastline; ξ is the correction factor.

Chlorideion diffusion coefficient D_{Cl} is calculated according to equation (3) [11,12].

$$D_{Cl} = D_{Cl,ref} f_1(T) f_2(RH) \tag{3}$$

$$D_{Cl,ref} = 10^{-12.06+2.4w/c} \tag{4}$$

$$f_1(T) = e^{\left[\frac{U}{R}\left(\frac{1}{T_{ref}}-\frac{1}{T_c}\right)\right]} \tag{5}$$

$$f_2(RH) = \left[1+\frac{(1-H_a)^4}{(1-H_c)^4}\right]^{-1} \tag{6}$$

Where, $D_{Cl,ref}$ is the reference value of chloride ion diffusion coefficient, m^2/s; $f_1(T)$ is the temperature influence coefficient of chloride ion diffusion. $f_2(RH)$ is the humidity influence coefficient of chloride ion diffusion. w/c is the water-cement ratio; U is the activation energy of the diffusion process, U = 35000J/mol; R is the gas constant, R = 8.134J/(mol·K); T_{ref} is the reference temperature of concrete, T_{ref} = 293K; T_c is the actual temperature of concrete, K; H_a is the relative humidity; H_c is the critical relative humidity, H_c = 75%.

2) Degree of corrosion of steel bars

The corrosion current density of rebar after corrosion can be calculated by equation (7) [13].

$$ln(1.08i_{corr}) = 8.37 + 0.618ln(1.69w_{Cl}) - \frac{3034}{T_s} - 0.000105R_c + 2.32t_c^{-0.215} \tag{7}$$

Where, i_{corr} is the corrosion current density of reinforcement, $\mu A/cm^2$; W_{Cl} is chlorideion concentration, kg /m^3; T_s is the steel bar surface temperature, K; R_c is the resistivity of concrete, which can be directly measured in actual engineering. If there is no specific measured data, it can be calculated by equation (8) [14]; t_c is the corrosion time of steel bar, a.

$$R_c = (750605\, w/c - 106228)EXP\left[-0.4417W_{r,Cl} - 7.7213S + 2889\left(\frac{1}{T_s}-\frac{1}{303}\right)\right] \tag{8}$$

Where, $W_{r,Cl}$ are chlorideion content in concrete (percentage of cement mass), taking 0.35%; S is the pore saturation of concrete, which is 0.70.

The relationship between steel bar diameter change and corrosion current density can be calculated by equation (9).

$$\Delta D(t) = 0.0232 i_{corr} t \tag{9}$$

Where, ΔD (t) is the change of steel bar diameter, mm; t is the corrosion time of the steel bar.

3) Mechanical properties of steel bars after corrosion

In literature [15,16], the variation laws of yield strength, ultimate strength, elongation and elastic modulus of polished round steel and ribbed steel with different degrees of corrosion with different corrosion rates were tested by artificial climate environment test, and it was considered that corrosion had the same effect on polished round steel and ribbed steel. Through regression analysis, when the corrosion rate η_{avg} of steel was less than 6%, The relationship between material yield strength and corrosion rate is shown in equation (10).

$$f_{y,s} = (1 - 0.031\eta_{avg}) f_{y,0} \tag{10}$$

In the formula, $f_{y,s}$, $f_{y,0}$ are the yield strength of the post-steel bar and the uncorroded steel bar respectively.

2.2 Concrete

On the basis of summarizing and analyzing the long-term exposure tests of concrete at home and abroad and the measured results of buildings over the years, literature [17] analyzed the law of the change of concrete compressive strength with time under Marine environment, and proposed the mathematical model of concrete strength in time, as shown in Formula (11).

$$\mu_f(t) = \left[1.2488e^{-0.0347(lnt-0.3468)^2}\right]\mu_0 \quad (11)$$

Where, $\mu_f(t)$ and μ_0 are the compressive strength of concrete after t and t = 0 years (that is, 28 days), respectively.

3. Degradation of bearing capacity and wind resistance with service time

3.1 Bearing capacity degradation

This paper analyzes the variation of load-bearing performance of a conical single-pole linear electric pole with time. It is assumed that the pole is 0.1km from the shoreline, with ϕ230mm in top diameter and 390mm in root diameter, with 16ϕ12 HPB300 threaded steel bar at the center of the section at the fixation point, and HPB300 spiral stir-bar in diameter 3 with pitch S = 120mm. The concrete strength grade is C40, the water-cement ratio is 0.42, the protective layer thickness of stirrup is 12mm, and the protective layer thickness of longitudinal reinforcement is 15mm. The ambient temperature is 25 ℃ and the relative humidity is 77%.

According to formula (2) to formula (7), the rusting time of stirrup is 27a, and the rusting time of longitudinal rib is 42a. The corrosion current density of reinforcement can be calculated from formula (8) and (9), and the development of the corrosion rate of stirrup and longitudinal reinforcement with service time can be converted from formula (10), as shown in Figure 1.a. It can be seen that the corrosion rate of steel bars increases with the increase of service time. The corrosion is fast in the initial stage, and then tends to be stable. The curve is steeper in the early stage and has little change in the slope in the later stage. In the stable stage, the corrosion rate of stirrup increases faster than that of longitudinal reinforcement. The main reason is that the diameter of stirrup is smaller, and the corrosion rate of stirrup is larger. In service for 50a, the rusting rates of stirrup and longitudinal reinforcement were 15.74% and 1.28%, respectively. Service 95a, stirrup completely corroded (actually due to concrete cracks, later corrosion speed may be faster, earlier than 95a is completely corroded); After 100 years of service, the corrosion rate of longitudinal bar was 4.50%.

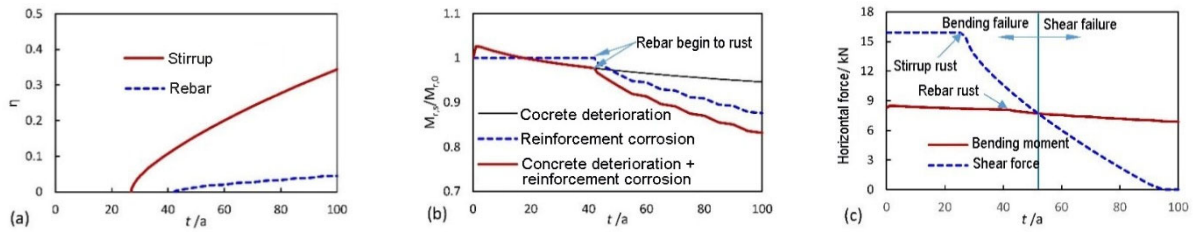


Figure 1. Performance degradation of concrete pole with service time: (a) steel corrosion rate; (b) root bending capacity; (c) Horizontal bearing capacity.

According to formula (1) and formula (11), the strength of concrete and steel bar after different service times can be calculated respectively. The bearing capacity calculation method of the circular reinforced concrete transmission tower in reference [17] calculated the bearing capacity of the tower root bending moment at different service times, and took the bearing capacity at t = 0 as the standard to carry out dimensionless, and obtained the change law of the bearing capacity with the service time, as shown in Figure 1.b. As can be seen from the figure, whether the concrete performance degradation is considered alone, or the concrete performance degradation and steel corrosion are considered at the same time, the bending capacity of the pole is reduced to different degrees. In service for 50a, the bearing capacity was 2.8% and 3.5% respectively considering concrete performance degradation and steel corrosion, while the performance degradation of the two materials

was 6.2%. After serving 100a, the bending capacity decreased by 5.4%, 13.4% and 13.8% respectively due to concrete degradation, steel corrosion and both degradation.

Stirrup rust early, high corrosion rate, mainly bear the horizontal shear. Figure 1.c shows the change of the horizontal bearing capacity of the pole over the service time, and also shows the horizontal bearing capacity determined by the bending failure, which is obtained by dividing the bending moment by the moment arm when the pole is tested. It can be seen from the figure that the horizontal shear bearing capacity of the pole decreases after the stirrup rusts, and the decreasing speed is much greater than the horizontal bearing capacity controlled by bending failure. When the service time is 52a, the shear bearing capacity is equivalent to the horizontal bearing capacity under flexural control. With the increase of service time, the failure mode of the pole changes from bending failure to shear failure.

3.2 Wind resistance degradation

According to the degradation of flexural and horizontal bearing capacity of concrete pole root with service time obtained above, the degradation of wind speed corresponding to flexural failure and shear failure of the pole can be calculated respectively, as shown in Figure 2. It can be seen that with the increase of service time, the decreasing speed of the wind speed corresponding to the shear failure of the pole is much higher than that corresponding to the bending failure of the pole. When the service time increased from 0a to 44a, the maximum wind speed that the pole can withstand decreased from 47m/s to 45m/s, a decrease of 4.3%. During this period, the failure mode of the pole is bending failure. When the service time was increased from 44a to 52a, the maximum wind speed that the pole could withstand was reduced from 45m/s to 41m/s, a decrease of 9.1%. During this period, the failure mode of the pole changes to shear failure. When the service time exceeds 52a, the maximum wind speed that the pole can resist is less than the local basic wind speed of 41m/s, and the pole fails due to the degradation of the material properties.

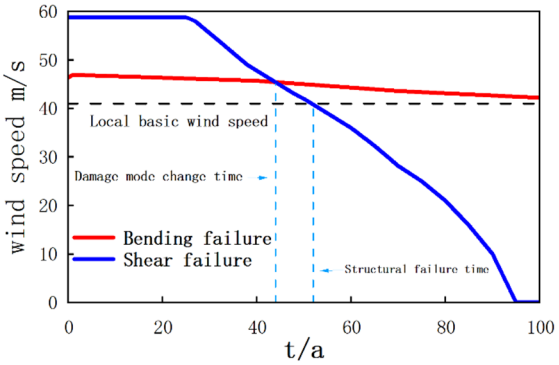


Figure 2. Degradation of wind resistance of concrete pole with service time.

3.3 Analysis of influence factors

1) Thickness of the protective layer

The size of the protective layer thickness c directly affects the rusting time of the steel bar. The national standard "Ring concrete Pole" (GB4623-2014) [19] only requires the thickness of the longitudinal reinforcement protective layer to be no less than 15mm, and does not require the protective layer of the stirrup. The Code for the Design of Concrete Structures (GB50010-2010) [20] is based on the outermost edge of the outermost reinforcement (i.e. stirrups) to calculate the thickness of the protective layer, and different protective layer thickness requirements are adopted in different environments. The rusting time t_i calculated when the protective layer thickness c of the stirrup is 12mm and 15mm respectively (that is, when the protective layer thickness of the longitudinal reinforcement is 15mm and 18mm respectively), the strength of the steel bar $f_{y,50}$ / $f_{y,0}$ during the service period of 50a, and the bearing capacity of the pole $P_{s,50}$ /$P_{s,0}$ are listed in Table 4. It can be seen from the table that the larger the protective layer of stirrup, the later the stirrup rust; When GB4623-2014 is met, although the nominal strength of 50a stirrup in service is reduced by more than

50%, due to the control effect of bending at this stage, the nominal strength of longitudinal reinforcement is decreased by 16% and the horizontal resistance of member is decreased by 6%. When the protective layer thickness of the stirrup is 15mm, the service period is 50a, the longitudinal reinforcement is not rusted, and the horizontal bearing capacity of the member is reduced by 2.8%.

Table 4. Calculation results of different protective layer thicknesses serving 50a

c /mm	$t_{i,s}$ /a	$t_{i,r}$ /a	$f_{ys,50}/f_{ys,0}$	$f_{yr,50}/f_{yr,0}$	$P_{s,50}/P_{s,0}$
12	27	42	0.51	0.84	0.94
15	42	non-corroded	0.84	1.000	0.972

2) Stirrup diameter

In order to compare the long-term wind resistance of specimens with different stirrup diameters, the stirrup diameter and stirrup spacing S were changed at the same time, while A_{sv}/S remained unchanged, and the relationship between the horizontal bearing capacity and service time of the three specimens with stirrup diameters of 2mm, 3mm and 4mm was analyzed, as shown in Figure 3. The position of the longitudinal bar is unchanged, and the thickness of the protective layer of the stirrup is slightly different due to the change of the diameter of the stirrup. Due to the small thickness of the protective layer, the larger diameter stirrup begins to rust first. However, after corrosion, the shear strength of stirrup specimens with larger diameter decreases slowly, because the cross-section weakening rate of stirrup specimens with larger diameter is lower under the same corrosion thickness. For specimens with stirrup diameters of 2mm, 3mm and 4mm, the service time controlled by shear resistance was 36a, 42a and 55a, respectively. The service time of complete loss of horizontal carrying capacity was 62a, 84a and over 100a, respectively. Therefore, the larger diameter stirrup can be used as far as possible in the design to improve the wind resistance of the pole after long-term service.

3) Strength coefficient

From the previous analysis, it can be seen that the shear capacity of the pole deteriorates faster than the bending performance. In order to ensure that the pole does not fail after long-term service, relatively high shear capacity can be adopted in the design. Define the strength coefficient $\rho=H_v/H_m$, where H_v and H_m are the horizontal bearing capacity determined by the shear and bending performance of the pole, respectively. Figure 4 shows the relationship between horizontal bearing capacity and service time when ρ is 1.0, 1.5 and 2.0, respectively. It can be found from the figure that when ρ=1.0, the horizontal bearing capacity of the member decreases significantly after the stirrup rusts. For the samples ρ=1.5 and ρ=2.0, the stirrup rusted and the bearing capacity of the component did not decrease significantly. When H_v is less than Hm, the bearing capacity of these two specimens decreases significantly. For the specimen ρ=2.0, when the service time is about 50a, its bearing capacity is controlled by the shear performance, and the bearing capacity does not decrease significantly before. In order to ensure that the horizontal bearing performance of the pole does not decrease significantly after long-term service, a higher strength coefficient is recommended.

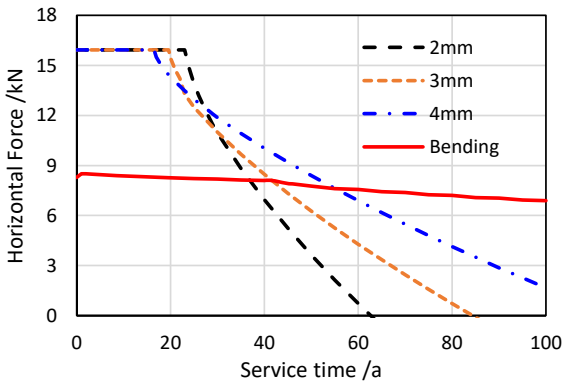


Figure 3. Resistance to horizontal force degradation of electric poles with different stirrup diameters during long service time.

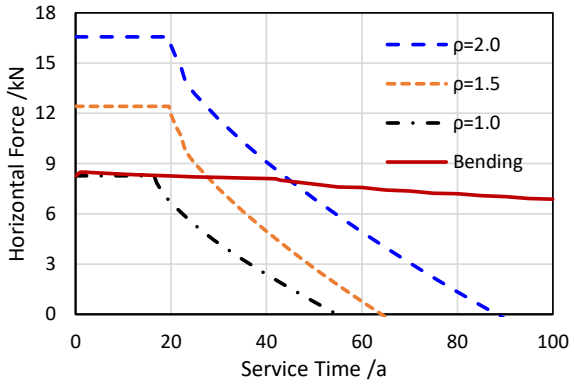


Figure 4. Long-term resistance to horizontal force degradation of electric poles with different shear bearing capacity.

4. Conclusion

(1) In the Marine atmosphere environment, with the increase of service time of concrete pole, the shear strength of the pole deteriorates faster than the bending strength, and the failure mode will change from bending failure to shear failure.

(2) The thickness of the protective layer significantly affects the rusting time of the stirrup, and the long-term wind resistance of the pole can be improved by increasing the thickness of the protective layer.

(3) The stirrup with a larger diameter can be used as far as possible in the design of the pole to improve its wind resistance after long-term service.

(4) In order to ensure that the horizontal bearing performance of the pole does not decrease significantly after long-term service, it is recommended to use a higher strength coefficient (ρ=2.0).

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