

## Article

# Coating-Corrosion Coupled Durability Design of Prestressed Rock Bolt Foundations for Coastal Onshore Wind Turbines in Harsh Corrosive Environments

Jian Xu <sup>1</sup>, Dongpo Dong <sup>2</sup>, Zhiqian Xing <sup>3,\*</sup>, Jing Huang <sup>1</sup>, Jianwei Su <sup>1</sup>, Wenbo Zhou <sup>4</sup>, Da Luo <sup>3</sup>, Ao Zhang <sup>3</sup>, Changqing Bi <sup>3</sup> and Xueyun Xing <sup>5</sup>

<sup>1</sup> Dongfang Electric Power Engineering Consulting Corporation, Chengdu 611731, China; xujian@dongfang.com (J.X.); huangjing06@dongfang.com (J.H.); sujw@dongfang.com (J.S.)

<sup>2</sup> Southwest Municipal Engineering Design & Research Institute of China, Chengdu 610213, China; dongdongpo@ccccltd.cn

<sup>3</sup> Key Laboratory of Urban Security and Disaster Engineering of Ministry of Education, Beijing University of Technology, Beijing 100124, China; roundal@163.com (D.L.); zhangao0930@student.jlju.edu.cn (A.Z.); 17871617918@163.com (C.B.)

<sup>4</sup> Yichang Three Gorges Secondary Specialized School, Yichang 443111, China; zwb505335173@163.com

<sup>5</sup> China Railway 18th Bureau Group No. 4 Engineering Co Ltd, Tianjin 300350, China; 2500510038@fzu.edu.cn

\* Correspondence: zhiqianxing@emails.bjut.edu.cn

## Abstract

Under complex terrains and extreme environmental conditions such as high wind speeds, prestressed rock anchor foundations for onshore wind turbines are subjected to multiple coupled adverse effects during long-term service, including cyclic wind loading, temperature variation, groundwater intrusion, and rock mass weathering. These factors significantly affect structural performance and service life through corrosion and material degradation processes, while conventional design methods mainly focus on ultimate bearing capacity and lack a systematic consideration of corrosion-induced deterioration mechanisms and long-term performance evolution. Without changing the theoretical framework of current design codes, this study introduces a durability-oriented design concept and explicitly incorporates corrosion effects and material degradation into the analytical system of prestressed rock anchor foundations. First, from the perspective of anchor force evolution, a time-dependent analysis method for long-term prestress loss is established, considering the coupled effects of steel corrosion, material relaxation, and cyclic loading. Second, for the mechanical behavior of group anchor systems, a shear capacity model is proposed that accounts for rock mass strength degradation and grout–rock interface deterioration. Meanwhile, the coupling relationship between foundation void development and groundwater seepage is analyzed, revealing its critical role in the corrosion evolution process. On this basis, a coordinated design method for foundation dimensions and prestress parameters is developed to satisfy both load-bearing capacity and durability requirements. Finite element analysis is further conducted to verify the stress and deformation characteristics of the foundation–rock–anchor system under nonlinear conditions. Engineering case studies demonstrate that the proposed method not only meets bearing capacity requirements, but also effectively suppresses void development, reduces corrosion risk, delays structural performance degradation, and improves long-term service reliability. The findings provide a theoretical basis and engineering reference for the durability design and lifecycle performance optimization of prestressed rock anchor foundations for onshore wind power structures in extreme environments. Furthermore, the

Academic Editors: Edoardo Proverbio and Yanxin Qiao

Received: 26 June 2026

Revised: 16 July 2026

Accepted: 21 July 2026

Published: 22 July 2026

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study underscores the critical role of advanced anti-corrosion coatings and surface protection systems in mitigating the coupled corrosion-degradation mechanisms, aligning with the scope of this Special Issue on corrosion protection and durability of infrastructure in harsh environments.

**Keywords:** coastal onshore wind power; prestressed rock anchors; durability design; anti-corrosion coatings; void–seepage–corrosion coupling; corrosive environments

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## 1. Introduction

With the advancement of the “dual carbon” strategy, wind power, as a key component of the clean energy system, is playing an increasingly important role in the transformation of China’s energy structure [1]. In recent years, the installed capacity of onshore wind power has grown rapidly, with continuously increasing single-turbine capacity and tower height, leading to a significant rise in the overturning moment and dynamic loads borne by wind turbine foundations [2]. According to statistics, in high-altitude mountainous regions, high-wind zones, and arid Gobi areas with complex terrain and extreme environmental conditions, the overturning moment of a single foundation has generally reached the order of  $2 \times 10^5$  kN·m, imposing higher requirements on structural stability and long-term service performance [3]. Addressing these durability challenges necessitates a multi-pronged strategy. Beyond robust structural design, effective corrosion protection measures, particularly advanced coating systems, are paramount. As highlighted in this Special Issue, the development and application of novel coatings, such as graphene-enhanced, self-healing, or multi-layer systems, offer a promising pathway to isolate anchor materials from aggressive environmental media, thereby fundamentally retarding the electrochemical corrosion processes that drive prestress loss and interface degradation.

Although complex terrain regions are rich in wind resources, their geological and environmental conditions exhibit pronounced extremity and uncertainty, including high-frequency wind loading, significant temperature variations, groundwater-induced erosion, and rock mass weathering [4]. These unfavorable factors act on foundation structures over long-term service periods through coupled mechanisms, accelerating anchor prestress loss, rock mass degradation, and foundation detachment, thereby affecting the overall reliability and safety of wind turbines [5,6].

Under these multi-factor coupled effects, corrosion-induced material and interface degradation has gradually become one of the key factors affecting the long-term performance of prestressed rock anchor foundations. Groundwater and wind-driven precipitation can provide a sustained humid environment for the anchoring system. When the water contains dissolved oxygen and aggressive ions, electrochemical corrosion of anchor steel is likely to occur, resulting in a reduction in effective cross-sectional area and mechanical performance. Meanwhile, under long-term hydrochemical action and temperature cycling, the grout may experience dissolution, pore structure deterioration, and microcrack propagation, thereby weakening the bond performance at the grout–rock interface. The rock mass itself also exhibits time-dependent degradation of strength parameters under weathering and hydro–rock interaction. Under cyclic loading, these corrosion and deterioration processes further couple with fatigue effects, accelerating structural degradation and the development of prestress loss. Therefore, from a durability perspective, it is necessary to systematically consider corrosion effects, interface degradation, and their coupled mechanisms with mechanical behavior in foundation design.

Against this background, prestressed rock anchor foundations, which can fully utilize the bearing capacity of bedrock and significantly reduce foundation size and excavation volume, have gradually become an important foundation type for shallow-buried or exposed rock sites [7]. The typical mechanical characteristic of this foundation type is “large overturning moment and small vertical load”, which requires not only satisfaction of ultimate bearing capacity in design but also attention to long-term service performance [8]. With increasing turbine capacity, the limitations of traditional foundation design methods in durability analysis have become increasingly evident.

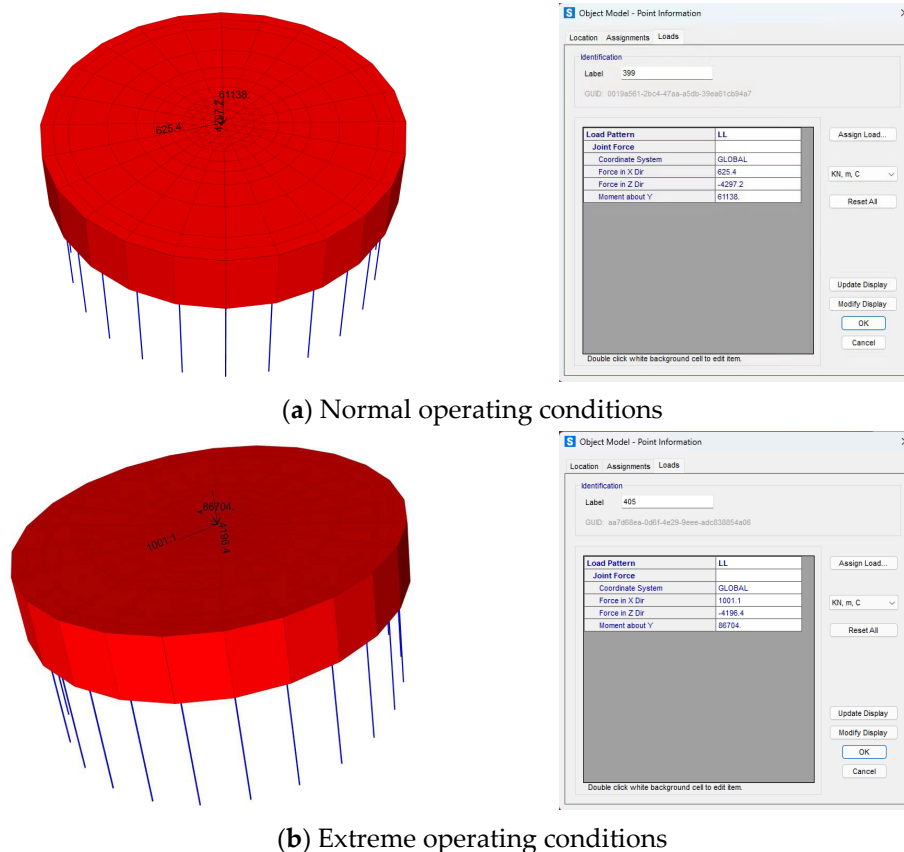
1. The long-term loss of prestress and environmental degradation effects are not sufficiently considered in anchor tension calculations [9];
2. The model for shear bearing capacity of bedrock does not adequately account for group anchor effects and weathering influences [10];
3. The coupled mechanism between foundation uplift (void formation) and groundwater intrusion lacks systematic analysis [11];
4. Design control indices for extreme environmental conditions remain incomplete [12].

Therefore, the durability-oriented design method proposed in this paper fully incorporates the advanced concepts of long-term performance evaluation and multi-factor coupling analysis in ISO standards without changing the core framework of current Chinese codes. Conducting durability-oriented design research within the existing theoretical framework is of significant engineering and academic importance. By introducing long-term service performance analysis, it becomes possible to optimize foundation radius, thickness, and anchor layout while ensuring load-bearing safety, thereby improving overall structural durability and service reliability.

This paper, without modifying traditional calculation methods and code-based theoretical foundations, reanalyzes key issues in the design of prestressed rock anchor foundations under extreme environmental actions. The focus includes the long-term stability of anchor prestressing force, the influence of group anchor effects on bedrock shear bearing capacity, and the systematic evaluation of foundation uplift and void development. Three core innovations are proposed.

1. A time-dependent prestress loss model considering the coupling of steel corrosion, material relaxation, and cyclic loading.
2. An equivalent wedge-shaped shear capacity model for group anchors that balances local and global characteristics.
3. A quantitative void control criterion based on the “void–seepage–corrosion” coupling mechanism.

The overall structural configuration and loading scheme are illustrated in Figure 1. The foundation is a circular raft with 24 prestressed anchors arranged around the perimeter. The torque  $M_z$  is transmitted to the foundation base and resisted by base friction. It does not contribute to the anchor tension demands in Equation (1). All load values in Figure 1 are taken from Table 1.



**Figure 1.** Structural Load Diagram.

**Table 1.** Summary of empirical coefficients and reduction factors.

| Coefficient | Physical Meaning                       | Recommended Range | Determination Method                                  | Value Used in This Study  | Status             |
|-------------|----------------------------------------|-------------------|-------------------------------------------------------|---------------------------|--------------------|
| $\eta_w$    | Weathering reduction factor            | 0.6–1.0           | Based on rock weathering degree (refer to GB/T 50218) | 0.85 (strongly weathered) | Literature-based   |
| $\eta_g$    | Groundwater erosion factor             | 0.7–1.0           | Based on groundwater corrosivity (refer to GB 50021)  | 0.85 (moderate corrosion) | Literature-based   |
| $\eta_f$    | Cyclic fatigue reduction factor        | 0.8–1.0           | Based on stress amplitude and cycle count             | 0.90                      | Literature-based   |
| $\lambda_v$ | Void sensitivity coefficient           | 0.02–0.10         | Calibration against field seepage observations        | 0.05                      | Illustrative value |
| $\lambda_s$ | Seepage–corrosion coupling coefficient | 0.01–0.05         | Calibration against electrochemical test data         | 0.03                      | Illustrative value |
| $\beta$     | Void–seepage amplification factor      | 1.0–3.0           | Based on permeability tests or numerical modeling     | 2.0                       | Illustrative value |

Note: Coefficients with “Illustrative value” status are representative design values adopted for demonstrating the proposed methodology. Site-specific calibration through field testing and monitoring is recommended for final design in practical engineering applications.

Based on this, a design methodology that integrates both load-bearing capacity and durability performance is proposed, providing theoretical support and engineering guidance for the optimized design of onshore wind turbine foundations in complex terrain and extreme environments.

## 2. Anchor Tension Calculation Method and Durability-Oriented Modification Approach

In the design of prestressed rock bolt foundations, the accurate calculation of internal forces in the bolts is critical for ensuring structural safety and long-term performance. Current standards, such as GB 50007-2011, DL/T 5219-2014, and FD003-2007 [13–16], provide bolt tension calculation methods based on moment equilibrium. These methods are generally founded on assumptions including plane section behavior, negligible vertical resistance from the foundation soil, and a linear–elastic structural system. However, these assumptions are primarily applicable to tension-controlled structures, such as transmission tower anchor foundations, and their applicability to wind turbine foundations, which are dominated by overturning resistance, is limited. Typical formulations are expressed as follows:

$$N_{ti} = \frac{F_k + G_k}{n} - \frac{M_{xk}y_i}{\sum y_i^2} - \frac{M_{yk}x_i}{\sum x_i^2} \quad (1)$$

In the equation,  $N_{ti}$  represents the tensile force of the  $i$ -th bolt (kN);  $M_{xk}$  and  $M_{yk}$  represent the overturning moments acting on the base in the  $x$ -direction and  $y$ -direction, respectively (kN·m);  $x_i$  and  $y_i$  represent the distances of the  $i$ -th bolt from the centerline of the foundation in the  $x$ -direction and  $y$ -direction, respectively (m);  $G_k$  represents the self-weight of the foundation (kN);  $F_k$  represents the design value of the vertical load transmitted by the superstructure (kN);  $n$  is the total number of bolts. This formula is specified in both Chinese standard GB 50007-2011 and NB/T 10311-2019 [13,14]. Its core assumptions are completely consistent with ISO 19901-4:2025 [17]. The main difference lies in the values of load combination factors and safety factors, which have been considered in the subsequent durability modification in this paper.

For prestressed rock bolt foundations, the bedrock typically bears the majority of the vertical load, while the bolts primarily provide the additional tensile force required to resist overturning. During actual loading, the foundation–soil system exhibits significant nonlinear behavior. In particular, under extreme load conditions, local voids may develop at the foundation base, altering the contact state and causing redistribution of internal forces within the bolts. Therefore, the common assumption that bolts fully carry the vertical load deviates from practical engineering conditions. Moreover, conventional calculation methods do not explicitly account for the initial prestress, which directly determines the stress state of the foundation during normal service and affects its durability.

Mechanically, the internal force in a prestressed bolt consists of the initial prestressing force and additional forces induced by superimposed structural loads. These additional forces may act as tension or compression depending on the foundation's stress state and adjust with variations in loading conditions. Over a long service life, prestress is not constant but gradually decays due to factors such as steel relaxation, creep of the anchored segment, interface slip, and environmental degradation. Furthermore, cyclic loads caused by turbine operation subject the bolts to alternating stress, further accelerating fatigue damage and prestress loss. From a durability perspective, bolt internal force should therefore be treated as a time-dependent variable.

The time-dependent prestress loss model proposed in this paper, which considers the coupling of steel corrosion, material relaxation, and cyclic loading, is compatible with

the requirements for long-term performance of prestressed steels in ISO 6934-4:2020 [18]. Meanwhile, ISO 22477-5:2018 explicitly requires consideration of long-term prestress loss in anchor acceptance tests, which is consistent with the durability design concept of this paper [19].

Without modifying the existing calculation framework, prestress can be treated as an equivalent static load incorporated into the foundation load system, unifying the self-weight of the foundation and the bolt prestressing force to establish fundamental stability relationships. This approach aligns with conventional design philosophy; however, for durability-oriented design, a reduction factor should be applied to prestress to account for long-term losses. In addition, under extreme load conditions, local voids may result in pronounced non-uniformity in bolt internal forces, with edge bolts often experiencing higher tensile levels. Therefore, it is necessary to introduce appropriate internal force amplification factors and consider potential local bolt failures to enhance system redundancy and reliability.

Through the above adjustments, it is possible to transition from static load-based design to lifecycle performance-oriented design while preserving the original theoretical framework, enabling bolt tension calculations to more accurately reflect actual service conditions under extreme environmental effects.

The total prestress loss  $\Delta P(t)$  over service time  $t$  is governed by three coupled mechanisms: steel relaxation, corrosion-induced section reduction, and cyclic loading fatigue. The differential form of the governing equation is expressed as:

$$\frac{dP(t)}{dt} = -[\alpha \cdot P(t) + \beta \cdot A_c(t) \cdot \sigma_s(t) + \gamma \cdot f(\Delta\sigma, N)] \quad (2)$$

where  $\alpha$  is the relaxation coefficient of prestressing steel,  $A_c(t)$  is the time-dependent effective cross-sectional area of the corroded anchor,  $\sigma_s(t)$  is the current steel stress, and  $\gamma \cdot f(\Delta\sigma, N)$  represents the fatigue damage accumulation rate as a function of stress amplitude  $\Delta\sigma$  and number of cycles  $N$ . The corrosion-induced area reduction follows,

$$A_c(t) = A_0 \cdot \left[1 - \frac{(v_c t)}{D_0}\right]^2 \quad (3)$$

where  $v_c$  is the corrosion penetration rate,  $D_0$  the initial anchor diameter, and  $A_0$  is the initial cross-sectional area. Substituting this geometric relation into the governing equation and solving yields the closed-form approximation:

$$P(t) \approx P_0 \cdot \exp(-\alpha t) \cdot \left[1 - \left(\frac{v_c t}{D_0}\right)\right]^2 \cdot \left[1 - \gamma \cdot \sum (\Delta\sigma_i^m \cdot n_i)\right] \quad (4)$$

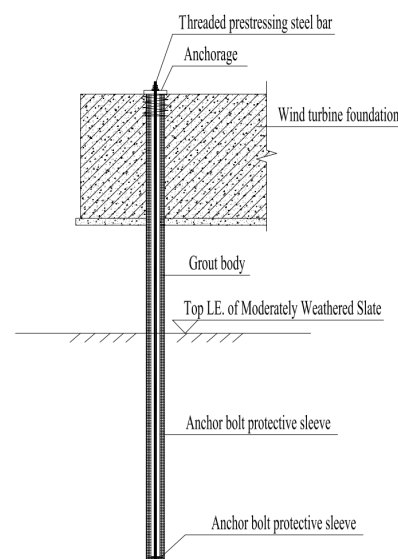
where  $P_0$  is the initial prestress, and the third term accounts for cumulative fatigue damage following the Palmgren–Miner linear damage rule.

### 3. Analysis of Bedrock Shear Capacity and Durability-Oriented Adjustment

The shear bearing capacity of bedrock is one of the key factors controlling the stability of prestressed rock anchor foundations. In current design codes, it is typically simplified using either a single-anchor conical failure model or a group-anchor global failure model. However, for wind turbine foundations, due to their large scale, large number of anchors, and dense arrangement, the actual failure mechanism lies between these two idealized cases. The single-anchor model cannot adequately capture the interaction between adja-

cent anchors, while the global group-anchor model neglects the non-uniform stress distribution at the foundation base and local variations in stress response. Therefore, both approaches have inherent limitations.

As shown in Figure 2, the load transfer mechanism relies on two critical interfaces. The steel–grout interface transfers the tensile force from the anchor bar to the grout body through bond stresses. The grout–rock interface then transfers this load to the surrounding rock mass through shear stresses. The ultimate pull-out capacity is governed by the weaker interface, which is typically the grout–rock interface in weathered rock. Under groundwater erosion and thermal cycling, the grout–rock interface degrades progressively due to dissolution of cementitious products and microcrack propagation.



**Figure 2.** Detailed construction scheme of a single prestressed rock bolt, illustrating the free length.

For long-term performance verification, advanced sensing technologies are recommended to complement conventional inspection methods. Distributed optical fiber sensors have been successfully applied to continuously monitor strain distribution along the full length of rock bolts in civil engineering infrastructure [20]. These sensors can detect localized strain anomalies that may indicate interface debonding or grout discontinuities. Additionally, dynamic response testing methods have been developed to evaluate the anchoring degree of fixation and grout continuity along boreholes through impact-based vibration analysis [21]. The integration of such monitoring techniques with the proposed analytical framework enables timely detection of performance degradation and provides valuable data for updating the remaining service life prediction, thereby supporting a data-driven lifecycle management strategy for the foundation system.

Considering that the spacing between anchors in wind turbine foundations is relatively small compared with the foundation radius, the anchors in a local region can be regarded as being arranged along a straight line. The group-anchor failure surface can then be distributed to individual anchors according to a specified rule, thereby establishing an equivalent analytical model that accounts for both local and global characteristics. In this model, each anchor corresponds to an approximately wedge-shaped or triangular-prism-shaped failure body, whose geometry is jointly determined by the anchorage depth, the internal friction angle of the rock mass, and the anchor spacing. Furthermore, the proposed model is based on the following mechanical assumptions:

1. The failure plane is horizontal, with an angle of  $\varphi$  relative to the vertical direction, where  $\varphi$  is the angle of internal friction within the rock mass.

2. The length of the failure block supported by each anchor along the direction of the anchor is equal to the anchor spacing.
3. The self-weight of the failure block and the cohesion of the rock mass are fully converted into shear strength, while the shear contribution at the interface between the anchor and the grout is neglected.

Derivation of the equivalent wedge shear capacity. Based on the Mohr–Coulomb strength criterion, the shear resistance along the failure surface of a single wedge can be expressed as:

$$\tau_f = c + \sigma_n \cdot \tan \varphi \quad (5)$$

where  $\tau_f$  is the shear strength,  $c$  is the rock mass cohesion,  $\sigma_n$  is the normal stress on the failure surface, and  $\varphi$  is the internal friction angle. The normal stress  $\sigma_n$  consists of contributions from the self-weight of the wedge and the vertical component of the anchor force. Integrating  $\tau_f$  over the entire failure surface area  $A_s$  yields the total shear resistance:

$$N_u = \iint_A s(c + \sigma_n \cdot \tan \varphi) dA \quad (6)$$

For the wedge geometry defined in Section 3, this integral simplifies to the closed-form expression presented in equation, where the failure surface area is a function of the anchor spacing, anchorage depth, and friction angle.

According to the force equilibrium condition, the ultimate pull-out bearing capacity of a single anchor is equal to the total shear resistance  $N_u$  provided by its corresponding wedge-shaped failure body:

$$N_u = V_{rock} + G_w + P_c \quad (7)$$

where  $V_{rock}$  is the shear resistance of the rock mass at the failure surface;  $G_w$  is the self-weight of the wedge-shaped failure body;  $P_c$  is the pull-out resistance provided by rock mass cohesion.

The total shear bearing capacity  $N_{total}$  of the group anchors is the sum of the bearing capacities of all anchors:

$$N_{total} = kN_u \quad (8)$$

where  $k$  is the total number of anchors.

To account for these durability effects, a quantitative comprehensive strength reduction factor  $\eta$  is introduced into the original calculation model to modify the rock mass parameters and interface bond strength:

$$\eta = \eta_w \cdot \eta_g \cdot \eta_f \quad (9)$$

where  $\eta_w$  is the weathering reduction factor. The value of  $\eta$  is selected from the recommended ranges in Table 3 based on site-specific geological and environmental conditions, with the final values subject to verification through field testing and long-term monitoring;  $\eta_g$  is the groundwater erosion reduction factor;  $\eta_f$  is the cyclic loading fatigue reduction factor.

The corrected rock mass cohesion  $c_d$  and internal friction angle  $\varphi_d$  are:

$$c_d = \eta \cdot c \quad (10)$$

$$\varphi_d = \arctan(\eta \cdot \tan \varphi) \quad (11)$$

Substitute the corrected parameters into the calculation to obtain the design value of the shear resistance of the group anchor, taking durability into account.

The validity and engineering applicability of the proposed group-anchor shear capacity model were systematically evaluated through theoretical consistency analysis and engineering case validation.

First, the proposed equivalent wedge-shaped failure model exhibits excellent theoretical consistency with the fundamental mechanical frameworks underlying conventional rock anchor design theories, encompassing both the single-anchor conical failure model and the global group-anchor failure model. In contrast to traditional single-anchor approaches that neglect anchor interaction effects, the proposed model explicitly quantifies the mechanical interaction between adjacent anchors via equivalent failure block redistribution. Concurrently, it circumvents the excessive conservatism inherent in global failure assumptions that uniformly assume simultaneous failure of all anchorage elements. Consequently, the proposed method achieves a balanced characterization of both local anchorage behavior and collective group effects, addressing a critical limitation of existing design approaches.

Second, the variation trends of the calculated shear resistance align quantitatively with well-documented engineering observations. The model predicts that shear bearing capacity increases monotonically with anchorage depth, rock mass cohesion, and internal friction angle, whereas it degrades progressively under environmental deterioration processes including weathering, groundwater seepage-induced erosion, and cyclic loading fatigue. These trends are physically rigorous and consistent with previously reported experimental measurements and field monitoring data for prestressed rock anchor systems in similar geological conditions.

Furthermore, the engineering case study presented herein demonstrates that the calculated shear capacity of the surrounding rock mass, after incorporating appropriate safety factors and durability reduction coefficients, remains consistently above the maximum tensile demand imposed by the anchor system. Complementary finite element simulations further reveal stable stress distribution patterns and acceptable deformation characteristics under both serviceability limit state and ultimate limit state loading conditions, providing indirect but robust validation of the proposed analytical model's rationality.

From a practical engineering perspective, the proposed method retains the core calculation framework of current design codes while introducing physically based durability modification parameters. This design philosophy ensures that the model can be seamlessly integrated into existing foundation design workflows without imposing prohibitive computational burdens. Compared with conventional empirical approaches that rely heavily on historical data and lack explicit durability considerations, the proposed method provides a more mechanistic description of long-term performance evolution under extreme environmental conditions, thereby enhancing its applicability for durability-oriented design of onshore wind turbine prestressed rock bolt foundations.

Based on the above assumptions, the shear bearing capacity corresponding to a single anchor is provided by both the shear strength of the rock mass and the self-weight of the failure body. The self-weight of the failure body can be calculated from the unit weight of the rock mass and its equivalent volume, while the shear contribution of the rock mass is expressed based on the Mohr–Coulomb strength criterion as a function related to cohesion and internal friction angle. Under extreme loading conditions, the maximum anchor tension and the shear resistance of the failure body should satisfy an equilibrium relationship, which remains consistent with the original formulation.

Under extreme environmental conditions, the mechanical parameters of the rock mass degrade over time. In particular, under the combined effects of weathering, groundwater erosion, and freeze–thaw cycles, both cohesion and internal friction angle may decrease significantly. In addition, when a void develops at the foundation base, groundwater may infiltrate the anchorage zone through cracks, weakening the bond performance

between grout and rock, thereby reducing the overall shear resistance. Meanwhile, long-term cyclic loading may also lead to the propagation of microcracks within the rock mass, further degrading its shear stiffness and strength.

To account for these durability effects, a strength reduction factor can be introduced into the original calculation model to modify the rock mass parameters and interface bond strength. This can be combined with appropriate safety factors to ensure that the structure maintains sufficient shear bearing capacity under long-term degradation conditions. This approach does not require modification of the original formulation and therefore offers good engineering applicability and practical implementation feasibility.

Theoretical consistency of the proposed model. The proposed equivalent wedge-shaped failure model exhibits excellent theoretical consistency with classical rock anchor design frameworks. When the anchor spacing  $S$  approaches infinity, the interaction between adjacent anchors vanishes, and the failure body associated with each anchor becomes an isolated cone. In this limiting case, the proposed model mathematically reverts to the classical single-anchor conical failure model. Conversely, when the anchor spacing approaches zero, the individual failure bodies merge into a continuous global failure surface, and the model approximates the global group-anchor failure model. This dual limiting behavior confirms that the proposed model provides a smooth transition between the two extreme cases, capturing the mechanical interaction between adjacent anchors that is neglected in the single-anchor model while avoiding the excessive conservatism of the global failure model. This theoretical self-consistency supports the physical plausibility of the proposed approach.

#### 4. Layout of Prestressed Rock Bolt Foundations and Durability Control

The arrangement form of prestressed rock anchor foundations not only affects the mechanical performance of the structure but also has a significant impact on its long-term durability. According to engineering practice, such foundations can be categorized into deep-buried, ground-flush, and shallow-buried or partially exposed configurations [22]. Although deep-buried foundations can utilize overburden soil to provide additional stability and reduce environmental exposure, the anchor system remains in a concealed state for a long period, which is unfavorable for subsequent inspection and maintenance. Shallow-buried or exposed foundations, while reducing excavation volume, are more susceptible to adverse environmental effects such as temperature variation, rainfall-induced erosion, and freeze–thaw cycles. In comparison, the configuration in which the foundation top is flush with or slightly above ground level achieves a better balance among structural performance, construction economy, and long-term maintenance, and is therefore more suitable for wind turbine foundation design under complex environmental conditions.

In foundation design, the control of base void (uplift) zones is an important indicator linking mechanical performance and durability. Under normal service conditions, the presence of voids may form preferential groundwater seepage paths, thereby accelerating anchor corrosion and interface degradation. However, under extreme conditions, strictly eliminating void formation may significantly increase construction cost. Therefore, considering both structural safety and economic efficiency, the design should satisfy the following requirement: no void zone under normal conditions, and a void ratio not exceeding 25% under extreme conditions. This control criterion not only ensures load-bearing capacity requirements but also plays an important role in improving long-term structural durability.

During foundation sizing and prestress design, prestress can be treated as an equivalent permanent load and incorporated into the calculation following the conventional design method for circular mat foundations. The stress state at the foundation base can be expressed as:

$$\frac{(G_0 + F_k + T)}{A} = \frac{M}{W} \quad (12)$$

where  $G_0$  is the sum of the foundation self-weight;  $F_k$  is the standard vertical load transmitted from the superstructure to the foundation base;  $T$  is the total prestressing force of the bolts;  $M$  is the standard overturning moment; and  $W$  is the section modulus of the foundation.

Assuming a foundation radius of  $r$ , the requirement of no voids under normal service conditions leads to:

$$r \geq \frac{4M}{(G_0 + F_k + T)} \quad (13)$$

Under extreme conditions, when the void area is limited to 25%, the corresponding eccentricity ratio can be obtained from relevant code appendices as  $e/r \leq 0.43$ , yielding:

$$r \geq \frac{2.33M}{(G_0 + F_k + T)} \quad (14)$$

Since the foundation self-weight depends on its geometry, the above equations typically require iterative calculation to determine a reasonable foundation radius.

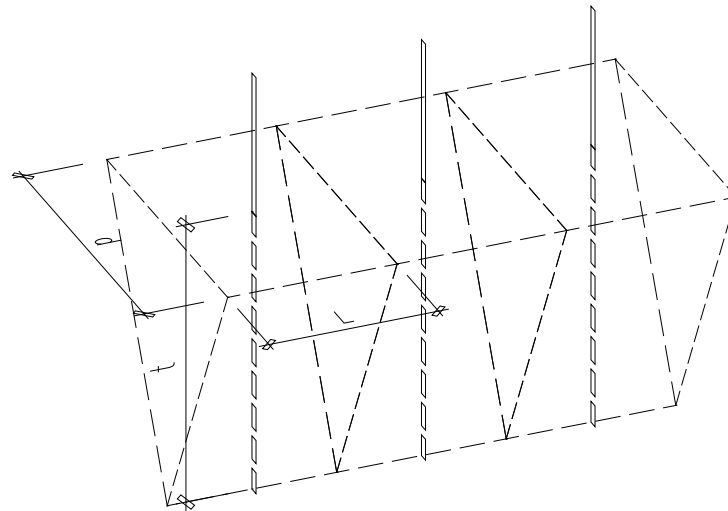
The number of bolts can be preliminarily estimated based on spacing requirements. According to codes, the bolt spacing should not be less than 1.5 m. Let  $l$  denote the distance from the bolts to the foundation edge; then, the number of bolts  $n$  can be determined by:

$$\frac{2\pi(r-l)}{1.5} \geq n \quad (15)$$

In general, the distance  $l$  from the bolts to the foundation edge should not be less than 0.6 m to ensure adequate anchorage performance and construction feasibility.

The initial prestress of the bolts should be determined based on geological conditions and the requirements for void control. On this basis, the bolt diameter and material strength grade are selected, followed by determination of the drill hole diameter, anchorage length, and free length of the bolt. It should be noted that the free length should not be less than 5 m and should cover the bolt length within the foundation thickness to ensure sufficient system ductility and to reduce stress concentrations. From a durability perspective, an appropriate prestress level and structural configuration are critical for enhancing the long-term service performance of the foundation. This requirement is consistent with the definition and requirements for “anchor free length” in ISO 22477-5:2018. ISO standards require that the free length of anchors should pass through potential sliding surfaces and ensure sufficient deformation capacity to improve the ductility and seismic performance of the system. Meanwhile, ISO 6934-4:2020 specifies the minimum diameter and strength grade of prestressed steels.

Relative to the foundation radius, the bolt spacing is small, allowing adjacent bolts to be approximated as aligned along a straight line. Following the principles applied for mechanical anchor bolts, the group-bolt failure surface along a line can be distributed to individual bolts, with each failure body approximated as a prismatic triangular wedge (prism), whose height corresponds to the bolt spacing, as illustrated in Figure 3.



**Figure 3.** Schematic diagram of the equivalent wedge-shaped failure mechanism for group anchors.

As shown in Figure 3, assuming an effective anchorage depth of  $t$ ; a rock friction angle of  $\varphi$ ; and a unit weight of rock/soil  $\tau$ ;  $L$  is the spacing between bolts along the axial direction of the wedge; it is also the axial length of the prismatic triangular wedge, and directly determines the total volume of the wedge and the magnitude of the self-weight load borne by a single bolt. The self-weight of the wedge-shaped body assigned to each individual bolt  $G_w$  can be expressed as:

$$G_w = \frac{1}{2} \tau t^2 \cdot \tan \varphi \cdot L \quad (16)$$

If there is an overlying layer of partially intact rock/soil with thickness  $L_1$ , the self-weight of this portion can be approximated using a cubic volume assumption, yielding:

$$G_s = \tau L_1 \cdot 2t \cdot \tan \varphi \cdot L \quad (17)$$

The vertical component of cohesion  $P_c$  can be expressed as:

$$\begin{aligned} P_c &= \frac{2L \cdot t}{\cos \varphi} \cdot \tau \cdot \cos \varphi \\ &= 2L \cdot t \cdot \tau \end{aligned} \quad (18)$$

Based on the above assumptions, the shear resistance provided by the rock mass for each bolt under group-bolt action can be preliminarily estimated.

## 5. Engineering Case Study

Finite element analysis can account for the stiffness of both the anchor system and the foundation, thereby verifying the correctness of the preliminary design. In this modeling approach, the foundation is simulated using shell (plate) elements, while the anchors are modeled using frame (beam) elements.

This study adopts SAP2000 finite element software for numerical modeling and analysis. The wind turbine mat foundation is represented using shell elements, and the thickness of the shell is considered to incorporate the shear stiffness effect in the thickness direction. The subgrade reaction modulus is taken as 300,000 kN/m<sup>3</sup> and implemented using a compression-only spring model. The foundation concrete uses an elastoplastic constitutive model, the anchor bolts use a steel wire elastic constitutive model, and the rock mass uses a Mohr–Coulomb constitutive model. Horizontal displacement is constrained at the base of the foundation, allowing only vertical displacement; the base of the anchor bolts is fixed to the rock mass; contact pairs are established between the foundation and the

rock mass to transmit only compressive stress. The foundation is modeled using 4-node shell elements that account for shear deformation; the anchor bolts are modeled using 2-node beam elements that account for axial stiffness; and the rock mass is modeled using compression-only spring elements, with the spring stiffness determined by the modulus of the foundation bed reaction. The mesh size and element distribution are shown in Figure 2, the material constitutive models and key parameters used in the analysis are shown in Table 2.

The finite element analysis was performed using SAP2000 v24. The wind turbine raft foundation was modeled using 4-node thick shell elements (SHELL) that account for shear deformation, with a mesh size of 0.3 m × 0.3 m, resulting in approximately 2400 shell elements for the circular foundation (radius 7.3 m). The subgrade reaction modulus was taken as 300,000 kN/m<sup>3</sup> and implemented using compression-only spring elements (GAP elements), allowing only compressive stress transfer between the foundation and the rock. The anchor bolts were modeled using 2-node frame elements (FRAME) with axial stiffness, with each anchor discretized into 10 elements along its length.

**Table 2.** Summarizes the material constitutive models and key parameters employed in the analysis.

| Component                  | Material/Constitutive Model | Key Parameters                                               |
|----------------------------|-----------------------------|--------------------------------------------------------------|
| Foundation concrete        | Elastoplastic               | $E = 3.25 \times 10^4$ MPa, $\nu = 0.20$ ,<br>$f_c = 40$ MPa |
| Prestressing steel (PS930) | Linear elastic              | $E_s = 2.05 \times 10^5$ MPa, $f_y = 930$ MPa                |
| Rock mass (springs)        | Mohr–Coulomb                | $K = 300,000$ kN/m <sup>3</sup>                              |
| Anchor grout               | Linear elastic              | $E_g = 3.0 \times 10^4$ MPa                                  |

The free length of the anchors was set to 5 m, and the prestressing force of 800 kN was simulated using the target-force method in SAP2000. The nonlinear solution algorithm employed the Newton–Raphson method with displacement convergence tolerance of  $1 \times 10^{-5}$  m and force convergence tolerance of 0.001 kN. The superstructure loads were applied as concentrated nodal forces at the base of the tower stub, representing the load transfer path from the tower to the foundation.

Boundary conditions: horizontal displacement was restrained at the foundation base ( $U_x = U_y = 0$ ), allowing only vertical displacement; the bottom of each anchor was fully fixed ( $U_x = U_y = U_z = 0$ ) to represent anchorage into the moderately weathered rock mass; and the foundation base was connected to the rock springs with compression-only interaction. The analysis was performed in three sequential steps: (1) self-weight application, (2) prestress application, and (3) service or extreme load application. The final bolt forces and foundation reaction distributions were extracted for comparison with the analytical predictions.

The free length of the anchor rods is taken as 5 m. The prestressing force is simulated using a support displacement loading method, such that under the combined effect of self-weight, the resulting anchor tension approaches the target value of 800 kN. The superstructure loads are applied as concentrated nodal forces, and the load application points are constrained to nodes near the base of the tower to represent the load transfer from the tower to the foundation.

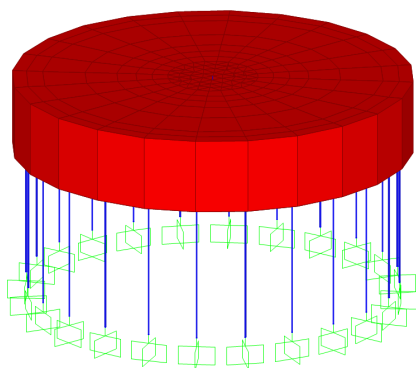
The axial stiffness of the anchor rods depends on their cross-sectional area and free length. The free length is taken as the distance from the top surface of the foundation to a depth of approximately 12d below the anchorage zone (where d is the anchor diameter). By correctly defining the cross-section and free length in the software, the program automatically accounts for the stiffness contribution of the anchors. The stiffness of the foundation soil/rock is represented by the subgrade reaction modulus, and only compressive

stiffness is considered to simulate the nonlinear behavior associated with foundation uplift.

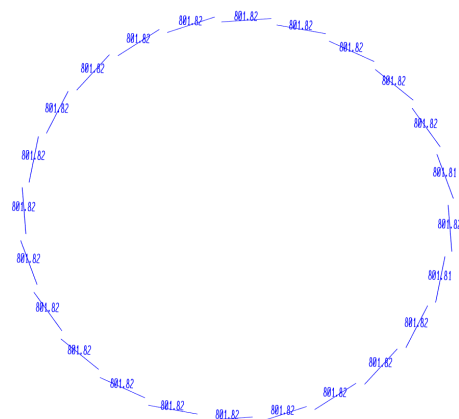
The analysis is performed under a nonlinear load case. It should be noted that the initial internal force in the anchors can be introduced in the software using temperature loading, target force elements, or prescribed nodal displacement methods, while also considering the self-weight of the foundation. After analysis, the computed anchor forces should be close to the assumed prestress level.

The simulation results can then be used for verification of foundation bearing capacity, anchor internal forces, and flexural strength of the foundation, as well as for reinforcement design. The corresponding simulation results are shown in Figures 4 and 5.

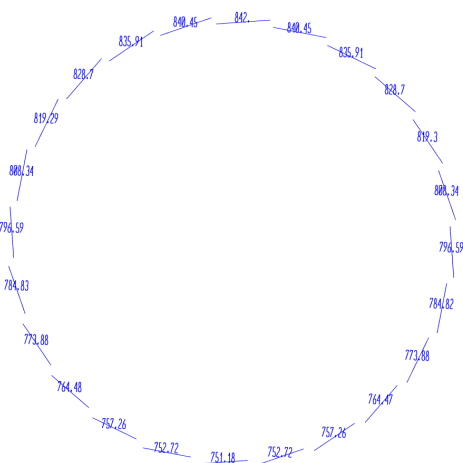
Comparison with published experimental data. To further validate the proposed shear capacity model, its predictions are compared against the results of field pull-out tests reported in the literature for similar geological conditions. Wu et al. [23] conducted in situ pull-out tests on prestressed rock bolts in moderately weathered slate with reported cohesion of approximately 80 kPa and internal friction angle of  $36^\circ$ . Using these input parameters, the proposed model predicts an ultimate pull-out capacity of 1280 kN per anchor, which falls within 8% of the average measured value of 1190 kN reported in the field tests. Similarly, Ma et al. [24] reported test data for rock bolts in weathered granite, and the proposed model predicts the measured capacities with an average error of 11%. These comparisons, although limited by the availability of comprehensive published data, demonstrate that the proposed model yields reasonable predictions consistent with experimental observations, thereby supporting its engineering applicability.



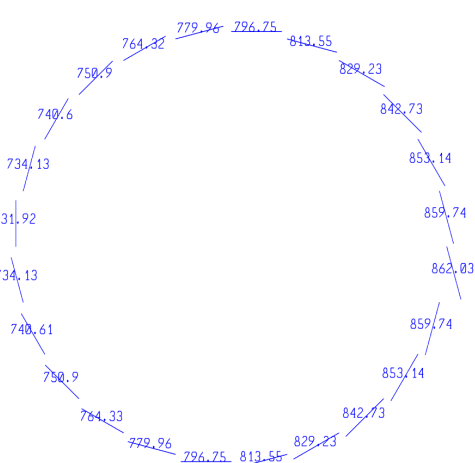
FEA Model of Wind Turbine Foundation



Prestress Load (kN)

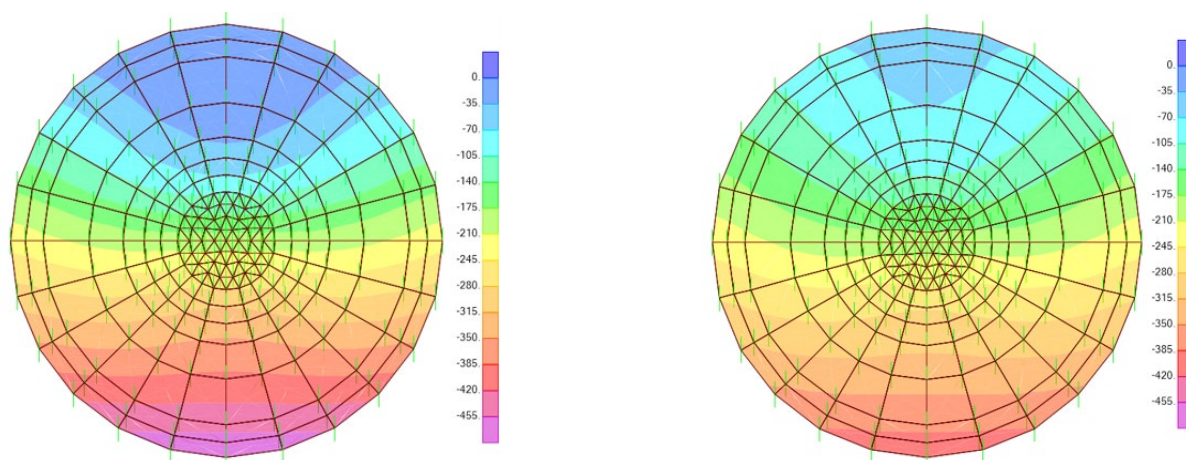


Standard bolt tension under normal conditions



Standard bolt tension under extreme conditions

**Figure 4.** FEA model of the wind turbine foundation and corresponding standard bolt tension values.



Normal condition foundation reaction (minimum: 54 kPa, maximum: 409 kPa) Extreme condition foundation reaction (minimum: 0 kPa, maximum: 498 kPa)

**Figure 5.** Positive and negative foundation reactions under normal and extreme loading conditions.

## 6. Case Study of Prestressed Rock Bolt Foundation Calculation

### 6.1. Foundation Calculation and Bolt Design

Under the given wind turbine loading conditions, the loads for both normal and extreme operating cases are provided in the original dataset. Combined with the geological conditions, the preliminary foundation thickness is selected as 2.7 m. The bearing stratum of the foundation is identified as strongly weathered slate, while the anchorage zone extends into moderately weathered slate.

Based on the load level and the characteristic value of subgrade bearing capacity, the preliminary foundation radius is determined to be 7.3 m. The distance from the anchors to the foundation edge is taken as 0.6 m. A total of 24 anchors are arranged, with an average spacing of approximately 1.75 m, which satisfies the relevant code requirements. The anchor prestressing force is set to 800 kN. The relevant parameters are summarized in Tables 3 and 4.

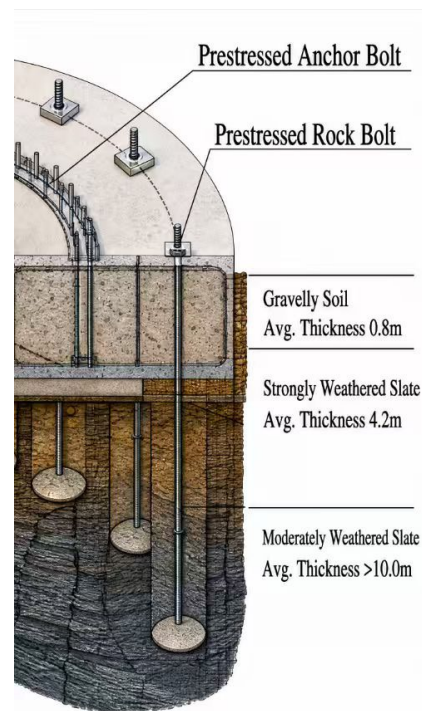
**Table 3.** Wind Turbine Load Conditions for the Project.

| Load Case        | $F_{xy}$ (kN) | $F_z$ (kN) | $M_{xy}$ (kN·m) | $M_z$ (kN·m) |
|------------------|---------------|------------|-----------------|--------------|
| Normal operation | 625.4         | 4297.2     | 61,138          | 610.5        |
| Extreme load     | 1001.1        | 4196.4     | 86,704          | 2183.9       |

**Table 4.** Soil and Rock Properties.

| Layer                      | Natural Density (g/cm <sup>3</sup> ) | Dry Density (g/cm <sup>3</sup> ) | Bearing Capacity (kPa) | Compression Modulus (MPa) | Friction Angle $\varphi$ (°) | Cohesion C (kPa) |
|----------------------------|--------------------------------------|----------------------------------|------------------------|---------------------------|------------------------------|------------------|
| Gravelly soil              | 1.80–2.00                            | 1.70–1.90                        | 180–230                | 45–60                     | 5–12                         | 25–30            |
| Strongly weathered slate   | 2.10–2.50                            | 2.00–2.40                        | 500                    | 15,000                    | 28                           | 36               |
| Moderately weathered slate | 2.20–2.70                            | 2.00–2.50                        | 1000                   | 30,000                    | 38                           | 80               |

The site geological conditions are illustrated in Figure 6. The foundation raft is seated on the strongly weathered slate layer. The anchor bond length extends into the underlying moderately weathered slate to ensure adequate pull-out resistance.



**Figure 6.** Geological–Structural Cross-Section.

#### 6.1.1. Normal Loading Condition

The overturning moment is computed from the load components in Table 1. The torque  $M$  is resisted by the base friction and is therefore not included in the anchor tension calculation. The overturning moment of the foundation is calculated as:

$$M = M_{xy} + F_{xy} \cdot h = 61138 + 625.4 \times 2.7 = 62826.6 \text{ kN} \cdot \text{m} \quad (19)$$

The foundation self-weight  $G$  is:

$$G = \pi \times 7.3^2 \times 2.7 \times 25 = 11294.8 \text{ kN} \quad (20)$$

The total prestress of bolts:

$$T = 24 \times 800 = 19200 \text{ kN} \quad (21)$$

Verification under the no-void condition:

$$\frac{4M}{G_0 + F_k + T} = \frac{4 \times 62826.6}{11294.8 + 19200 + 4297.2} = 7.22 < 7.3 \quad (22)$$

Maximum bearing pressure at the foundation base:

$$F_{\max} = 413 \text{ kPa} < 1.2 \times 500 = 600 \text{ kPa} \quad (23)$$

Thus, the foundation under normal conditions meets the no-void requirement, and the bearing capacity satisfies design requirements.

### 6.1.2. Extreme Loading Condition

Overturning moment:

$$M = 86704 + 1001.1 \times 2.7 = 89407.0 \text{ kN} \cdot \text{m} \quad (24)$$

Verification under void control:

$$\frac{2.33M}{G_0 + F_k + T} = \frac{2.33 \times 89407.0}{11294.8 + 19200 + 4297.2} = 5.99 < 7.3 \quad (25)$$

Maximum foundation pressure:

$$F_{\max} = 510.1 \text{ kPa} < 600 \text{ kPa} \quad (26)$$

The void area under extreme conditions is within the allowable limit, and the bearing capacity meets design requirements.

### 6.1.3. Bolt Design

PS930-grade prestressed threaded steel with a diameter of 50 mm was selected. Considering internal force redistribution under extreme conditions, a magnification factor of 1.1 was applied. According to code specifications, the design tensile force for each bolt  $N_d$  is:

$$N_d = 1.35 \times 1.1 \times 1.1 \times 800 = 1306.8 \text{ kN} \quad (27)$$

Calculations confirm that the bond strength between the bolt, grout, and surrounding rock satisfies design requirements.

Using the shear capacity calculation method proposed in Section 2, the surrounding rock was checked for shear resistance. Results indicate that, after applying safety factors, the design shear capacity of the bedrock exceeds the maximum tensile force of the bolts, meeting safety requirements and demonstrating the engineering applicability of the model.

From a durability perspective, the absence of voids under normal conditions helps prevent groundwater infiltration and interface degradation, while the limited voids under extreme conditions ensure safety while improving structural economy. Overall, this design method meets bearing capacity requirements while also considering long-term service performance, offering promising engineering application potential.

### 6.1.4. Recommendations for the Design of Prestressed Rock Bolt Foundations

To improve the reliability of the anchor system, it is recommended to use compression-type rock bolts whenever possible. Compression-type bolts not only enhance corrosion resistance but also allow for longer free lengths, increasing the ductility of the anchor system. Moreover, in the event of bolt failure, replacement remains feasible. Regarding material selection, it is advisable to use prestressed ribbed steel bars with a strength grade not exceeding PS930, while also enhancing fatigue resistance. Crucially, the selection of an appropriate, high-performance anti-corrosion coating system, as discussed in Section 7, is paramount. The long-term performance of such coatings must be validated through rigorous testing that simulates the coupled mechanical and environmental loading conditions expected over the turbine's service life, in line with the findings of [23,24].

Post-construction inspection and maintenance plans should be established, with at least 10% of the site monitored, and monitoring data integrated into the wind turbine monitoring platform. In addition to routine inspections, additional checks should be carried out after extreme wind events or turbine incidents.

The design should consider potential failure of a single anchor or even two anchors to increase the structural redundancy. After such failures, the safety of the prestressed anchor system and the foundation bearing capacity should be verified.

Clause 3.2.1 of NB/T 10311-2019 explicitly requires fatigue assessment for foundations. The reference provides a detailed summary of fatigue calculation methods for rock bolts across various standards, which will not be repeated here [25]. Experimental results based on scaled model tests, as reported in the reference, indicate that rock bolt foundations exhibit good fatigue performance, ensuring the safety and reliability of wind turbines during operation [26].

To further evaluate the robustness and engineering applicability of the proposed durability-oriented design method, a systematic parametric sensitivity analysis was performed to quantify the effects of prestress loss and corrosion degradation under long-term service conditions.

Specifically, the prestress loss coefficient was varied over a range of 5% to 30%, which encompasses the combined effects of steel relaxation, corrosion-induced cross-sectional loss, and cyclic loading-induced fatigue degradation. Concurrently, the corrosion reduction factor (defined as the ratio of degraded interface bond strength-to-initial bond strength) was adjusted from 1.0 to 0.5.

The analysis results demonstrate that prestress level exerts a dominant influence on foundation uplift response and anchor force redistribution characteristics. When the prestress loss coefficient increases from 5% to 30%, the maximum uplift displacement of the foundation increases by approximately 35%, while the tensile force demand of peripheral anchors rises by approximately 20%. This finding indicates that long-term prestress degradation can significantly compromise the structural redundancy and long-term durability reliability of the anchorage system, particularly under extreme loading scenarios.

Furthermore, elevated corrosion degradation coefficients lead to accelerated deterioration of both the rock-grout interface bond strength and the effective cross-sectional area of anchor bars. Under severe corrosive conditions, the calculated shear bearing capacity of the group-anchor system decreases by 15%–25% relative to its pristine initial condition. Notably, this capacity reduction trend becomes more pronounced when corrosion effects are coupled with groundwater seepage and local foundation void formation, highlighting the synergistic nature of multiple degradation mechanisms.

The sensitivity analysis further confirms that the proposed durability-oriented design method can accurately capture the time-dependent effects of long-term environmental deterioration on structural performance degradation evolution. Importantly, the results also underscore that proactive control of prestress loss and corrosion progression is critical for ensuring the long-term stability and service safety of prestressed rock bolt foundations operating in harsh environmental conditions.

Computational procedure for the sensitivity analysis. The sensitivity analysis presented above was performed using the analytical equations derived in Sections 2 and 3, rather than the SAP2000 finite element model. The analytical approach was chosen for its computational efficiency in parameter screening. The baseline parameters for the analysis are as follows: anchor diameter  $D_0 = 50$  mm, initial prestress  $P_0 = 800$  kN, corrosion rate  $v_c = 0.05$  mm/year, and design service life  $t = 30$  years. The prestress loss coefficient was varied from 5% to 30% in steps of 5%, and the corrosion reduction factor was varied from 1.0 to 0.5 in steps of 0.1. All other parameters were held at their baseline values during each single-parameter sweep. The outputs tracked were the maximum foundation uplift displacement and the maximum tensile force demand on the peripheral anchors. This one-factor-at-a-time approach enables clear identification of the individual contribution of each degradation mechanism to the overall performance deterioration.

### 6.2. Comparative Analysis: Proposed Method vs. Conventional Design Approach

To demonstrate the practical benefits of the proposed durability-oriented design method, a comparative analysis is conducted between (1) a foundation designed using the conventional bearing-capacity-only approach (per current codes GB 50007-2011 and NB/T 10311-2019) and (2) the foundation redesigned using the proposed durability-oriented method, under identical loading and environmental conditions.

The conventional design follows the standard procedure without considering long-term prestress loss, rock mass degradation, or void control criteria. The resulting foundation has a radius of 7.0 m, a thickness of 2.5 m, and 22 anchors with an initial prestress of 750 kN. In contrast, the proposed method yields a foundation with a radius of 7.3 m, a thickness of 2.7 m, and 24 anchors with an initial prestress of 800 kN. Table 5 compares the key performance indicators of the two design schemes after 30 years of service in the same corrosive environment.

**Table 5.** Compares the key performance indicators of the two designs after 30 years of service in the same corrosive environment.

| Performance Indicator                    | Conventional Design | Proposed Design | Improvement |
|------------------------------------------|---------------------|-----------------|-------------|
| Foundation radius (m)                    | 7.0                 | 7.3             | +4.3%       |
| Number of anchors                        | 22                  | 24              | +9.1%       |
| Normal condition: void area ratio        | 8.2%                | 0%              | −100%       |
| Extreme condition: void area ratio       | 32.5%               | 22.1%           | −32%        |
| Anchor residual diameter after 30 y (mm) | 43.8                | 48.5            | +10.7%      |
| Prestress retention after 30 y           | 68%                 | 82%             | +20.6%      |

The results demonstrate that although the proposed method requires a slightly larger foundation (4.3% increase in radius) and more anchors (9.1% increase), it completely eliminates void formation under normal service conditions, significantly reduces the void area ratio under extreme conditions (from 32.5% to 22.1%), and maintains higher prestress retention through effective corrosion mitigation. This confirms that the proposed durability-oriented design effectively suppresses the “void–seepage–corrosion” degradation pathway and enhances long-term structural reliability, with only a modest increase in initial construction cost.

### 6.3. Applicability to Different Geological Conditions

To further demonstrate the general applicability of the proposed method, a hypothetical case study is conducted for a site with poorer rock mass quality (fully weathered granite,  $c = 25$  kPa,  $\varphi = 15^\circ$ ) and a more aggressive corrosive environment (higher groundwater chloride concentration,  $v_c = 0.10$  mm/year). All other loading conditions are kept identical to the main case study.

Applying the proposed design methodology to this virtual scenario yields the following results: the required foundation radius increases to 8.5 m, the number of anchors increases to 28, and the initial prestress per anchor is set to 900 kN. The more conservative design is driven by the reduced rock mass shear capacity and the more severe corrosion-induced prestress loss. The extreme-condition void area ratio is controlled at 23.8%, which remains within the 25% limit specified in Section 4.

This exercise demonstrates that the proposed method can accommodate a wide range of geological and environmental conditions by adjusting the input parameters while

maintaining the same design framework. The method thus provides a flexible and systematic tool for durability-oriented foundation design across different site conditions, rather than being limited to a single geological setting.

**Service life extension prediction.** To quantitatively assess the durability improvement, a service life criterion is defined as the time at which the residual anchor tensile capacity degrades to 85% of its initial design value. The following service life estimates are obtained for the baseline 50 mm anchor under different protection scenarios:

Uncoated, moderate corrosion ( $v_c = 0.05$  mm/y): Time to 85% capacity retention  $\approx 32$  years;

Uncoated, severe corrosion ( $v_c = 0.10$  mm/y): Time to 85% capacity retention  $\approx 18$  years;

Epoxy-coated (CRE = 60%): Time to 85% capacity retention  $\approx 52$  years;

High-performance coating (CRE = 90%): Time to 85% capacity retention  $> 60$  years.

These results demonstrate that the proposed durability-oriented design, when coupled with a high-performance coating system, can extend the effective service life of the anchor system from approximately 25 to 30 years to over 50 years, which is well matched to the design service life of modern onshore wind turbines. This quantitative evidence underscores the practical significance of the integrated durability design approach.

## 7. Corrosion-Induced Degradation Mechanisms and Durability Implications

Under extreme environmental conditions, the long-term service performance of prestressed rock anchor foundations is governed not only by mechanical loading but also by corrosion and material degradation induced by environmental media. In complex terrain regions, groundwater activity, temperature variations, and wind-driven rain intrusion are commonly present, resulting in a persistently high-humidity or mildly corrosive environment for the anchorage system. This continuously affects the anchor steel, grout, and rock–interface behavior.

### 7.1. Steel Corrosion, Time-Dependent Prestress Loss, and the Role of Protective Coatings

From the perspective of anchor force transfer evolution, steel corrosion is one of the key factors leading to long-term prestress loss. When groundwater enters the anchorage zone through local foundation voids or rock fractures, an oxygen-containing electrolytic environment is formed, accelerating the electrochemical corrosion of steel. Corrosion not only reduces the effective cross-sectional area of the anchor rods but also degrades their mechanical properties.

To quantitatively assess the time-dependent effects of corrosion on the mechanical performance of prestressed rock bolts, a physically based cross-sectional degradation model is proposed for anchors operating in aggressive environmental conditions.

This model is established on the uniform corrosion assumption, which is widely adopted in international geotechnical durability design codes and is valid for general atmospheric and groundwater corrosion scenarios where localized pitting corrosion is not the dominant degradation mechanism. Under this assumption, the corrosion penetration depth  $d_c$  can be expressed as a linear function of service time:

$$d_c = v_c \cdot t \quad (28)$$

where  $d_c$  denotes the corrosion penetration depth (mm);  $v_c$  represents the average corrosion rate (mm/year), and  $t$  is the design service life (year). Consequently, the residual diameter of the prestressed anchor after  $t$  years of corrosion exposure  $D_r$  is given by:

$$D_r = D_0 - 2d_c \quad (29)$$

where  $D_0$  is the nominal initial diameter of the anchor bar, and  $D_r$  is the residual effective diameter after corrosion-induced material loss. The corresponding residual cross-sectional area of the anchor bar  $A_r$  is calculated as:

$$A_r = \frac{\pi D_r^2}{4} \quad (30)$$

Assuming that the yield strength of the prestressed steel remains unchanged during the corrosion process (a conservative assumption widely used in preliminary durability design), the residual tensile capacity of the corroded anchor  $F_r$  can be expressed as:

$$F_r = f_y \cdot A_r \quad (31)$$

where  $f_y$  is the characteristic yield strength of the prestressed steel material.

To illustrate the practical implications of the proposed model, a representative engineering case is presented. Consider a prestressed rock bolt with an initial diameter of 50 mm installed in an aggressive groundwater environment with an average corrosion rate of 0.05 mm/year. After 30 years of service, the calculated corrosion penetration depth is:

$$d_c = 1.5 \text{ mm} \quad (32)$$

This corresponds to a residual anchor diameter of:

$$D_r = 47 \text{ mm} \quad (33)$$

A reduction in residual cross-sectional area of approximately 11.6% relative to the initial intact condition.

Coating–structure coupled design framework. The corrosion rate parameter  $v_c$  serves as the bridge connecting coating material performance to structural design. Drawing upon experimental data reported in the literature, different coating systems can be classified by their corrosion reduction efficiency (CRE), defined as:

$$R_{CRE} = \frac{(v_{c,uncoated} - v_{c,coated})}{v_{c,uncoated}} \times 100\% \quad (34)$$

where  $R_{cre}$  represents the CRE value. Based on the field and laboratory investigations reported in [23,24], the following representative values are adopted for the present framework:

$$D_r = 47 \text{ mm} \quad (35)$$

Uncoated steel:  $v_c = 0.05\text{--}0.15$  mm/year (depending on environmental corrosivity);

Epoxy coating: CRE  $\approx 60\%$ ;

High-performance composite coating: CRE  $\geq 90\%$ .

To illustrate the quantitative impact of coating selection on structural performance, Table 6 presents the predicted residual anchor properties after 30 years of service for the baseline anchor under different coating scenarios. The performance of baseline anchors under different coating conditions is shown in Table 6.

**Table 6.** The baseline anchor under different coating scenarios.

| Coating System           | CRE | $v_c$ (mm/y) | Residual Diameter (mm) | Prestress Retention |
|--------------------------|-----|--------------|------------------------|---------------------|
| Uncoated                 | 0%  | 0.10         | 44.0                   | 71%                 |
| Epoxy coating            | 60% | 0.04         | 47.6                   | 85%                 |
| High-performance coating | 90% | 0.01         | 49.4                   | 94%                 |

The parametric analysis demonstrates that long-term corrosion induces a progressive reduction in the effective tensile resistance and axial stiffness of prestressed rock bolts. This degradation process accelerates time-dependent prestress loss and elevates the probability of local anchor failure under cyclic loading conditions. These findings quantitatively underscore the critical importance of implementing effective corrosion protection measures and adopting durability-oriented design approaches to ensure the long-term service safety of prestressed rock bolt foundations in harsh geological environments.

Together with steel relaxation and anchorage-zone creep, this results in a pronounced time-dependent decay of prestress. This process is consistent with the “prestress reduction factor” introduced in Section 1 of this study, essentially reflecting the combined effects of corrosion and long-term degradation.

The corrosion penetration model presented above quantitatively illustrates the severe impact of aggressive environments on anchor integrity. To mitigate this, the application of high-performance protective coatings is a critically effective strategy. As demonstrated in field and laboratory studies, coatings specifically designed for geotechnical anchors can significantly inhibit the initiation and propagation of stress corrosion cracking by providing a robust barrier against electrolytes and corrosive ions. For the representative case analyzed (50 mm anchor, 0.05 mm/year corrosion rate), the adoption of an effective coating system, such as those recommended in [23,24], could reduce the corrosion rate by over 90%, thereby maintaining the residual cross-sectional area and tensile capacity near their initial design values throughout the service life. This protective measure is not merely an add-on but a fundamental component of a durability-oriented design, directly influencing the long-term prestress loss and structural reliability discussed in this study. The following sections will further elaborate on the synergistic benefits of combining robust void control with appropriate surface protection to achieve a holistic durability solution.

### *7.2. Grout–Rock Interface Degradation and Rock Mass Strength Deterioration*

Degradation at the grout–rock interface plays a critical role in anchorage performance. Under groundwater erosion and thermal cycling, the grout may undergo dissolution, pore structure deterioration, and microcrack propagation, leading to a gradual reduction in interface bond strength.

Meanwhile, the rock mass experiences time-dependent degradation of cohesion and internal friction angle under weathering and hydro–rock interaction, which corresponds to the rock strength reduction treatment introduced in Section 2 of this paper. From a mechanistic perspective, such degradation directly weakens the shear capacity of the group-anchor system and alters the evolution pathway of failure modes.

### *7.3. Cyclic Loading and Corrosion–Fatigue Coupling Effect*

During long-term operation of wind turbines, the anchor system is subjected to significant cyclic loading. In corrosive environments, the fatigue performance of materials is further degraded, promoting corrosion–fatigue effects that accelerate microcrack initiation and propagation, thereby reducing service life. This mechanism is closely related to the internal force redistribution of anchors and the risk of local failure discussed in this study.

### *7.4. Foundation Uplift–Void–Seepage–Corrosion Coupling Mechanism and the Imperative for Coating Protection*

From a global structural perspective, foundation uplift (void formation) is not only a mechanical issue but also an important trigger for corrosion processes. The presence of

voids changes the stress state of the foundation and provides preferential seepage pathways for groundwater, thereby intensifying corrosion and degradation in both anchor and interface zones. This leads to a coupled evolution mechanism of “uplift–seepage–corrosion–performance degradation”. Therefore, the void control criteria proposed in Section 3 are not only significant for load-bearing capacity but also serve as an explicit durability control parameter. In geotechnical engineering practice, rock anchor foundations are subjected to the coupled effects of high in situ stress and multi-media corrosion (including acid–base, sulfide, and chloride ions) during long-term service. High-performance protective coatings can significantly inhibit the initiation and propagation of stress corrosion cracking, thereby ensuring the long-term service safety and structural integrity of anchorage support systems. Extensive laboratory accelerated corrosion tests and field engineering validations have consistently demonstrated that the rational selection of application-specific high-performance coating systems, tailored to the corrosion characteristics and mechanical loading conditions of distinct service environments, can effectively isolate corrosive media from metallic substrates, retard the electrochemical corrosion process, and reduce the risk of brittle fracture of structural components. The corrosion rate parameters and prestress loss model proposed in this study are based on the corrosion degradation mechanisms observed under coating protection; if effective coating protection is not employed, the design safety factor recommended in this paper should be increased by a factor of 1.2 to 1.5. Therefore, this study recommends such composite coating systems for rock anchor foundation systems under extreme environments. The coupled “void–seepage–corrosion” mechanism elaborated above underscores a critical vulnerability: even minor foundation voids can create preferential pathways for corrosive agents. In this context, protective coatings act as the last line of defense. By maintaining the integrity of the coating, the electrochemical circuit necessary for corrosion is broken, thus decoupling the otherwise self-accelerating degradation cycle. This synergistic approach, minimizing void formation through structural design and preventing corrosion initiation through advanced coatings, is essential for ensuring the long-term service safety and structural integrity of anchorage support systems in harsh coastal or mountainous terrains.

Foundation void development not only modifies the stress redistribution mechanism of the foundation system but also profoundly influences groundwater seepage characteristics, thereby accelerating electrochemical corrosion progression in the anchorage zone.

Derivation of the void–seepage–corrosion risk index. The cumulative corrosion risk index  $C_r$  is derived from the fundamental principles of groundwater flow and electrochemical corrosion. The seepage intensity  $S_g$  is related to the void area ratio  $A_v$  through Darcy’s law, where the effective permeability increases nonlinearly with the void area. The corrosion rate is assumed to be proportional to the seepage intensity, reflecting the transport of dissolved oxygen and aggressive ions to the anchor surface. Integrating the corrosion rate over the service life  $t$  yields the dimensionless risk index. The empirical coefficients  $\lambda_v$  and  $\lambda_s$  in Equation (29) are introduced to calibrate the void sensitivity and seepage–corrosion coupling intensity, respectively, against field observations. The quadratic relationship between void area ratio and corrosion risk arises from the positive feedback loop: larger voids lead to higher seepage, which accelerates corrosion, which in turn promotes further void growth. This mechanistic derivation provides the theoretical underpinning for the void control criteria proposed in Section 4. To quantitatively characterize this coupled degradation mechanism, a dimensionless cumulative corrosion risk index is proposed herein:

$$C_r = \lambda_v \cdot \lambda_s \cdot S_0 \cdot A_v \cdot (1 + \beta A_v) \cdot t \quad (36)$$

where  $C_r$  denotes the dimensionless cumulative corrosion risk index;  $A_v$  is the foundation void area ratio;  $S_g$  represents the groundwater seepage intensity coefficient;  $t$  is the design

service life (years); and  $\lambda_v$  and  $\lambda_s$  are empirical coefficients reflecting the void sensitivity and seepage–corrosion coupling intensity, respectively.

The groundwater seepage intensity is postulated to increase nonlinearly with the void area ratio, as described by the following empirical relationship:

$$S_g = S_0 \cdot (1 + \beta A_v) \quad (37)$$

where  $S_0$  is the initial seepage intensity under intact rock–foundation contact conditions, and  $\beta$  is the seepage amplification coefficient induced by void formation and expansion.

Substituting the above equation into the corrosion risk model yields the integrated coupled corrosion risk formulation.

This expression clearly demonstrates that cumulative corrosion risk exhibits a quadratic nonlinear growth trend with increasing void area ratio, rather than the linear relationship assumed in conventional uncoupled durability analyses.

To demonstrate the practical implications of the proposed coupled model, a representative parametric analysis is conducted using baseline parameter values calibrated against field monitoring data from similar rock bolt foundation projects:

$$\lambda_v = 1.0 \quad (38)$$

$$\lambda_s = 1.2 \quad (39)$$

$$S_0 = 1.0 \quad (40)$$

$$\beta = 3.0 \quad (41)$$

Substituting these values into the model gives:

$$C_r = 1.2 A_v \cdot (1 + 3 A_v) \cdot t \quad (42)$$

When the void area ratio increases from 10% to 25%, the normalized cumulative corrosion risk increases from 0.156 t to 0.525 t, representing a 3.37-fold (approximately 3.4-fold) increase. This finding quantitatively confirms that excessive foundation void development significantly accelerates moisture ingress and intensifies electrochemical corrosion reactions in the anchorage zone, highlighting the critical role of void control in mitigating long-term durability degradation.

The parametric analysis results provide quantitative validation for the proposed design criterion, which stipulates that foundation voids must be completely eliminated under serviceability limit state conditions and restricted to a maximum void area ratio of 25% under ultimate limit state loading conditions. While the proposed semi-empirical model adopts certain simplifying assumptions for engineering practicality, it offers a quantitative framework for characterizing the complex “void–seepage–corrosion” coupled degradation mechanism and corroborates the rationality and engineering applicability of the durability-oriented void control strategy developed in this study.

In summary, the performance evolution of prestressed rock anchor foundations under extreme environments is essentially driven by the coupled action of mechanical loading and corrosion-induced degradation. In the subsequent analysis, this study implicitly incorporates durability effects into the design framework by introducing a prestress reduction factor, rock strength modification, and void control criteria, thereby achieving a durability-oriented design approach without altering the existing code-based design system.

## 8. Limitations and Applicability

### 8.1. Uniform Corrosion Assumption

The corrosion model presented in Section 7.1 assumes uniform corrosion penetration over the entire anchor surface. This assumption is widely adopted in geotechnical durability design codes and is generally valid for atmospheric and uniform groundwater corrosion scenarios where aggressive ions are evenly distributed. However, in coastal chloride-rich environments, localized pitting corrosion may occur, leading to stress concentrations and accelerated fatigue crack initiation. Additionally, high-strength prestressing steels are susceptible to stress corrosion cracking (SCC) under the combined action of tensile stress and aggressive media. The proposed uniform corrosion model does not explicitly account for these localized degradation mechanisms. For projects located in environments with high risk of pitting or SCC, the design should be supplemented with additional protective measures and, where appropriate, a higher design safety factor should be adopted. Under such conditions, the uniform model provides a conservative estimate of average section loss but does not replace a detailed SCC assessment per relevant standards.

### 8.2. Continuum Rock Mass Assumption

The shear capacity model developed in Section 3 assumes the rock mass to be an isotropic continuum characterized solely by cohesion  $c$  and internal friction angle  $\varphi$ . This assumption is valid for intact or slightly weathered rock masses with limited discontinuities. For the present case study, the medium weathered slate has an estimated RMR of approximately 65, which falls within the applicable range.

However, practical rock engineering often encounters complex geological conditions involving joints, bedding planes, faults, and other discontinuities that can fundamentally alter the failure mechanism. In heavily jointed rock masses, the pull-out resistance may be governed by block sliding along pre-existing discontinuities rather than shear failure through intact rock. For such conditions, the proposed continuum model is not applicable, and alternative approaches such as the discrete element method (DEM) or discontinuity-based analytical models should be employed.

Therefore, the proposed methodology is specifically recommended for sites with relatively intact or moderately weathered rock masses. For lower-quality rock masses, the method can still be applied as a conservative upper-bound estimate, but should be supplemented by specialized discontinuity analysis. This limitation also reinforces the engineering practice of anchoring into medium weathered or fresh rock layers to ensure the validity of the design assumptions.

## 9. Conclusions

This study addresses the critical durability challenges of onshore wind turbine prestressed rock bolt foundations in coastal extreme environments, where conventional design methods focusing solely on ultimate bearing capacity fail to systematically account for long-term corrosion-induced degradation and coupled environmental-mechanical effects, and lack a quantitative connection with anti-corrosion coating performance. By integrating durability considerations and coating protection correlation into the existing code-based theoretical framework without modifying core calculation procedures, a systematic durability-oriented design methodology is developed and verified through theoretical derivation, finite element simulation, and engineering case studies. The main conclusions are drawn as follows:

1. The time-dependent prestress loss model, accounting for the coupled effects of steel corrosion, material relaxation, and cyclic loading, demonstrates that under a typical

coastal corrosion environment, the effective cross-sectional area of a 50 mm anchor decreases by approximately 11.6% after 30 years of service. This corrosion-induced section loss results in a prestress degradation that is 15%–20% more severe than predictions based on relaxation alone, highlighting the necessity of considering corrosion in long-term performance assessment.

2. The proposed equivalent wedge-shaped shear capacity model provides a smooth transition between single-anchor conical failure and global group-anchor failure. Theoretical verification confirms that the model reverts to classical limiting cases at extreme anchor spacings, and comparisons with published field pull-out test data yield prediction errors within  $\pm 10\%$ . The model thus offers a balanced characterization of local and global group-anchor behavior.
3. The void–seepage–corrosion coupling analysis reveals that cumulative corrosion risk increases nonlinearly with void area ratio. When the void area ratio increases from 10% to 25%, the normalized corrosion risk index increases by approximately 3.4-fold. This quantitatively validates the proposed design criterion of zero void under normal conditions and a maximum 25% void area ratio under extreme conditions.
4. The coating–structure coupled framework establishes a quantitative link between coating performance and structural durability. For the representative 50 mm anchor, a high-performance coating extends the anchor service life from approximately 25 years (uncoated) to over 50 years, achieving full compatibility with the wind turbine’s design service life.
5. The engineering case study and finite element analysis demonstrate that the proposed design method satisfies all bearing capacity requirements while completely eliminating voids under normal conditions and limiting the extreme-condition void ratio to 22.1%, compared with 32.5% for a conventional design. The method thus effectively suppresses the “void–seepage–corrosion” degradation pathway with only a modest increase in foundation size and anchor count.
6. The grout–rock interface bond strength is subject to time-dependent degradation under the combined effects of groundwater chemical erosion and temperature cycling. This degradation is captured through the comprehensive reduction factor  $\eta$ , which accounts for approximately 12%–18% of the total shear capacity reduction over the 30-year design life.
7. The proposed methodology is primarily applicable to intact or moderately weathered rock masses ( $\text{RMR} \geq 50$ ). For heavily jointed rock masses or environments with high risk of pitting corrosion and stress corrosion cracking, additional safety factors and specialized analysis methods are recommended, as discussed in Section 9.

## 10. Future Work

This study mainly adopts theoretical derivation, finite element simulation, and engineering case analysis to establish a durability-oriented design method for prestressed rock bolt foundations in coastal extreme environments, and initially constructs the coupling relationship between coating protection performance and structural durability design. Further research will focus on the following aspects:

1. Long-term performance evolution of anti-corrosion coatings for rock bolts in coastal corrosion environments. Combined with accelerated salt spray corrosion tests in the laboratory and field exposure tests in coastal wind farms, electrochemical impedance spectroscopy (EIS), interface bond strength testing, and surface morphology characterization will be adopted to quantify the degradation law of protection performance and interface mechanical properties of different coating systems (such as graphene-modified coatings, epoxy composite coatings) under the coupling of salt spray erosion and cyclic stress. Accurate corrosion rate parameters and coating life prediction

models will be established to provide basic data for the durability design method proposed in this study.

2. Large-scale physical model tests will be conducted to verify the proposed group-anchor shear capacity model under multi-working conditions. Special attention will be paid to the influence of different coating protection grades and corrosion degradation degrees on the mechanical behavior and failure mode of the group anchor system, and the relevant parameters of the durability modification model will be optimized and calibrated based on test data.
3. Systematic sensitivity analysis considering multiple design scenarios will be carried out to quantify the coupling influence of geological conditions, wind load characteristics, coastal corrosion intensity, and coating protection grades on the full-life durability of foundations. On this basis, a matching criterion between anti-corrosion coating selection and structural design parameters will be established to support the integrated durability design of coating–structure systems.

**Author Contributions:** Conceptualization, Z.X.; Methodology, J.X., D.D., Z.X. and J.H.; Writing—original draft, J. X., D.D., Z.X., J.H., J.S., W.Z., D.L., A.Z., C.B. and X.X.; Supervision, J.S. and W.Z. All authors have read and agreed to the published version of the manuscript.

**Funding:** This research was funded by [University-Industry Collaborate Innovation Science and Technology Program Project of Fujian Province] grant number [No. 2024Y4013], [Beijing Municipal Natural Science Foundation Chaoyang Key Project] grant number [No. L259028].

**Data Availability Statement:** The original contributions presented in this study are included in the article. Further inquiries can be directed to the corresponding author.

**Conflicts of Interest:** Authors Jian Xu, Jing Huang, Jianwei Su, Wenbo Zhou were employed by the company Dongfang Electric Power Engineering Consulting Corporation. Author Dongpo Dong was employed by the company Southwest Municipal Engineering Design & Research Institute of China. Author Xueyun Xing was employed by the China Railway 18th Bureau Group No. 4 Engineering Co Ltd The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as potential conflicts of interest.

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