Study on the Time-dependent Reliability of Corroded Reinforced Concrete Bridge Structures due to Ship Impact

Tao Fu,1 Zhixin Zhu,1 Yan Li,1 Yue Sun,1 and Lingxiao Meng2

1School of Transportation Engineering, Shandong Jianzhu University, Jinan, China
2China State Construction Infrastructure Corporation, Beijing, China

Correspondence should be addressed to Tao Fu; greenvillage_17@163.com

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The cumulative damage caused by chloride ion erosion to coastal bridge structures reduces the ship impact resistance of channel bridge structures and leads to changes in reliability of bridge structures on ship impact. It is of great theoretical and practical significance to study the performance degradation change law of corroded bridge structures and to evaluate the ship impact time-dependent reliability of in-service corrosion-damaged bridge structures. Based on the durability decay model of reinforced concrete structures in a chloride ion erosion environment, the time-dependent resistance analysis of bridge structures is carried out considering the reduction of yield strength of longitudinal reinforcement and hoop reinforcement in pile sections and the decay of compressive strength of protective layer concrete. Based on the study of the probabilistic model and parameters of random variables affecting the time-dependent reliability of ship-bridge collision, the typical damage modes of bridge structures under ship impact are analyzed, the time-dependent reliability analysis model of bridge structures under ship impact is established based on the structural damage criterion, the ship-bridge crash limit state functional function is given, and the time-dependent reliability analysis of ship-bridge collision is carried out based on the response surface method.

1. Introduction

Bridges are destroyed or severely damaged by ship impacts around the world. Ship impacts not only cause huge economic losses and casualties but also have adverse political consequences and cause serious environmental damage [1–3]. Therefore, safety around ship impacts on existing and proposed bridges is one of the key technical issues that must be carefully addressed by engineers and bridge managers. Due to the advantages of concrete structures in terms of material extraction, cost, and maintenance, they have been absolutely dominant in bridge engineering and have a very wide range of applications in major bridge projects. Corrosion of concrete bridge structures is a major threat to the durability of concrete bridges around the world, and many bridge engineering materials are aging, deteriorating in performance, and decreasing in use function and load-bearing capacity. Under an impact load, such as a ship collision, when the substructure of the bridge is corroded, its vertical bearing capacity and lateral anti-scour ability are significantly decreased. For bridges in navigable waters that often suffer ship collisions, various types of protective structures have been used over the past decades to protect bridge piers from ship collisions in navigable waters [4–6]. Cai et al. established a ship collision risk model based on AIS data, considering ferry and multiojective ships, which provided a new idea to ensure the safety of ferry [7]. Song et al. proposed a simplified analysis model of ship-bridge collision to predict the reliability and safety of the bridge structure under ship collision impact load [8]. Wu et al. pointed out the challenges faced by data-driven bridge operation and maintenance [9]. Zhang et al. applied impact load on precast segmental assembled concrete columns to study the mechanical properties of these, and the results showed that under impact load, the precast segmental assembled concrete columns had good flexibility and deformation ability [10]. Wang et al. showed that when a barge collided with a pier, the shape and size of the pier would
affect the ship collision process [11]. Luperi and Pinto proposed a simplified method to determine the development process of impact force during collision based on the collision between barge–ship and multiple piers [12]. Wang and Morgenthal simplified the ship as a mass-spring model, and the wharf was discretized into linear elastic beam elements, which improved the efficiency of simulating the ship collision process [13]. Guo et al. simulated the ship collision of twin tower cable stayed bridge subjected to local scour [14]. The results showed that the local scour had an effect on the location where the ship collision occurred. Chen et al. developed a new type of adaptive arresting vessel device (AAVD) and determined the key parameters affecting the arresting effect through the proportional model test, which verified the feasibility of the arresting device [15]. Jacinto et al. used Bayesian method to evaluate the reliability of bridges and considered the corresponding statistical uncertainty [16]. Ma et al. studied the variation of steel section area loss and evaluated the influence of steel corrosion, according to different corrosion types [17]. For regional risk and loss assessment, Li et al. proposed a two-stage method of vulnerability function of engineering structure based on field measurement and experimental data [18]. Considering the boundary of parameter distribution, Alam et al. proposed a reliability-based prediction framework of failure structure residual life, and established a Bayesian probability box (p-box) model of cognitive uncertainty [19]. Alfred et al. proposed a new method of prediction function and standard of structure engineering monitoring based on Bayesian method; meanwhile, the historical data of engineering structure monitoring were included in structural reliability assessment [20]. Zanini et al. proposed a comprehensive evaluation method for the service life curve of existing bridges and calibrated it with visual data [21].

By studying the degradation law of corrosion bridge structure performance and changing reliability over time, potential risks can be identified in a timely manner, and maintenance decisions can be made to avoid the occurrence of major ship collision accidents and ensure bridge service life.

2. Reliability Analysis Model with Limit State Functional Functions of Ship-Bridge Collision

According to the statistics of PIANC Working Group 19 of the Standing Conference of International Nautical Associations (1998) [22], the rate of ship impacts on bridges in the world with damage of over $100,000 was about 1 accident/year in 1975, whereas in 1990, the rate had increased to about 2 accidents/year, and the annual accident rate is following an increasing trend. The International Association of Bridge Structural Engineering (IABSE) found through the study of 29 major international ship collisions that despite the progress in science and technology to improve the quality of equipment on ships, safety supervision and management, as well as the construction and management of bridges, ship-bridge collisions have continued to occur over the years [23].

Since ship collisions are short-duration impact loads and of high intensity, the impact response of the structure is both local and overall, and appropriate structural measures can be taken to ensure that local penetration of the bridge structure does not occur under impact loads, so the focus should be on the overall dynamic behavior of the bridge members under impact loads. A summary analysis of ship-bridge collisions shows that most of them damage weak piers or pile foundations. From the accident phenomena and the basic mechanical concepts, it can be qualitatively determined that the pier and pile foundations may suffer either bending damage or impact shear damage.

In terms of ship impacts on bridges, the ship may strike the bearing, pier, main girder, or main arch ring of the bridge. This article focuses on the study of substructures subjected to ship impacts where bending damage or shear damage of bridge members may occur. Bridge piers and pile foundations are generally slender reinforced concrete members or steel pipe (steel pipe concrete) members, and to ensure the ductile damage pattern of concrete members under impact loading, the shear capacity of the members should be appropriately increased to avoid the occurrence of shear damage mode due to punching shear damage effect. For the case of bending damage of bridge piers and pile foundations, the degree of damage of the members can be measured using the section turning angle index.

2.1. Reliability Analysis Model and Limit State Functional Function for Pile and Column Piers. In general, for pile and column piers, the damage form of ship impact as a horizontal force can be classified as bending damage and shear damage.

2.1.1. Shear Damage Mode. For piers in shear damage mode, according to the strength damage criterion, the shear force that the pier may be subjected to is less than its shear capacity, so its limit state functional function is

\[ g(X) = Q_{R,pier} - Q_{S,pier}. \]  

2.1.2. Bending Damage Mode. For piers in bending damage mode, according to the deformation damage criterion, the possible turning angles of piers should be less than their ultimate turning angles, whose limit state functional function is

\[ g(X) = \theta_{R,pier} - \theta_{S,pier}. \]  

For group pile foundations, the limit state functional function for the most unfavorable monopile is

\[ g(X) = \theta_{R,pile foundation} - \theta_{S,pile foundation}. \]  

For the bridge substructure system of pile-column piers, it can be considered as a tandem system consisting of group pile foundation, bearing, and pier; therefore, it can be considered that after the failure of any element, the whole tandem system fails, that is, the whole bridge structure fails.
According to the simplified bridge substructure tandem system, the limit state functional function of pile-column piers is established as follows:

\[ G_f(X) = \min\{\theta_{R, \text{pile foundation}} - \theta_{S, \text{pile foundation}}, \min\{V_{R, \text{pile}} - V_{S, \text{pile}}, \theta_{R, \text{pile}} - \theta_{S, \text{pile}}\} \}. \] (4)

corrosion of reinforcing bars in a chloride salt attack environment, given \( T_{\text{corr}} \) in the literature:
\[ T_{\text{corr}} = \left( \frac{c}{K} \right)^2 \times 10^{-6} + 0.2t_1, \] (8)
where \( c \) is the thickness of the protective layer of concrete, \( K \) is the chloride salt erosion factor, \( t_1 \) is the time for the chloride ion concentration on the concrete surface to reach stability, \( D \) is the chloride ion diffusion coefficient, \( \text{erf} \) is the Gaussian error function, \( C_{cr} \) is the critical chloride ion concentration for reinforcement corrosion, and \( C_s \) is the chloride ion concentration on the concrete surface.

3.2.2. Rate of Corrosion of Steel Reinforcement. The corrosion rate of steel bars is the corrosion depth of a steel bar section in a unit of time. Using the model suggested in Dimitri et al. [25], it is considered that the corrosion rate of steel bars remains unchanged after concrete cover cracking.

Based on the principle of electrochemical corrosion to determine the corrosion current density of reinforcement \( i_{\text{corr}} \) (\( \mu \text{A/cm}^2 \)), the literature [26] gives the \( i_{\text{corr}} \) formula for calculating the corrosion current density of reinforcement under the action of chloride salt attack:
\[ \ln i = 8.617 + 0.618 \ln C_d + \frac{3034}{T + 273} - 5 \times 10^{-3} \rho + \ln m_{cl}, \] (9)
\[ C_d = C_{d0} + (C_s - C_{d0}) \left[ 1 - \text{erf} \left( \frac{c \times 10^{-3}}{2\sqrt{D}} \right) \right], \] (10)

From equation (9), we obtain \( i_{\text{corr}} \) and the rate of corrosion of the reinforcing steel can be determined. Referring to Dimitri et al. [25], corrosion current density is obtained from Faraday’s law of electrolysis as 1 \( \mu \text{A/cm}^2 \), which corresponds to an average annual corrosion depth of 11.6 \( \mu \text{m} \). The average annual corrosion rate of reinforcing steel is a constant and the rate of corrosion of reinforcing steel is
\[ \lambda = 0.0232i_{\text{corr}}. \] (11)

3.2.3. Time-dependent Area of Rusted Reinforcement Sections. It is assumed that all reinforcing bars lose the same diameter at the same rate. The time-dependent area of the rebar cross section is determined from the relationship

3.2. Rebar Corrosion Model

3.2.1. Corrosion Initiation Time of Reinforcement. The following equation is used to calculate the time for the onset of...
between the rebar corrosion current density \( i_{\text{corr}} \) and the rebar diameter or radius loss established by Faraday’s electrolysis theorem. It is assumed \( i_{\text{corr}} \) is constant.

The rate of reinforcement corrosion, the reduction in reinforcement diameter is expressed as follows:

\[
\Delta d = \lambda (t - t_i).
\]  

(12)

Denoted by \( d(t) \) the remaining diameter of the reinforcement at moment \( t \) is obtained from (12):

\[
d(t) = d_{\text{r}} - \lambda (t - t_i) = d_{\text{r}} - 0.0232i_{\text{corr}} (t - t_i).
\]  

(13)

Thus, the time-dependent cross-sectional area \( A(t) \) can be expressed as follows:

\[
A(t) = \begin{cases} 
\frac{nn d_{\text{r}}^2}{4}, & t \leq t_i, \\
\frac{nn d(t)^2}{4}, & t_i \leq t \leq t_i + \frac{d_{\text{r}}}{\lambda}, \\
0, & t \geq t_i + \frac{d_{\text{r}}}{\lambda},
\end{cases}
\]  

(14)

where \( n \) is the number of reinforcement bars and \( t \) is the time of rusting of the bars.

| Table 1: Concrete surface chloride ion concentration \( C_\text{c} \). |
|---------------------------------|-----------------|-----------------|-----------------|-----------------|
| Water level change zone         | Splash zone     | Atmospheric salt spray area | Offshore atmospheric zone |
| 19                             | 17              | 11.5             | 0.1 km          | 0.25 km         | 0.5 km          | 1.0 km          |
|                                 |                 |                  | 5.87            | 3.83            | 2.57            | 1.28            |

Now, the depth of corrosion of reinforcement \( x \) under uniform corrosion is calculated according to (18):

\[
x = \frac{\Delta d(t)}{2}.
\]  

(18)

Bringing the time-dependent rust depth into (17), the compressive strength of the concrete at different stages of protection can be calculated.

3.4. Time-dependent Resistance Model. With material deterioration, the bridge structure resistance also changes continuously, according to the above material deterioration model, considering concrete time-dependent compressive strength, steel time-dependent yield strength, steel time-dependent cross-sectional area parameters. Introducing parameters as a correction to the resistance, the resistance model is expressed as follows:

\[
R(t) = K_p R_p (f_{\text{cu}}(t), f_{\text{st}}(t), A_s(t)),
\]  

(19)

where \( t \) is the service time of the structure.

4. Ship-Bridge Impact Dynamic Response Analysis Method

In order to obtain the dynamic response of the ship-bridge impact, the dynamic analysis model shown in Figure 1 is established. The bridge bearing is simulated as a rigid block, and the rest is modeled by beam units. The ship impact is a short-time impact load, and the impact process is characterized in the form of a time-course curve, which is applied to the dynamic analysis model to obtain the time-course response of the structure.
The collision load time course model uses the dimensionless $F-T$ probability model \cite{28} and ship impact force time course is calculated by equation (20):

$$F(t) = \frac{I}{T} \times \frac{\pi}{K} \times T^2 \times \left( \frac{t}{T} - m \right)^2 + \frac{n^2}{T} \times \sin \left( \frac{\pi t}{T} \right), \quad (0 < t < T).$$

The dimensionless parameters $K$, the impulse $I$, and the duration $T$ of the model are calculated according to Equations (21)–(23).

$$K = 1 - \frac{4}{\pi^2} - 2m + 2m^2 + 2n^2;$$

$$I = 1.39 \times 10^3 \times v \times \text{DWT},$$

$$T = 0.0936v^{0.25} \times \text{DWT}^{0.1}. \quad (23)$$

The parameter $m$ is uniformly distributed with a value interval of $[0.425, 0.525]$; the parameter $n$ is normally distributed with a mean of 0.36, a standard deviation of 0.10, and a coefficient of variation of 0.28.

5. Stochastic Probability Model for Ship-Bridge Collision Reliability Analysis

According to the aforementioned form of the ship impact time curve, the main factors that have a large impact on the reliability of ship-bridge impacts are impact speed and impact angle.

5.1. Impact Tonnage Probability Model. A uniform distribution was used to simulate the tonnage of the impacting ship.

Assume that the ship impact tonnage DWT obeys a uniform distribution on $(a, b)$ with a probability density function:

$$f(x) = \begin{cases} \frac{1}{b-a}, & (a \leq x \leq b), \\ 0, & \text{else}. \end{cases} \quad (24)$$

5.2. Impact Velocity Probability Model. According to the AASHTO Bridge Design Code a relational curve in the form of a zigzag line was chosen to simulate the velocity distribution of a yawing vessel, assuming that the law of reduction of the ship’s speed decreases linearly from the edge of the channel to a distance of three times the typical length of the ship (LOA), that the maximum speed is taken as the typical speed of the ship, that the minimum speed is taken as the average current speed, and that the mathematical expression for the ship impact velocity is used.

$$V = \begin{cases} \frac{x_L V_T - x_c V_{\text{min}} - x(V_T - V_{\text{min}})}{x_L - x_c}, & x_c < x \leq x_L, \\ V_{\text{min}}, & x > x_L, \end{cases} \quad (25)$$

where $V$ is the design impact velocity; $V_T$ is the typical navigational speed of ships in the waterway; $V_{\text{min}}$ is the minimum impingement velocity (not less than the annual average current velocity); $x$ is the distance of the ship from the bridge abutment; $x_c$ is the distance of the ship from the edge of the channel; $x_L$ is $3 \times \text{LOA}$ distance from the centerline of the ship’s channel.

The impact velocity of a ship depends on $V_T$ and $V_{\text{min}}$, two random variables, and they are described by a normal distribution, and the impact velocity $V$ is a linear combination of these two mutually independent variables so that the impact velocity $V$ also follows a normal distribution.

6. Time-dependent Reliability Analysis of Ship-Bridge Collisions Based on Response Surface Method

The basic idea of the response surface method is to approximate the real functional function or limit state surface by constructing a response surface function or response surface, making the implicit limit state functional function $G(X)$ explicit, and fitting the response surface to a series of sample points to obtain the response surface function, and then completing the reliability analysis using first second order moment theory.

The specific computational procedure for the ship-bridge impact reliability solution using the response surface method is shown in Figure 2, the specific calculation process is as follows:
(1) The ship-bridge impact reliability influencing factors are taken as the basic random variables, and the ship impact force time curve is obtained according to the mean value of its probability distribution as the initial iteration point $X_0$. The ship impact force time curve is applied to the bridge structure, and finite element analysis is performed to obtain the ship impact dynamic response of the structure, and the function value of the structural function $G(X)$ at the mean point is calculated.

(2) The quadratic polynomial is selected as the response surface function, and the distribution range of the test points is determined according to the test design method of the response surface, combined with the probability characteristics of the random variables under consideration, to generate a series of sample test points at the initial iteration point $X_0$, to generate the ship impact force time curve according to the test points, and then to find the ship impact dynamic response of the structure, and to calculate the
function value of the structure function \( G(X) \) at the test points. For nonnormal random variables, equivalent normalization is required.

(3) Determination of the response surface function. According to the selected form of the response surface function, the corresponding coefficient matrix \([A]\) is generated; the vector \( g \) is established from the calculated values of the function at the sample test points, and the coefficients to be determined for the response surface function are derived by solving the system of linear equations, that is, the response surface function is determined.

An explicit functional function is often used as a quadratic multivariate polynomial.

\[
G(X) = a_0 + \sum_{i=1}^{n} a_i x_i + \sum_{i=1}^{n} b_i x_i^2.
\] (26)

Response surface function coefficients to be determined.

\[
\lambda = \frac{[A]}{G(X)}
\] (27)

The coefficients to be derived from (27) are substituted into the response surface function.

(4) Solving reliability indicators and verification points based on the first second-order moment method \( x_i^* \).

(5) Iterative calculation.

The new sampling center is found by equation (28):

\[
X_M = \mu_x + (x^* - \mu_x) \times \frac{G(\mu_x)}{G(x^*) - G(\mu_x)},
\] (28)

where \( G(\mu_x) \) is the value of the structure function \( \mu_x \) and \( G(x^*) \) is the value of the structure function \( x^* \).

Steps (2)–(4) are repeated until the set termination criterion is satisfied, and the difference between the reliability metrics of the two preceding and following iterations is chosen to be less than 0.001 as the convergence criterion.

### 7. Application to Engineering Examples

#### 7.1. Overview of the Project

The structure is a coastal bridge located in a hot and humid zone with an average temperature of 20°C and 57% humidity throughout the year. In the offshore environment, the structure is subjected to erosion of chloride ions all year round. The superstructure of the main bridge is a (65 + 110 + 65) m three-span prestressed concrete continuous rigid system with variable section box girders for the main girders. The piers of the main bridge are double-limbed rectangular solid piers with equal sections, and the foundations are drilled (dug) piles. Piers 1 and 4 are pile-column type.

#### 7.2. Finite Element Model of the Bridge Structure

The bridge structure is simulated using spatial beam unit with consideration of pile-soil interaction and elastic connection units for the bridge bearings. The main girders are rigidly connected to the piers, and master–slave constraints are set between the piers and the bearing and the bearing and the pile foundation, and the pile bottoms are solidified. The bridge diagram is shown in Figure 3 and the established finite element model is shown in Figure 4.

#### 7.3. Analysis of the Deterioration Pattern of Reinforced Concrete Materials for Bridge Structures

#### 7.3.1. Analysis of Corrosion Deterioration of Steel Reinforcement

In view of the coastal environmental conditions in which the bridge is located, this article considers chloride salt erosion as the most important factor leading to reinforcement corrosion, and the reinforcement corrosion degradation parameters are listed in Table 2. The corrosion rate of the reinforcement was calculated to be 0.0444 mm/a, and the time to start corrosion of the reinforcement was 23.5 years, from which the diameter of the reinforcement and the corresponding corrosion rates and standard values of the yield strength of the reinforcement for 0, 40, 60, 80, and 100 years were obtained as shown in Table 3.

#### 7.3.2. Analysis of the Deterioration of the Compressive Strength of Concrete

The attenuation of compressive strength of the protective layer of concrete due to corrosion of reinforcement is considered. The calculated decay of the compressive strength of the protective layer concrete is shown in Table 4.

#### 7.4. Analysis of Time-Dependent Resistance of Bridge Structures

The top section of the most dangerous monopile of the group pile foundation, which is most susceptible to damage by ship impact, is selected for the resistance calculation analysis to obtain its maximum allowable angle of rotation. The bending moment curvature of the pile foundation members was calculated to obtain the ultimate characteristic values of the members. The pile foundation of the bridge is made of C30 concrete with 36 longitudinal reinforcements configured with 28 mm diameter HRB335 grade steel bars and hoop reinforcement configured with 8 mm diameter HRB335 grade steel bars, as shown in Figure 5. Based on the yield strength of longitudinal reinforcement and hoop reinforcement at 40, 60, 80, and 100 years and the compressive strength of the protective layer concrete, the maximum allowable turning angle is calculated and listed in Table 5, and its variation curve with time is plotted in Figure 6.

This shows a decreasing trend in the maximum allowable turning angle of monopiles over time due to chloride ion erosion, with an overall decrease of about 20.9%; between 80 and 100 years, it decreases at a faster rate of about 9.5%.

#### 7.5. Time-dependent Reliability Analysis of Bridge Structures under Ship Impact

A dimensionless F–T probabilistic model load time curve is used to consider the effects of two random variables, ship tonnage and ship impact velocity, on the ship
impact force, and the impact velocity is described by a normal distribution with a mean ship impact velocity of 3.25 m/s and a coefficient of variation of 0.1116. By conducting research on actual navigable ships in the bridge area, a uniform distribution obeying [0, 7000] is used to simulate the ship impact tonnage.

In this article, the quadratic polynomial ignoring the cross term is chosen as the response surface function, and the test sample sampling is carried out using the central composite design on only the axial points in the response surface method according to the statistical parameters and distribution types of the impact velocity and impact tonnage.

### Table 2: Deterioration parameters of steel corrosion.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diffusion coefficient of chloride ions</td>
<td>(5.38 \times 10^{-8} \text{cm}^2/\text{s})</td>
</tr>
<tr>
<td>The average year-round temperature of the local environment (T)</td>
<td>20°C</td>
</tr>
<tr>
<td>Water-cement ratio (w/c)</td>
<td>0.4</td>
</tr>
<tr>
<td>Annual average ambient relative humidity RH</td>
<td>57%</td>
</tr>
<tr>
<td>Concrete surface chloride ion concentration (C_s) (0.1 km from shoreline)</td>
<td>5.87 kg/cm³</td>
</tr>
<tr>
<td>Chloride ion concentration on rebar surface (C_{cr})</td>
<td>2.1 kg/cm³</td>
</tr>
</tbody>
</table>

### Table 3: Analysis of corrosion deterioration of reinforcing steel at various moments.

<table>
<thead>
<tr>
<th>Time/years</th>
<th>Diameter/mm</th>
<th>Rust rate %</th>
<th>Yield strength/MPa</th>
<th>Yield strength/MPa</th>
<th>Yield strength/MPa</th>
<th>Yield strength/MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>28</td>
<td>0</td>
<td>335</td>
<td>329</td>
<td>316</td>
<td>294</td>
</tr>
<tr>
<td>40</td>
<td>27.3</td>
<td>4.9</td>
<td>329</td>
<td>316</td>
<td>305</td>
<td>284</td>
</tr>
<tr>
<td>60</td>
<td>26.4</td>
<td>11.1</td>
<td>316</td>
<td>305</td>
<td>294</td>
<td>266</td>
</tr>
<tr>
<td>80</td>
<td>25.5</td>
<td>17.1</td>
<td>305</td>
<td>294</td>
<td>284</td>
<td>266</td>
</tr>
<tr>
<td>100</td>
<td>24.6</td>
<td>22.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 4: Calculation of concrete attenuation in the protective layer at each moment.

<table>
<thead>
<tr>
<th>Time/years</th>
<th>Compressive strength/MPa</th>
<th>Compressive strength/MPa</th>
<th>Compressive strength/MPa</th>
<th>Compressive strength/MPa</th>
<th>Compressive strength/MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>25.5</td>
<td>8.6</td>
<td>4.6</td>
<td>3.2</td>
<td>2.4</td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>80</td>
<td></td>
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<tr>
<td>100</td>
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</table>

Figure 3: Bridge layout diagram.

Figure 4: Dynamic finite element model of the bridge.
and the sampling is taken as 1.7. The obtained test sample is brought into the dimensionless F-T probability model to generate the load time curve, and it is applied to the bridge bearing to obtain the pile plastic hinge. The maximum turning angle in the region, the initial test point samples, and the values of the function at the test points, are shown in Table 6.

Combining the quadratic polynomial form without cross terms.

\[ Z = a_0 + a_1 V + a_2 DWT + a_3 V^2 + a_4 DWT. \]  

(29)

The system of linear equations is obtained from the data equation (29) in Table 6 as follows:

\[
\begin{align*}
0.013735165 &= a_0 + 2.63324a_1 + 3500a_2 + 6.93395a_3 + 12250000a_4, \\
0.013663165 &= a_0 + 3.86676a_1 + 3500a_2 + 14.9518a_3 + 12250000a_4, \\
0.013817165 &= a_0 + 3.255a_1 + 64.9659a_2 + 10.5625a_3 + 4194.621585a_4, \\
0.006846165 &= a_0 + 3.255a_1 + 6935.23a_2 + 10.5625a_3 + 48097472.05a_4, \\
0.013703165 &= a_0 + 3.255a_1 + 3500a_2 + 10.5625a_3 + 12250000a_4. \\
\end{align*}
\]  

(30)

Solving the linear system of equations yields the response surface equation as follows:

\[ Z = 0.0138 + 9.9808 \times 10^{-7} DWT - 1.0515 \times 10^{-5} V^2 - 2.587 \times 10^{-10} DWT^2. \]  

(31)

After the response surface equation is obtained, the impact tonnage obeying uniform distribution is equivalently normalized and the mean \( m_{X_i} \) and standard deviation of the equivalence normalization \( \sigma_{X_i} \) are 3500 and 2792.596, respectively, to establish the constrained optimization model.

\[
\beta_{\text{min}} = \left( \frac{X(1) - 3.25}{0.3628} \right) + \left( \frac{X(2) - 3500}{2792.596} \right) . \]  

(32)

The solution yields a 1st iteration reliability index of 1.9243 and the checkpoint (3.2513, 8873.8701).

Combining the quadratic polynomial form without cross terms.

\[ Z = a_0 + a_1 V + a_2 DWT + a_3 V^2 + a_4 DWT. \]  

(29)

The system of linear equations is obtained from the data equation (29) in Table 6 as follows:

\[
\begin{align*}
0.013735165 &= a_0 + 2.63324a_1 + 3500a_2 + 6.93395a_3 + 12250000a_4, \\
0.013663165 &= a_0 + 3.86676a_1 + 3500a_2 + 14.9518a_3 + 12250000a_4, \\
0.013817165 &= a_0 + 3.255a_1 + 64.9659a_2 + 10.5625a_3 + 4194.621585a_4, \\
0.006846165 &= a_0 + 3.255a_1 + 6935.23a_2 + 10.5625a_3 + 48097472.05a_4, \\
0.013703165 &= a_0 + 3.255a_1 + 3500a_2 + 10.5625a_3 + 12250000a_4. \\
\end{align*}
\]  

(30)

Solving the linear system of equations yields the response surface equation as follows:

\[ Z = 0.0138 + 9.9808 \times 10^{-7} DWT - 1.0515 \times 10^{-5} V^2 - 2.587 \times 10^{-10} DWT^2. \]  

(31)

The new sampling center \( X_M = (3.2507,6264.8500) \) was calculated to obtain the new sample test points, as shown in Table 7.

The iterations were carried out according to the response surface method iteration procedure, and the termination criterion was satisfied after seven iterations. The test points for the seventh iteration are shown in Table 8.

The iterative process of calculating the ship-crash reliability index at bridge formation is shown in Table 9. After seven iterations, the reliability index at bridge formation is 2.4985, and the ship collision failure probability is 0.6236%.
The response surface equations and reliability metrics obtained from the 0, 40, 60, 80, and 100 years iterations of the bridge after completion are listed in Table 10.

The trends of time-dependent reliability indicators and probability of failure during ship impact of bridges are shown in Figures 7 and 8. With the change of time, the reliability indicators of bridge structures under ship impact show a decreasing trend throughout the bridge life cycle considering the effect of chloride salt erosion on the durability of concrete structures. During the first 60 years of the
Table 9: Iterative process of ship crash reliability at bridge formation.

<table>
<thead>
<tr>
<th>Number of iterations</th>
<th>Response surface equation (math.)</th>
<th>Reliable indicators</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$Z = 0.0138 + 9.9808 \times 10^{-9}V + 9.8527 \times 10^{-7}DWT - 1.0515 \times 10^{-3}V^2 - 2.587 \times 10^{-10}DWT^2$</td>
<td>1.9243</td>
</tr>
<tr>
<td>2</td>
<td>$Z = -0.044 + 0.0353V + 8.2406 \times 10^{-6}DWT - 0.0063V^2 - 9.8489 \times 10^{-11}DWT^2$</td>
<td>2.5545</td>
</tr>
<tr>
<td>3</td>
<td>$Z = -0.0232 + 0.0224V + 6.9138 \times 10^{-6}DWT - 0.0043V^2 - 9.34 \times 10^{-10}DWT^2$</td>
<td>2.2751</td>
</tr>
<tr>
<td>4</td>
<td>$Z = -0.0257 + 0.022V + 8.7687 \times 10^{-6}DWT - 0.0045V^2 - 1.0706 \times 10^{-11}DWT^2$</td>
<td>2.5474</td>
</tr>
<tr>
<td>5</td>
<td>$Z = -0.0283 + 0.0254V + 6.7497 \times 10^{-6}DWT - 0.0048V^2 - 8.9472 \times 10^{-10}DWT^2$</td>
<td>2.3759</td>
</tr>
<tr>
<td>6</td>
<td>$Z = -0.0328 + 0.0274V + 7.5383 \times 10^{-6}DWT - 0.0052V^2 - 9.4735 \times 10^{-10}DWT^2$</td>
<td>2.4977</td>
</tr>
<tr>
<td>7</td>
<td>$Z = -0.0344 + 0.0286V + 7.0432 \times 10^{-6}DWT - 0.0053V^2 - 9.0375 \times 10^{-10}DWT^2$</td>
<td>2.4985</td>
</tr>
</tbody>
</table>

Table 10: Iterative process of ship crash reliability at bridge formation.

<table>
<thead>
<tr>
<th>Bridge completion time/years</th>
<th>Response surface equation (math.)</th>
<th>Reliable indicators</th>
<th>Failure probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>$Z = -0.0344 + 0.0286V + 7.0432 \times 10^{-6}DWT - 0.0053V^2 - 9.0375 \times 10^{-10}DWT^2$</td>
<td>2.4985</td>
<td>0.006236</td>
</tr>
<tr>
<td>40</td>
<td>$Z = 0.2048 - 0.1033V + 9.5107 \times 10^{-6}DWT + 0.0123V^2 - 1.2551 \times 10^{-9}DWT^2$</td>
<td>2.2061</td>
<td>0.0137</td>
</tr>
<tr>
<td>60</td>
<td>$Z = 0.2888 - 0.1009V - 7.597 \times 10^{-6}DWT + 0.012V^2 - 6.7326 \times 10^{-11}DWT^2$</td>
<td>2.1513</td>
<td>0.0157</td>
</tr>
<tr>
<td>80</td>
<td>$Z = 0.0154 - 0.0159V + 1.7952 \times 10^{-5}DWT + 0.0012V^2 - 2.1236 \times 10^{-9}DWT^2$</td>
<td>1.6616</td>
<td>0.0483</td>
</tr>
<tr>
<td>100</td>
<td>$Z = 0.31354 - 0.1265V - 1.4099 \times 10^{-5}DWT + 0.0157V^2 + 6.2182 \times 10^{-10}DWT^2$</td>
<td>0.7765</td>
<td>0.2187</td>
</tr>
</tbody>
</table>

Figure 7: Trends in ship-crash time-dependent reliability indicators.

Figure 8: Trends in ship-crash time-dependent failure probabilities.
life cycle, the reliability index changes relatively smoothly, and when it has been in service for more than 60 years, its reliability index decreases more rapidly.

8. Conclusion
Since the cumulative damage caused by the coastal environment on the bridge structure will reduce the anti-ship collision ability of the bridge structure, the time-dependent reliability analysis of the bridge structure due to ship collision can detect the potential risks in time and make maintenance decisions, which can effectively avoid the occurrence of ship collision accidents and improve the safety of bridge structures on fairways. The conclusions are mainly as follows:

1. This article establishes the basic process of time-dependent reliability analysis of ship-bridge collision, considering time-dependent compressive strength of concrete, time-dependent yield strength of steel, time-dependent cross-sectional area parameters of steel for bridge time-dependent resistance analysis, based on the established reliability analysis model, functional function for ship-bridge collision failure probability calculation, and probability model of each influencing factor affecting failure probability.

2. The response surface equations and reliability indexes are obtained after finite iterations based on the convergence criterion, which improves the efficiency of the calculation of the ship-bridge collision failure probability.

3. The time-dependent reliability analysis of ship-bridge impact is carried out for an actual bridge project by using the method established in this article. The analysis results show that the ship impact reliability index of the bridge structure shows a decreasing trend throughout the bridge life cycle considering the influence of chloride ion erosion.

4. In the middle and late stages of the bridge life cycle, there is an obvious decreasing trend, gradually increasing the risk of bridge structure failure under the action of ship impact. The bridge should be strengthened and maintained for ship impact according to the service time and acceptable risk guidelines.

Data Availability
All data, models, and code generated or used during the study appear in the submitted article.

Conflicts of Interest
The author(s) declare no potential conflicts of interest with respect to the research, authorship, and/or publication of this article.

Authors’ Contributions
Tao Fu conceptualized the study; reviewed and edited the article; supervised the study, and did funding acquisition.

Zhixin Zhu performed investigation, wrote the original draft; performed formal analysis; performed data curation; performed validation; reviewed and edited the article. Yan Li performed formal analysis; reviewed and edited the article. Yue Sun performed formal analysis. Lingxiao Meng: data curation.

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References
[12] F. J. Luperi and F. Pinto, ”Determination of impact force history during multicolumn barge flotilla collisions against


