Research Article

Vessel-Bridge Collision Reliability Assessment Based on Structural Dynamic Analysis

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Reliability analysis of vessel-bridge collision plays an important role in the construction of inland bridges. In this paper, a new method is proposed based on structural dynamic analysis. The random characteristics of three factors—impact angle, deadweight tonnage of vessels, and impact velocity—are considered. This method combines the method of moments with nonlinear dynamic finite element analysis, which can enhance the efficiency of calculating failure probability.

1. Introduction

As structures crossing waterways, bridges not only bring huge economic profits to a country or local area, but also have a significant impact on the vessel transportation industry. Bridges around the world often collapse or are seriously damaged by impacts from vessels.

In order to avoid such catastrophic events, researchers and government officials around the world have conducted in-depth research into how to reduce the frequency vessel-bridge collisions.

2. Review of Vessel-Bridge Collision Reliability Analysis

Some economically developed countries have studied and published some guidelines and norms to guide their bridge design to deal with vessel-bridge collision problems. In 1987, according to Fujii's research on the collision of Japanese coastal strait vessels [1], COWI Consulting developed a method (Heinrich ratio method or probability statistics method) to estimate collapse probability. In 1994, AASHTO [2] used the collapse probability to describe the possible damage state of a bridge subjected to vessel collision and compiled the American "Guide Specification and

Commentary for Vessel Collision Design of Highway Bridges." In 1997, for the purpose of guiding the design of bridges for vessel collision, Eurocode 1 (Section 2.7) [3] was published, which stipulated the method of determining the design loads of impact and explosion accidents. It is worth noting that only the above two specifications have proposed a method to calculate the probability of collapse so far.

The AASHTO [2] on the ratio of pier strength to vessel collision force is shown in Figure 1. With $H/P = 1/10$ as the boundary, when the value of H/P tends to 0, the probability of collapse will be one and when the value of H/P tends to 1, the probability of collapse will be zero. This algorithm is undoubtedly convenient.

However, the damage mechanism of vessel-bridge collisions is not exactly the same as that of vessel collisions. Specifically, the collapse of a bridge is not only related to the ratio of vessel collision force to the lateral ultimate strength of bridge components, but also to the failure of the bridge. Therefore, there is a certain gap between the empirical formula in Figure 1 and the actual situation.

With the maturity and perfection of structural reliability calculation technology and the improvement of computer calculation speed, more and more studies have contributed to overcoming the limitations of the collapse probability calculation method in AASHTO [2] and proposed refined structural reliability calculation models.

FIGURE 1: Probability of collapse distribution.

Proske and Curbach (2005) and Geng (2007) calculated the failure probability of vessel-bridge collision based on FORM and SORM [4, 5]. Manuel (2006) addressed the probability of collapse considering impact angle, impact position, and deadweight tonnage of the vessel which treated them as discrete points [6]. Wang (2009) used the method of structural reliability to calculate the probability of vessel-bridge collision failure and proved the accuracy and effectiveness of this method [7]. Consolazio (2010) calculated the probability of vessel-bridge collision by Monte Carlo Method [8]. Fan (2011) proposed a simplified interaction model to efficiently evaluate the dynamic demand of bridge structure under vessel impact. In this method, ship motion is regarded as the motion of single degree of freedom, and ship-bow is modeled by a nonlinear spring element (only compression) connected to bridge structure. The nonlinear static relationship between impact force and crush depths of ship-bow is obtained by using a quasistatic method [9]. Zhu (2012) carried out different initial impact energy tests on the protective structures and on a single pile to study the energy transfer mechanism during a collision. A simplified energy-based analysis method was proposed to estimate the lateral deflection of the flexible pile-supported protective structures that are subjected to a given impact energy [10]. Through employing probabilistic descriptions for a multitude of random variables related to barge traffic characteristics and bridge structures, Davidson (2013) obtained the expression of the probability of collapse for bridge piers subject to barge impact loading in conjunction with nonlinear dynamic finite element analyses of barge-bridge collisions [11]. Shao (2014) compiled the response surface method reliability calculation program for vessel-bridge collision by VC++ language combined with ANSYS [12]. Zhang (2015) solved the reliability calculation problem of vessel-bridge collision by two methods with APDL language and compared the computing efficiencies of failure probability, which proved that the response surface method has a higher calculation efficiency

[13]. Zhou (2018) calculated the reliability index under different load component coefficients of accidental combination of vessel-bridge collision based on a Neural Network-Monte Carlo method [14]. Gholipour (2020) studied the progressive damage behavior and nonlinear failure mode of cable-stayed bridge piers subjected to vessel collision. A simplified two-degrees-of-freedom (2-DOF) model was proposed and was able to efficiently estimate the impact responses of the structure [15]. Guo (2020) established a finite element model that constructs a complex colliding system for a double-pylon cable-stayed bridge and a vessel to conduct nonlinear dynamic analysis. The impact forces, deformation of the pylon and pile foundation, and crush depths of the vessel were calculated [16]. Liu (2020) proposed a maneuverability-based analytical approach for evaluating the probability of a container vessel colliding with a channel bank under strong wind. The Monte Carlo simulation technique was applied to conduct numerous runs of the maneuvering simulation. These steady-state equations of vessel motion and the time-domain simulation method are combined in this approach to detect a vessel-bank collision [17]. Fan (2020) developed a novel fragility assessment framework for reinforced concrete (RC) bridges under vessel collision with the corrosion-induced structural deterioration being considered and further studied finite element (FE) modeling approaches considering the reinforcement bond-slip effects. The test results verify that bridges can be resilient to both episodic (vessel collision) and chronic (structural deterioration) hazards [18]. Lin (2021) proposed a framework for the performance assessment of reinforced concrete (RC) bridge piers under vehicle collision incorporating risk. The probabilities of collision under different scenarios were assessed by considering distance from the structural component to the road, angle of collision, and initial velocity, among others. Additionally, probabilistic structural demand and capacity models were developed considering different damage states within the evaluation process. Then, fragility contours of the investigated RC bridge were obtained [19]. Rong (2021) used Moran's I and Getis-Ord Gspatial autocorrelation method to determine whether near collisions display spatial clustering from a global and local perspective. The near-collision hotspot was associated with the local maritime traffic characteristics, such as the average vessel speed, speed dispersion, degree of speed acceleration, vessel route overlaps, and degree of angular deviation from vessel route centerline. A spatial correlation analysis method for near-collision clusters with local traffic characteristics was proposed [20]. Introducing the time factor, Peng (2021) provided an improved risk decision method for vessel-bridge collision based on AASHTO model, which can be applied to the risk assessment of vessel-bridge collisions during bridge design and service stages [21]. Based on the response surface theory and finite element (FE) model results, Fan (2021) established a typical four-span continuous reinforced concrete (RC) bridge surrogate model. The Monte Carlo method was used to efficiently generate the barge impact fragility surface, and the effects of corrosion and scouring on the impact fragility of bridge structures were studied [22].

The existing methods for bridge reliability assessment due to vessel impact, FORM and SORM, have deficiencies in design points and interactions, so they are not suitable for the case where the limit state equation is an implicit function. Furthermore, the direct or smart Monte Carlo simulation takes a lot of time when the probability of failure is very small.

3. Vessel-Bridge Collision Reliability Assessment

In this paper, a new method of vessel-bridge reliability assessment is proposed based on structural dynamic analysis. The probability models of influencing factors that affect vessel-bridge reliability are proposed. The failure probability is calculated based on the method of moments through calculating the dynamic response of the bridge due to vessel impact and the resistance of the bridge by finite element method. Figure 2 shows the process of the failure probability analysis.

3.1. The Limit State Equation of Pier. The pier foundation is composed of columns and piles. With respect to shear failure of the column, the limit state equation can be formulated as follows:

$$g = V_{R(\text{Column})} - V_{S(\text{Column})}(V, \text{DWT}, \theta), \quad (1)$$

where $V_{R(\text{Column})}$ is the shear resistance of the column. $V_{S(\text{Column})}$ is shear force response due to vessel impact. V, DWT , and θ represent impact velocity, deadweight tonnage of the vessel, and impact angle, respectively.

With respect to bending failure of the column, the limit state equation can be formulated as

$$g = \theta_{R(\text{Column})} - \theta_{S(\text{Column})}(V, \text{DWT}, \theta), \quad (2)$$

where $\theta_{R(\text{Column})}$ is the limit of rotating angle. $\theta_{S(\text{Column})}$ is rotating angle response due to vessel impact.

With respect to the pile, the limit state equation can be formulated as follows:

$$g = \theta_{R(\text{Pile})} - \theta_{S(\text{Pile})}(V, \text{DWT}, \theta). \quad (3)$$

The pile foundation can be treated as a series system and the limit state equation can be formulated as

$$g = \min\{\theta_{R(\text{Pile})} - \theta_{S(\text{Pile})}, \min \cdot \{V_{R(\text{Column})} - V_{S(\text{Column})}, \theta_{R(\text{Column})} - \theta_{S(\text{Column})}\}\}. \quad (4)$$

$\theta_{R(\text{Column})}$ and $\theta_{R(\text{Pile})}$ are formulated as

$$\theta \leq \theta_y + \phi_D \alpha \frac{\theta_{pu}}{K} \quad (5)$$

$$\theta_{pu} = (\phi_u - \phi_y) L_p,$$

where θ_y is yield rotating angle. ϕ_u is ultimate rotating angle. L_p is the length of the plastic hinge. ϕ_D is the reduction coefficient, which takes the value of 1.5. α takes the value of 0.6. K is a safety factor, which takes the value of 1.5.

FIGURE 2: Flowchart of failure probability analysis.

The limit state equation of the pile foundation is an implicit and nonlinear function. The dynamic response and resistance should be calculated by Finite Element Method. This is a problem of series system reliability analysis.

3.1.1. Dynamic Load Model of Vessel-Bridge Collision. The modified half-wave sinusoidal dynamic load model of vessel collision is computed as follows [23]:

$$F(t) = \frac{I}{T} \times \frac{\pi}{k} \times T^2 \left[\left(\frac{t}{T} - m \right)^2 + n^2 \right] \sin\left(\frac{\pi t}{T}\right) (0 < t < T), \quad (6)$$

where $F(t)$ is the time-history curve of vessel impact force. I is the impulse of vessel-bridge collision. T is the duration of the vessel-bridge collision.

k, I , and T are computed as follows:

$$\begin{aligned}
k &= 1 - \frac{4}{\pi^2} - 2m + 2m^2 + 2n^2, \\
I &= 1.39 \times 10^3 v \times \text{DWT}, \\
T &= 0.0936v^{0.25} \text{DWT}^{0.3}.
\end{aligned} \tag{7}$$

m is a uniform random variable with mean 0.49. n is a normal random variable with mean and variation coefficients of 0.36 and 0.28, respectively.

3.1.2. Probability Models of Random Variables. Impact angle, deadweight tonnage of the vessel, and impact velocity are considered to be important factors affecting the bridge failure probability.

The most reasonable method to determine the impact angle distribution is to obtain the statistics of the vessel-bridge collision accident data at the bridge location. Kunz has given the hypothetical distribution model of the impact angle [24]. This paper adopts the probability model of extreme type I distribution impact angle recommended by Geng [5]. In terms of parameter values, according to the actual engineering situation, the average impact angle of the bridge on the straight route can be taken as 10–15°, and the standard deviation can be taken as 4–6°. If the route is not straight, the mean value also needs to be added with the angle between the normal direction of the bridge shaft and the channel.

The deadweight tonnage of the vessel in each range is described by uniform distribution. According to the AASHTO bridge design specification of the United States, this paper uses the relationship curve in the form of broken lines to simulate the velocity distribution of yaw vessels. This model assumes that the reduction law of vessel speed decreases linearly from the channel edge to the distance of three times the length of the vessel. The maximum speed takes the typical speed of the vessel, and the minimum speed takes the average flow velocity. The probability model describing the impact velocity of the vessel can be expressed as follows:

$$V = \begin{cases} V_T, & x \leq x_c \\ \frac{x_L V_T - x_c V_{\min} - x(V_T - V_{\min})}{x_L - x_c}, & x_c < x \leq x_L \\ V_{\min} & x > x_L \end{cases} \tag{8}$$

where V represents the design impact velocity, V_T represents the typical navigation velocity of vessels in the channel, $V_{\min}$ represents the minimum impact velocity (not less than the annual average flow velocity), x represents the distance between the vessel and the pier, x_c represents the distance between the vessel and the edge of the channel, and x_L represents the distance of three times the length of the vessel from the centerline of the vessel channel.

Geng [5] concluded that the vessel speed can be described by normal distribution. Therefore, in the above

formula, this paper uses normal distribution to describe V_T and $V_{\min}$. While $x \leq x_c$ or $x > x_L$, the mean of V is equal to the mean of V_T , and the standard deviation of V is equal to the standard deviation of $V_{\min}$. At that time, the average impact velocity can be calculated by equation (8), and the standard deviation formula is

$$\sigma_V = \frac{1}{x_L - x_c} \sqrt{\sigma_{V_{\min}}^2 (x - x_c)^2 + \sigma_{V_T}^2 (x_L - x)^2}. \tag{9}$$

3.2. Method of Moments for Vessel-Bridge Collision Reliability Assessment. Iteration and correlation coefficient among failure modes are not necessary for calculating the failure probability by the method of moments, and thus it is convenient for application to structural system reliability analysis.

3.2.1. Point Estimates of Statistical Moments. The failure of probability is related to the location and shapes of the distribution, and obviously to the first few moments of the performance function.

The first few statistical moments of the performance function can be expressed as the following integrals:

$$\mu_Y = \int g(x) f(x) dx, \tag{10a}$$

$$\sigma_Y^2 = \int (g(x) - \mu_Y)^2 f(x) dx, \tag{10b}$$

$$\sigma_Y^k \alpha_{kY} = \int (g(x) - \mu_Y)^k f(x) dx, \tag{10c}$$

where $g(x)$ is performance function, μ_Y and σ_Y are the mean value and standard deviation of $g(x)$, and α_{kY} is the k th-order dimensionless central moment. $f(x)$ is the joint probability density function of basic random variables.

The point estimates method [25] is used for calculating the first few moments of performance function. Any random variables can be transformed into standard normal random variables through Rosenblatt transformation:

$$U = T(X), \tag{11}$$

$$X = T^{-1}(U). \tag{12}$$

Using the inverse transformation, equations (10a–12) can be rewritten as follows:

$$\mu_Y = \int g[T^{-1}(u)] \phi(u) du, \tag{13a}$$

$$\sigma_Y^2 = \int (g[T^{-1}(u)] - \mu_Y)^2 \phi(u) du, \tag{13b}$$

$$\sigma_Y^k \alpha_{kY} = \int (g[T^{-1}(u)] - \mu_Y)^k \phi(u) du \quad k > 2, \tag{13c}$$

where ϕ is the probability density function of standard normal random variables.

By obtaining the estimating points $u_1, u_2, \dots, u_m$ and their corresponding weighs $Pu_1, Pu_2, \dots, Pu_m$, the k th central moment of Y can be calculated as follows:

$$\mu_Y = \sum_{j=1}^m P_{u_j} g[T^{-1}(u_j)], \quad (14a)$$

$$\sigma_Y^2 = \sum_{j=1}^m P_{u_j} (g[T^{-1}(u_j)] - \mu_Y)^2, \quad (14b)$$

$$\sigma_Y^k \alpha_{kY} = \sum_{j=1}^m P_{u_j} (g[T^{-1}(u_j)] - \mu_Y)^k, \quad k > 2, \quad (14c)$$

where $T^{-1}(U)$ is the inverse Rosenblatt transformation, which is generally expressed as follows:

$$x_j = T^{-1}(u_j) = F_X^{-1}[\Phi(u_j)]. \quad (15)$$

The performance function $g(x)$ with multiple variables can be approximated by the following function [26, 27]:

$$g^*(x) = \sum_{i=1}^n (g_i - g_\mu) + g_\mu, \quad (16)$$

where $g_\mu = g(\mu)$, $g_i = g[T^{-1}(U_i)]$, and $U_i = [u_{\mu_1}, u_{\mu_2}, \dots, u_{\mu_{i-1}}, u_{\mu_i}, u_{\mu_{i+1}}, \dots, u_{\mu_n}]^T$. $\mu = [\mu_1, \mu_2, \dots, \mu_n]^T$ represents the vector in which all the random variables take their mean values. u_μ is the k th value of u_μ , which is the vector in the standard normal space corresponding to μ ($k = 1, 2, \dots, i-1, i+1, \dots, n$). g_μ is a constant and g_i is a function of only u_i . T^{-1} is the inverse Rosenblatt transformation.

The first four moments of $g^*(x)$ can be expressed as follows:

$$\mu_g = \sum_{i=1}^n (\mu_i - g_\mu) + g_\mu, \quad (17a)$$

$$\sigma_g^2 = \sum_{i=1}^n \sigma_i^2, \quad (17b)$$

$$\alpha_{3g} \sigma_g^3 = \sum_{i=1}^n \alpha_{3i} \sigma_i^3, \quad (17c)$$

$$\alpha_{4g} \sigma_g^4 = \sum_{i=1}^n \alpha_{4i} \sigma_i^4 + 6 \sum_{i=1}^{n-1} \sum_{j>i}^n \sigma_i^2 \sigma_j^2, \quad (17d)$$

where u_i and σ_i are the mean value and standard deviation of g_i . α_{3i} and α_{4i} are the third and fourth dimensionless central moments.

3.2.2. The Fourth-Moment Reliability Method on the Basis of the Pearson System. For the standardized variable Z_s , the probability density function of Z_s satisfies the following differential equation in the Pearson system [28]:

$$\frac{1}{f} \frac{df}{dZ_s} = -\frac{aZ_s + b}{c + bZ_s + dZ_s^2}, \quad (18)$$

where parameters a, b, c , and d can be expressed as follows:

$$a = 10\alpha_{4G} - 12\alpha_{3G} - 18, \quad (19a)$$

$$b = \alpha_{3G}(\alpha_{4G} + 3), \quad (19b)$$

$$c = 4\alpha_{4G} - 3\alpha_{3G}^2, \quad (19c)$$

$$d = 2\alpha_{4G} - 3\alpha_{3G}^2 - 6. \quad (19d)$$

The reliability index based on the fourth-moment method is given as follows:

$$\beta_{4M-P} = -\Phi^{-1} \left[\int_{-\infty}^{-\beta_{2M}} f(z_s) dz_s \right]. \quad (20)$$

3.3. Application in Practical Engineering. A detailed example of vessel-bridge collision reliability analysis is presented. Figure 3 shows the general view of a cable-stayed bridge.

The cable-stayed bridge is a 1430 m long with a span arrangement of 110 m + 240 m + 730 m + 240 m + 110 m. The main beam adopts a continuous steel box girder. The main tower adopts an inverted Y-shaped reinforced concrete cable tower with the height of 203.17 m. The upper column of 58.5 m is the anchor cable area of the steel skeleton section, the middle column of 90 meters is the concrete skeleton section, and the lower part of the main tower is a variable cross section of hollow box. There are 60 bored cast-in-place piles with variable diameters of 2.5–3.0 m under the main tower cap, and the pile length is 104 m. The cap thickness is 5 m. The auxiliary pier and transition pier all adopt separated thin-walled box hollow piers. Transition pier height is 43.50 m, and the auxiliary pier height is about 46.68 m. The auxiliary pier and transition pier foundation, respectively, use 18 and 12 bored cast-in-place piles with variable diameters of 2.5–3.0 m. The length of the piles is 80 m.

According to observed data, the values of the parameters for the reliability analysis are listed in Table 1. The 5 estimating points in the standardized normal space are listed in Table 2. The cases of each pier under different deadweight tonnage (DWT) of vessels can be obtained. Each estimating point stands for one analysis case. So, there are 5 cases according to each range of DWT of vessels.

Due to the word limit, only part of the cases of the main tower are listed. The calculation process of the auxiliary pier and the transition pier is similar to that for the main tower. According to the probability distribution of vessel impact velocity, the mean and coefficient of variation of vessel impact velocity at different DTW of vessels are shown in Table 3. The DTW of vessels in each range and impact velocity can be obtained by ROSENBLATT inverse transformation. Only the cases of 0DWT-5000DWT, 25000 DWT-30000DWT, and 45000DWT-50000DWT are listed in Table 4. According to the above analysis, the impact force time-history curves of vessel-bridge collision can be obtained. The typical impact force time-history curves are shown in Figures 4 and 5.

FIGURE 3: General view of a cable-stayed bridge.

TABLE 1: The value of analysis parameters.

Parameter	Distribution type	Mean	Variation coefficient
Typical vessel transit velocity	Normal distribution	5.8 m/s	0.25
Minimum design impact velocity	Normal distribution	1.2 m/s	0.25

TABLE 2: Estimating points in the standardized normal space.

1	2	3	4	5
-2.85697001	-1.35562618	0	1.35562618	2.85697001

TABLE 3: The value of mean velocity and variation coefficient when vessels impact the main tower.

Tonnage of vessel	0–5000	5000–10000	10000–15000	15000–20000	20000–25000
Mean	2.582	4.283	4.654	4.834	4.945
Variation coefficient	0.190	0.187	0.198	0.204	0.209
Tonnage of vessel	25000–30000	30000–35000	35000–40000	40000–45000	45000–50000
Mean	5.022	5.080	5.125	5.162	5.193
Variation coefficient	0.212	0.214	0.216	0.218	0.219

TABLE 4: Different analysis cases of vessels impacting the main tower.

DWT	0–5000		25000–30000		45000–50000	
	Tonnage of vessel	Vessel impact velocity (m/s)	Tonnage of vessel	Vessel impact velocity (m/s)	Tonnage of vessel	Vessel impact velocity (m/s)
Case 1	2500	2.581604	27500	5.02228	47500	5.19268
Case 2	2500	1.17768	27500	1.98055	47500	1.93648
Case 3	2500	1.91544	27500	3.57899	47500	3.64762
Case 4	2500	2.581604	27500	5.02228	47500	5.19268
Case 5	2500	3.24777	27500	6.46557	47500	6.73775

FIGURE 4: Time-history curve of vessel collision load at DWT = 27500, $v = 5.02$ m/s.

FIGURE 5: Time-history curve of vessel collision load at DWT = 42500, $v = 5.16$ m/s.

TABLE 5: Boundary and connection conditions of main bridge.

	X	Y	Z	XX	YY	ZZ
Tower-beam junction	0	1	1	1	0	0
Auxiliary pier-beam junction	0	1	1	1	0	0
Transition pier-beam junction	0	1	1	1	0	0

Note. X, Y, and Z are translational degrees of freedom along, across, and vertical bridge, respectively. XX, YY, and ZZ are the rotational degrees of freedom along, across, and vertical bridge, respectively. 1 represents constraint; 0 represents freedom.

FIGURE 6: The finite element model of the cable-stayed bridge.

As for calculation of the dynamic response of the bridge due to vessel impact, the finite element model of the bridge is set up with the plastic hinges at the top of each pile. Bridge structures (including the main tower, main beam, and pier as well as pile foundation) are simulated by spatial beam element. The stayed cable is simulated by a truss element. The cable is connected to the main beam by a rigid link. The boundary and connection conditions of the bridge are shown in Table 5.

Figure 6 shows the finite element model of the cable-stayed bridge. Tables 6 and 7 show the finite element analysis results for the main tower. The finite element analyses of the auxiliary pier and transition pier are not listed in this paper.

The stress-strain relationship of steel is bilinear. The Mander model is used to describe the constitutive equation of concrete. The moment-curvature analysis is carried out on the top section of each pile. Then, the $\theta_{R(\text{Pile})}$ is calculated as 0.0167 Rad.

TABLE 6: Dynamic response of main tower (Rad).

DWT	0–5000	5000–10000	10000–15000	15000–20000	20000–25000
Case 1	$4.70E-05$	$1.47E-04$	$2.11E-04$	$2.64E-04$	$3.11E-04$
Case 2	$6.76E-05$	$2.08E-04$	$3.10E-04$	$3.97E-04$	$4.74E-04$
Case 3	$8.42E-05$	$2.58E-04$	$3.89E-04$	$5.02E-04$	$6.03E-04$
Case 4	$9.89E-05$	$3.05E-04$	$4.62E-04$	$5.99E-04$	$7.21E-04$
Case 5	$1.16E-04$	$3.54E-04$	$5.39E-04$	$6.99E-04$	$8.43E-04$

TABLE 7: Dynamic response of main tower (Rad).

DWT	25000–30000	30000–35000	35000–40000	40000–45000	45000–50000
Case 1	$3.56E-04$	$3.96E-04$	$4.34E-04$	$4.70E-04$	$5.05E-04$
Case 2	$5.46E-04$	$6.1E-04$	$6.76E-04$	$7.35E-04$	$7.92E-04$
Case 3	$6.97E-04$	$7.83E-04$	$8.64E-04$	$9.41E-04$	$1.02E-03$
Case 4	$8.33E-04$	$9.38E-04$	$1.04E-03$	$1.13E-03$	$1.22E-03$
Case 5	$9.73E-04$	$1.10E-03$	$1.21E-03$	$1.32E-03$	$1.75E-03$

TABLE 8: The failure probability of each pier.

DWT	0–5000	5000–10000	10000–15000	15000–20000	20000–25000
Main tower	$0.00E+00$	$0.00E+00$	$0.00E+00$	$0.00E+00$	$0.00E+00$
Auxiliary pier	$0.00E+00$	$0.00E+00$	$0.00E+00$	$0.00E+00$	$0.00E+00$
Transition pier	$0.00E+00$	$0.00E+00$	$0.00E+00$	$0.00E+00$	$0.00E+00$

Note. The failure probability takes the value of 0 when it is smaller than $1.0E-30$.

TABLE 9: The failure probability of each pier.

DWT	25000–30000	30000–35000	35000–40000	40000–45000	45000–50000
Main tower	$0.00E+00$	$0.00E+00$	$0.00E+00$	0	$7.95E-27$
Auxiliary pier	$0.00E+00$	$3.66E-13$	$3.49E-12$	$8.68E-09$	$3.75E-07$
Transition pier	$3.53E-16$	$2.91E-10$	$1.97E-06$	$9.20E-06$	$7.05E-03$

Note. The failure probability takes the value of 0 when it is smaller than $1.0E-30$.

Tables 8 and 9 show the failure probability of each pier.

Using dimension reduction integration (DRI) with five estimating points, the failure probability of the main tower, auxiliary pier, and transition pier is given as 7.95×10^{-27} , 3.84×10^{-7} , and 7.06×10^{-3} . It can be seen from the results that the failure probability of the main tower due to vessel collision is small for its strong antivessel collision ability. Only the large DWT of vessels with 45000DWT–50000DWT can lead to the failure of the main bridge tower. The failure probability of vessel-bridge collision caused by vessels with DWT below 45000t is negligible and ignorable. The failure probability of the transition pier is 7.06×10^{-3} due to its weak anticollision ability. Therefore, reasonable active and passive anticollision measures should be taken. Navigation management of vessels should be strengthened so as to reduce the risk of vessel yaw and to avoid large vessels hitting the auxiliary pier directly.

4. Conclusions

A new method of vessel-bridge collision reliability analysis is proposed based on dynamic analysis. The probability models of influencing factors are presented for vessel-bridge collision reliability analysis. A mathematical model of reliability analysis is proposed. Compared with the AASHTO method, this method can calculate the reliability of vessel-bridge

collision accurately with consideration of the random characteristics of impact angle, impact velocity, and dead-weight tonnage of vessels.

This method combines the method of moments with nonlinear dynamic finite element analysis, which needs little calculation time to obtain failure probability. This method enhances the efficiency of calculating failure probability compared with Monte Carlo simulation, and is especially applicable to implicit performance function.

Data Availability

The underlying data used to support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that they have no conflicts of interest regarding the publication of this study.

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