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RELIABILITY EVALUATION OF BRIDGES THE RESPONSE SURFACE METHOD AND THE BAYESIAN MODIFICATION METHOD

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Abstract

To evaluate the performance of in-service bridges, the Bayesian theory was employed to evaluate the reliability of the bridges. According to the Bayesian prediction principle, a Bayesian modified parameter distribution model was established. As an example, the compressive strength of the concrete was modified. Based on the response surface method, the ultimate state equation of the bridge structure was established. The bearing capacity of the bridge was taken as the resistance. The self-weight of the bridge and the load of the vehicle were taken as the load effect. The parameters of the ultimate state equation were ascertained using the fitting results of the finite element model. After combining the response surface method and the Bayesian modification method, a bridge reliability evaluation method was proposed. The experimental verification was carried out to verify the feasibility and accuracy of the method. The results indicated that the proposed bridge reliability evaluation method was feasible for use in practical engineering. In addition, the problem evaluation method could provide guidance for the formulation of a maintenance strategy.

Key Words

Bridge, reliability evaluation, ultimate state, response surface method, Bayesian modification method

1. Introduction

In recent years, the number of bridges has grown rapidly worldwide. With the construction of these new bridges, the bridges that have served for decades have entered the period of concentrated defect exposure [1]–[3]. The bearing capacity of an in-service bridge decreases with the increase of the service period [4]. This capacity decrease is caused by many factors, such as concrete deterioration and steel corrosion. Thus, it is necessary to accurately evaluate and analyse in-service bridge structures. The aim of the evaluation and analysis was to ensure the bearing capacity of the bridge.

Researchers have conducted in-depth research studies on structural component evaluation standards and structural evaluation methods [5]. The proposed load test method, the design-based method, and the reliability evaluation method have been put forward [6]–[8]. Among these evaluation methods, the reliability evaluation method has been widely used. The reliability evaluation method has two advantages: the first is its advanced theoretical foundation. The second is its ability to accurately reflect the actual performance of a bridge. The research studies on the reliability evaluation method have mainly focused on the influencing factors and the time-varying effects [9]–[11]. At present, the reliability evaluation method is accomplished by two steps [12]–[14]. The first step is to establish the ultimate stage equation of a bridge based on the prior distribution of the parameters. The second step is to calculate the reliability to evaluate the bridge. However, the application of this method has been limited because the prior distribution of parameters cannot accurately reflect the actual situation of in-service bridges [15].

To accurately evaluate and predict the reliability of a bridge, the probability distribution model of the bridge's structural resistance and its load effect should be modified, and the modification should be conducted in real time. Since the Bayesian theory has been proposed, scholars have employed the Bayesian theory to evaluate bridges. The use of the Bayesian theory in bridge construction includes load model updating, the prior probability updating of random

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variables, concrete bridge life prediction, and the time-varying reliability correction of bridge members [16]–[18]. However, there have only been a few studies focused on the modification of the probability distribution of structural parameters based on the Bayesian theory [19], [20].

In view of this, the response surface method and the Bayesian modification method were employed to evaluate the reliability of bridges in this study. A bridge reliability evaluation method based on the response surface method and the Bayesian modification method was proposed. To verify the feasibility of the method, the experimental verification was conducted. In the experiment, the concrete strength was taken as the modification parameter. The probability distribution model of the concrete was modified using the Bayesian modification method, and the ultimate state equation was calculated with the response surface method [21]. Then, the reliability of the specimen was obtained. The results of the experiment indicated that the proposed method was feasible for evaluating the bridge with high accuracy. The evaluation result was consistent with the actual performance of the structure. Thus, the proposed method could be used in practical engineering to provide guidance for the development of maintenance strategies.

2. Methods

2.1 Reliability-based Evaluation Method

The reliability of a bridge structure refers to its ability to maintain the default function under specific conditions. According to the characteristics of the bridge's reliability, the ultimate stage equation of the bridge could be expressed as shown in the following equation:

$$\begin{aligned} Z = {} & R\left(X_1, \ldots, X_i, \ldots, X_n\right) - S_G\left(X_1, \ldots, X_n\right) \\ & - S_Q\left(X_1, \ldots, X_n\right) \end{aligned} \tag{1}$$

where Z is the functional function of the bridge, R is the structural load carrying capacity, S_G is the constant load effect, and S_Q is the live load effect. X_i includes the structural geometric parameters, material properties, and various factors that affected the performance of the bridge. When Z was greater than zero, the structure was in a safe state. When Z was equal to zero, the structure was in a critical state. When Z was less than zero, the structure was in a dangerous state.

The reliability of the structure was represented by the reliability index β. The reliability index was defined by (2), where μ_z is the mean value of the ultimate state, and σ_z is the standard deviation of the ultimate state:

$$\beta = \frac{\mu_Z}{\sigma_Z} \tag{2}$$

2.2 Bayesian Modification Method

According to the Bayesian theory, the prior probability distribution of the parameters could be modified by combining it with the real-time detection data. Then, the posterior probability distribution of the parameters could be obtained. The Bayesian modification for continuous random variables was defined as shown in (3), where $g_P(P = p)$ is the prior probability distribution of the random variable P,$f_{x|P}(x|P = p)$ is the conditional distribution of the observation factor x when $V = v$, and $f^*_{x|P}(P = p|x)$ is the posterior probability distribution of P:

$$f^*_{x|P}(P = p|x) = \frac{f_{x|P}(x|P = p)\mathrm{g}_P(P = p)}{\int_{-\infty}^{\infty} f_{x|P}(x|P = p)\mathrm{g}_P(P = p)dP} \tag{3}$$

The posterior probability distribution was the modified probability distribution of P.

For the bridge structure, the unit properties of the structure during the construction process could be obtained *via* theoretical derivation, experiment, and so on. If P was a parameter of the structure, after the prior probability distribution and the actual test results were obtained, the posterior probability distribution could be ascertained with the Bayesian modification method.

3. Distribution Model

3.1 Distribution of the Structural Resistance

To confirm the resistance of the structure, the uncertainties of the structural resistance had to be considered. The uncertainties of the structural resistance mainly included the uncertainty of the material properties, the uncertainty of the geometric parameters of the components, and the uncertainty of the resistance calculation model.

The uncertainty of the material property could be expressed as shown in the following equation:

$$K_M = \frac{f_C}{f_K} \tag{4}$$

where K_M is the uncertainty of the material property, f_C is the actual value of the material properties of the bridge, and f_K is the standard value of the material properties of the bridge.

The uncertainty of the geometric parameter of the component could be represented by the variable K_A, as shown in the following equation:

$$K_A = \frac{a}{a_K} \tag{5}$$

where a is the geometric parameter of the actual structural component, and a_K is the standard value of the geometric parameter of the structural component.

The uncertainty of the resistance calculation model was represented by the variable K_P, as shown in (6), where K_S is the actual resistance value of the structural component, and K_j is the resistance value calculated according to the specification:

$$K_P = \frac{K_S}{K_j} \tag{6}$$

3.2 Distribution Model of the Load

The constant load of the bridge was controlled by the self-weight of the bridge. To ensure that the statistical analysis results could be used for different bridges, the ratio of the measured constant load capacity and the standard load capacity was calculated. The ratio was used as the statistical variable K_r. The ratio could be expressed as shown in (7), where r is the measured bulk density of the structure, and r_K is the standard structural bulk density:

$$K_r = \frac{r}{r_K} \tag{7}$$

The live load of the bridge was controlled by the vehicle load. Thus, the probability distribution model of the vehicle load was established and the statistical variable was ascertained. As shown in (8), the dimensionless coefficient K_{SQ} was used to represent the uncertainty of the vehicle load effect. In (8), S_Q is the actual vehicle load and S_{QK} is the standard value of the vehicle load. When determining the vehicle load effect, the probability distribution of the bending moment effect had to be taken in account, and the extreme value Gumbel I type distribution model had to be considered to ensure the safety of the structure.

$$K_{SQ} = \frac{S_Q}{S_{QK}} \tag{8}$$

3.3 Modified Distribution Model of the Concrete Compressive Strength

With the increase of the service time of the bridge, the performance of the bridge would decrease. Thus, the resistance of the components had to be modified when analysing the reliability of the bridge. By taking the concrete compressive strength as the modified parameter, (3) could be rewritten as shown in (9). In the following equation, f_c is the concrete compressive strength, $f_c(x)$ is the modified distribution of the concrete compressive strength when $g(\theta)$ is taken as the original material characteristic distribution. $f(x|\theta)$ is the distribution of the concrete compressive strength under the condition of original material property distribution. As shown in (9), the modified probability distribution of concrete compressive strength could be obtained:

$$f_c(x) = \frac{f_{x|\theta}(x|\theta)\mathrm{g}(\theta)}{\int_{-\infty}^{\infty} f_{x|\theta}(x|\theta)\mathrm{g}(\theta)d\theta} \tag{9}$$

The modified probability distribution could be used to process the reliability evaluation for the bridge.

4. Establishment of the Ultimate State Equation

The methods of establishing the ultimate state equation mainly included the Monte Carlo method and the response surface method. Because the bridge structure was more complex than other normal structures, the ultimate state equation of the bridge is difficult to obtain. In view of the complexity of the bridge structure, the response surface method was used to calculate the reliability index of the bridge. The use of the response surface method to establish the ultimate state equation included two parts: the parameter design and the coefficient fitting.

4.1 Parameter Design

In the parametric design, an analysis of the parameter sensitivity was conducted to figure out the sensitive parameters. The parameters that had a significant influence on the structural bearing capacity were input in the ultimate state equation. As shown in (10), the sensitivity of the structural parameter was expressed as the partial derivative of the structural response to the input parameter:

$$\theta_i = \frac{\partial R}{\partial X_i} \tag{10}$$

where θ_i is the sensitivity of the parameter, R is the resistance of the bridge, and X_i is the parameter.

4.2 Coefficient Fitting

By assuming that the parameters that had a significant influence on the structural carrying capacity were $X_i (i = 1 : n)$, the response surface function (ultimate state equation) could be constructed. The function in the form of a quadratic polynomial was constructed as shown in (11), where a, a_i, and a_{ij} are the pending coefficients:

$$Y = f(X_1, X_2, \ldots, X_n) = a_0 + \sum_{i=1}^{n} a_i X_i + \sum_{i=1}^{n}\sum_{j=1}^{n} a_{ij} X_i X_j \tag{11}$$

The central composite design method was used to select the k test levels X_{ik} of the X_i variable. The total number of the test points was k^n. After calculating the value of the response surface function at each text point, the coefficients of the function were ascertained. To obtain the pending coefficients of the response surface function, the fitting method based on the least-squares was employed. The consistencies of the fitting result of the response surface function and the test results were examined. If the fitting result was accepted, the function could be used as an approximate expression of the actual response of the bridge.

After the ultimate state equation was established with the response surface method, the first- or the second-moment method could be used to calculate the reliability index of the bridge structure. Then the performance of the bridge could be evaluated.

5. Experimental Verification

5.1 Specimen and Finite Element Model

A simple support beam was designed and produced. The specimen was produced to verify the feasibility of the proposed reliability evaluation method based on the response

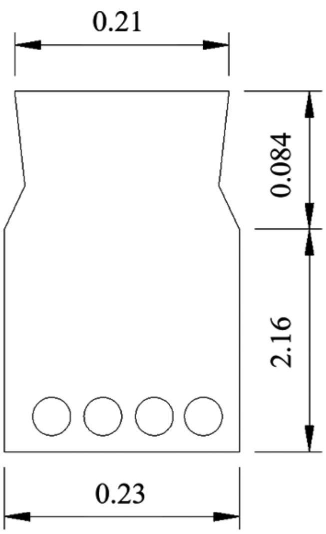

Figure 1. Section diagram of the simple support beam (units: m).

surface method and the Bayesian modification method. The span of the specimen was 5 m, the width of the specimen was 0.23 m, the height of the specimen was 0.3 m, and the effective height of the specimen was 0.26 m. The deck pavement of the specimen was made of C40 concrete and the thickness of the pavement was 0.05 m. In addition, the yield strength of the steel bars was 280 MPa. The sectional area of the steel bars was 804 mm^2. The section of the simple support beam specimen is shown in Fig. 1.

Based on the finite element software ABAQUS, the finite element model of the specimen was established. The model was established to calculate the ultimate bearing capacity of the specimen under different conditions. The section of the main beam, the length of the main beam, and the length direction of the steel bars were meshed with the spacing of 60 mm. The stress–strain relationship of the concrete was obtained from an actual inspection. The stress–strain relationship was input into the finite element model. To accurately simulate the effects of the steel (within the concrete), the embedded constraints were added between the concrete and the steel bars. In addition, the design value of each parameter of the specimen was taken as the input value. This model was taken as the basic model of the specimen. The vertical deflection cloud map of the basic model with a 94.5-kN mid-span load is shown in Fig. 2.

5.2 Ultimate State Equation

To determine the key parameters for the structural reliability analysis, several parameters were selected as random variables according to engineering experience. The parameters included the concrete compressive strength f_c, the yield strength of the steel bar f_y, the height of the specimen h, the width of the specimen d, and the sectional area of the steel bar A_s. The probability distribution characteristics of each parameter are shown in Table 1. As shown in Table 1, Ω1 is the height change ratio of the specimen, Ω2 is the width change ratio of the specimen, and Ω3 is the sectional area change ratio of the steel bar.

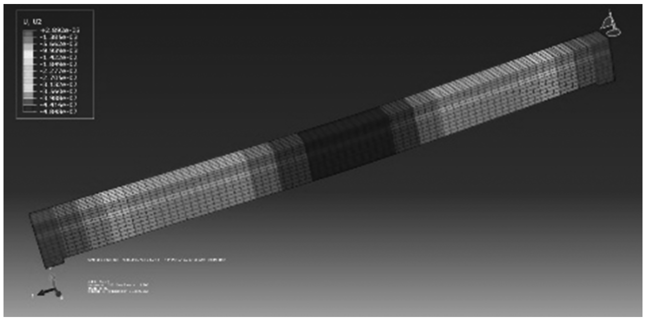

Figure 2. Vertical deflection diagram of the basic model.

Table 1
Probability Distribution Characteristics of the Parameters

Parameter	Distribution Pattern	Mean Value	Coefficient of Variation	Unit
f_c	Normal distribution	26.8	0.1221	MPa
f_y	Normal distribution	380	0.0685	MPa
Ω1	Normal distribution	1.0064	0.0255	/
Ω2	Normal distribution	1.0013	0.0081	/
Ω3	Normal distribution	1.0000	0.0350	/

The purpose of the parametric sensitivity analysis was to determine the sensitivity of the structural carrying capacity to different input parameter changes. Because the variation ranges of different structural parameters were different, the variation range of each parameter was determined by the mean value and the standard deviation (σ). For each parameter, the mean value was set as the basic value and multiplication factor of the standard deviation were added or subtracted. The magnitudes of the change were 0, $\pm\sigma$, $\pm 1.5\sigma$, $\pm 2\sigma$, and $\pm 3\sigma$. According to the above-mentioned parameter design method, the ultimate bearing capacities of the specimen under different working conditions were calculated. The calculation result is shown in Fig. 3. The sensitivity index of each parameter was calculated according to (10). The result is shown in Table 2.

As shown in Fig. 3, the bearing capacity of the bridge was positively correlated with several parameters. These parameters included the compressive strength of the concrete strength, the tensile strength of the steel bar, the height of the specimen, the sectional area of the steel bar, and the width of the specimen. The result was

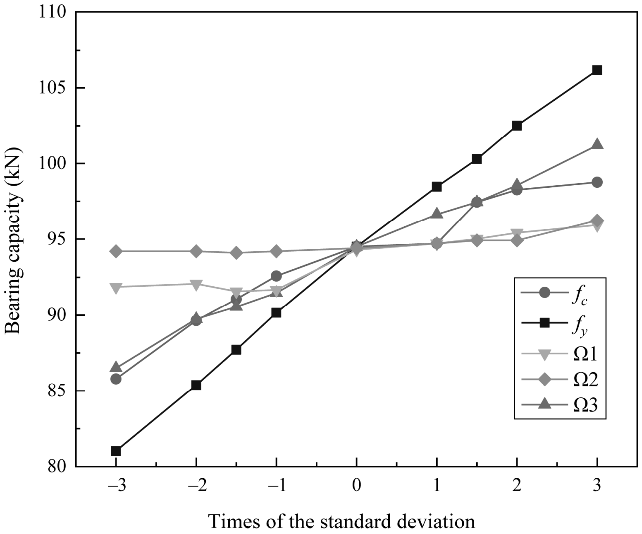

Figure 3. Ultimate bearing capacity under different working conditions.

Table 2
Parametric Sensitivity to the Ultimate Bearing Capacity

Parameter	f_c	f_y	A_S	h	d
θ_i	0.6391	0.1676	0.3405	1.0010	1.0242
$\lvert\theta_i - 1\rvert$	0.3609	0.8324	0.6595	0.0010	0.0242

in agreement with the theoretical result. As shown in Table 2, the width and the height of the specimen had little effect on the structural bearing capacity, while the concrete compressive strength, the yield strength of the steel bar, and the sectional area of the steel bar had a significant effect on the structural bearing capacity. Thus, the concrete compressive strength, the yield strength of the steel bar, and the sentimental area of the steel bar were taken as the input parameters of the ultimate state equation.

The values of each parameter were designed using the central composite rotation method. The ultimate bearing capacity under each working condition was calculated using the established finite element model. The response surface function was fitted based on the least-squares method, and the response surface function was fitted. The ultimate state function is shown in the following equation:

$$\begin{aligned} R = & -156.323 + 0.28775 f_y + 1.39 A_s - 1.417 f_c \\ & -2.4975 \times 10^{-16} f_y A_s + 0.0037835 f_y f_c \\ & +0.009685 A_s f_c - 0.00029 f_y^2 \\ & -0.00325 A_s^2 - 0.02453 f_c^2 \end{aligned} \tag{12}$$

where the units of f_y are MPa, the units of A_s are mm^2, and the units of f_c are MPa.

The fitting verification of the obtained response surface function is shown in Fig. 4. As shown in the same figure, the response surface fitting result agreed well with the result of the finite element analysis. This indicated that the response surface could be correctly obtained with the fitting. Then the next reliability analysis was performed.

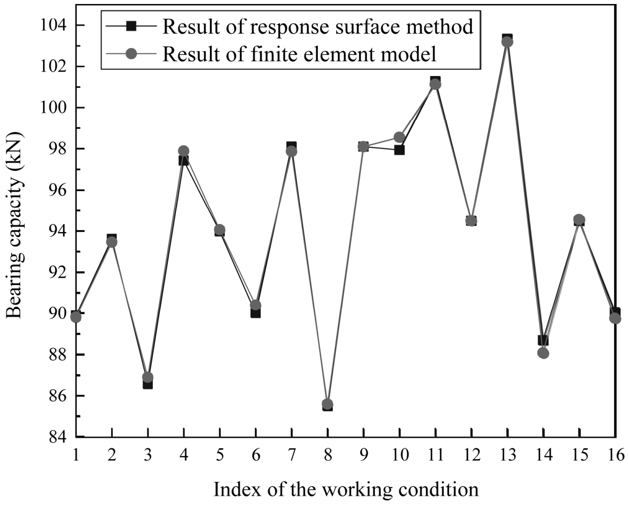

Figure 4. Correlation between the results of the finite element analysis and the predicted value using the response surface method.

5.3 Bayesian Modified Reliability Assessment

To modify the probability distribution model of the concrete compressive strength, 10 test areas were selected on the specimen. The concrete compressive strength of each test area was measured using the rebound method. After the distribution model was obtained by regression analysis, the distribution model was converted into the distribution model expressed by the standard value of the axial compressive strength. The aim of this conversion was to ensure the distribution's comparability to the input parameters of the finite element model. The converted distribution model was $g(x) \sim N(26.8, 3.3)$. According to (9), the concrete compressive strength was modified. As the corrected distribution was non-normal, the normalized deformation was performed, and the normalized result was $f_c(x) \sim N(27.98, 2.23)$.

Taking the self-weight of the simple support beam specimen as the constant load, the distribution of the constant load effect of the specimen was $MSG \sim N(4.703, 0.203^2)$. According to the design data for the specimen, the design live load of the specimen was $M = 23.18$ kN·m. The live load could be converted into the equal mid-concentration force $F = 19.6$ kN. The live load effect of the specimen was subjected to an extreme value Gumbel I type distribution with a mean value of 15.67 kN and a standard deviation of 1.35. Combined with the fitted ultimate state equation of the bearing capacity of the bridge and the distribution model of the load effect, the function of the bridge could be expressed as shown in (13):

$$Z = R(f_y, A_s, f_c) - S_G - S_Q \tag{13}$$

Table 3
Probability Distribution Characteristics of the Major Parameters

Parameter		Distributed	Mean Value	Standard Deviation	Coefficient of Variation
F_y (MPa)		Normal distribution	280	26.6	0.0685
A_S (mm^2)		Normal distribution	201.1	7.04	0.035
f_c (MPa)	Before Bayesian modification	Normal distribution	26.8	3.3	0.1221
	After Bayesian modification	Equivalent normalization distribution	27.98	2.23	0.0797
Constant load effect (kN)		Normal distribution	4.703	0.203	0.0431
Live load effect (kN)		Extreme value Gumbel I type distribution	15.67	1.35	0.0862

Table 4
Reliability Index of the Different Calculation Methods

Ultimate State Equation Calculation Method	Response Surface Method		Monte Carlo Method	
Bayesian Modification	Before Modification	After Modification	Before Modification	After Modification
Reliability index	9.703	9.834	8.13	8.36
Target reliability index	3.7			

The probability distribution characteristics of each random variable are shown in Table 3.

Based on the distribution characteristics, the software program Matlab was employed to calculate the reliability index using the second-order method. The modified reliability index was also obtained. To further analyse the feasibility of the response surface method, the Monte Carlo method was also used to obtain the ultimate state equation of the specimen. The reliability indexes were calculated before and after modification. The results are shown in Table 4.

As shown in Table 4, compared with the reliability index that was obtained with the Monte Carlo method, the reliability index that was obtained with the response surface method was higher. The first reason for this was that the ultimate state equation that was obtained with the response surface method was more refined. The second reason was that the response surface method fully utilized the potential bearing capacity of the bridge. Thus, the evaluation results of the response surface method were closer to the actual situation of the bridge. In addition, the reliability index of the specimen increased after the modification based on the Bayesian modification method. The reliability index increased because the actual measurement data of the specimen was used to process the evaluation. In addition, the evaluation result was more consistent with the actual situation of the bridge. Because the design parameters of the specimen were conservative, the calculated reliability index was greater than the target reliability, and the redundancy value was large. In the actual engineering, the actual condition of the bridge structure was close to the target reliability index. It was necessary to use the Bayesian modification-based bridge evaluation method proposed in this study to make a more accurate evaluation of the structure.

The experimental results showed that the proposed bridge evaluation method could effectively use the test data for the reliability evaluation of the bridge. The evaluation results were closer to the actual situation. The experimental results verified the validity and the feasibility of the evaluation method based on the response surface method and the Bayesian modification method.

6. Conclusion

In this research, the Bayesian theory and the response surface method were introduced into bridge reliability evaluation. The Bayesian modification method of the probability distribution of the bridge's parameter was analysed. The response surface method was combined with the finite element model to obtain the ultimate state equation. Then the bridge reliability evaluation method based on the Bayesian modification method and the response surface method was proposed. The feasibility of the proposed method was verified by experiment.

The main conclusions of this research were as follows:

1. By combining the finite element method and the response surface method, the ultimate state equation of the bridge was constructed. In this way, the problem of the expression of the ultimate state equation of the complex structure being difficult to obtain was effectively solved. The experimental results showed that the response surface method could accurately reflect

the relationship between the input parameters and the structural response. The response surface method could easily be used to construct the ultimate state equation of complex structures.

2. Based on the Bayesian theory, a modification method for the probability distribution of the bridge's parameters was proposed. The modification method realized the application of the detection data to the reliability evaluation of the bridge. The experimental results showed that the evaluation results of the bridge based on the Bayesian modification method were more consistent with the actual situation of the structure.
3. The reliability index of the specimen was calculated using the proposed reliability evaluation method. The results showed that the proposed method for bridge reliability evaluation based on the Bayesian modification method and the response surface method was feasible, and that it could be used in practical engineering.

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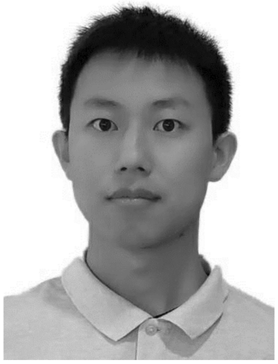

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