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SAFETY ASSESSMENT OF CONTINUOUS CONCRETE GIRDER BRIDGES SUBJECTED TO RANDOM TRAFFIC LOADS CONSIDERING COUPLED FLEXURAL-SHEAR FAILURE

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ABSTRACT

Bridges generally exhibit complicated mechanical behaviors under external loads, such as flexural-shear coupling, compression-bending coupling, and flexural-shear-torsion coupling. In the context of deterministic design approaches such as design codes, these complicated coupled issues are generally simplified to the safety verification of bridge components under a single mechanical state (i.e. flexural, shear, torsion). Currently, it is available to collect external loads acted on bridges and understand bridge performance under these stochastic external loads. In this manner, the reliability-based full probabilistic approach could be applied to investigate the performance of bridges over their lifetime. However, the majority of existing bridge reliability assessments involving site-specific traffic load measurements focuses on the analysis of bridge components under a single mechanical state. To address this shortcomings, a reliability-based probabilistic analytical framework is established to investigate the flexural-shear capacity of girder bridges subjected to random traffic loadings. The flexural-shear coupled failure path of bridge girders under random traffic loading is characterized for the first time, where the bivariate extreme value theory is incorporated to develop the extreme value distribution of combined flexural and shear load effects. The modified compression field theory recommended by AASHTO is employed to establish the coupled flexural-shear coupling resistances. Finally, the reliability of the flexural-shear performance of bridge girders is evaluated by solving the multivariate ultimate limit state equation. The proposed analytical framework is applied to a realistic bridge. The results show that the reliability index of the flexural-shear coupling evaluation is lower than that of the flexural or shear evaluation, which highlights the importance of the flexural-shear performance checking in the reliability assessment of bridges under random traffic loading. The proposed analytical framework could be further applied to the probabilistic assessment of bridge components subjected to combined loading mechanisms under random loadings.

KEYWORDS

Girder bridge, Safety assessment, Flexural-shear failure, Random traffic load, Multi-variate extreme value modelling

INTRODUCTION

The transport industry has achieved rapid growth over the past two decades. The bridge structure, as a lifeline node in the transport network, is of great importance. However, due to the high degree of temporal and spatial variability of traffic loads, the estimation of bridge load effects and resultant structural safety becomes a difficulty in focusing. As is well known, the actual traffic loads are not accurate to reflect the service loads which given by current design specifications [1-2]. It is, therefore, necessary to conduct traffic load modelling and bridge load effects analysis based on these measurements. This could provide more confidence in evaluating the bridge service life, and allocating limited maintenance resources.

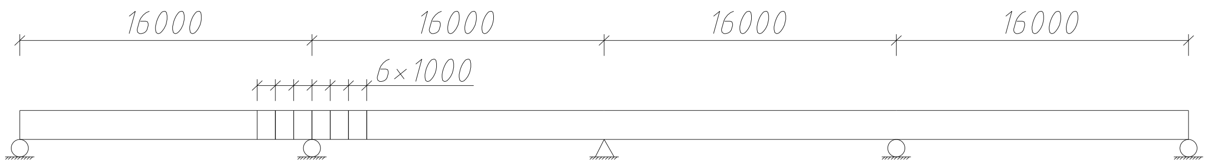
The current studies of traffic load modelling and bridge load effect analysis mainly focus on the evaluation of simple mechanisms, such as flexural or shear, of bridge components. These studies well expound the structural responses and reliability of bridges under site-specific traffic loads. However, bridge components are often subjected to combined forces. For example, the internal force for girder is bending moment and shear force, but axial force cannot be neglected for arch. Besides, due to the spatial randomness of degradation of structural materials, the concerned bridge position that only requires simple mechanism assessment before, may need a re-evaluation but based on a combined mechanism state [3]. Therefore, it is required to establish a probabilistic evaluation method for the combined loading mechanism of bridges under random traffic loading.

At present, a wide range of studies has been dedicated to investigating the structural performance, failure mode, and ultimate design principles of bridge components under combined forces. The flexural-shear coupling of steel members has been a key concern from the past to the present. As early as 1936, Way proposed the elastic critical force equation through the deflection-shear coupling experiment for steel structure. The flexural-shear coupling equation was first proposed by Basler in 1961 on the basis of forces, where the shear force is assumed to be carried only by the web. Then the theory was improved and used by the Eurocode. On this basis, researchers have improved and updated the designed flexural-shear coupling model for steel structures [4]. Meanwhile, the flexural-shear coupling is also important for steel-concrete composite structures. For example, Liang et al. [5] pointed out that the conventional shear design theory of steel plate was conservative as the effect of the deck was not considered, and proposed an improved resistance design formula considering the flexural-shear coupled failure in continuous composite beams. The formula was derived from the nonlinear finite element analysis of the ultimate bearing capacity of continuous beams under bending moments and shears. The angular formulation of the pressure field for flexural-shear coupled effect of concrete members was first proposed by Nilsen and Lampert, which is mainly based on the plasticity theory. Then, Elfgen conducted concrete strength tests and obtained the relationship between flexural-shear interaction of concrete structures through the constitutive model of concrete. Later, due to the effect of tensile stresses in cracks in concrete structures, based on strain coordination in the truss model, Vecchio and Collins proposed the modified compression field theory (MCFT) based on experimental research, which is mainly an analytical model for predicting the load-deformation response of concrete elements under shear stress and normal stress [6-7]. This theory was incorporated into design codes in Canada and the United States and still used now [8]. In summary, the combined resistance of structures under flexural-shear has been thoroughly studied by many scholars and abundant theoretical and experimental results have been obtained. However, few attention has been paid to the combined load effect of the flexural-shear combination of the structure under external loads.

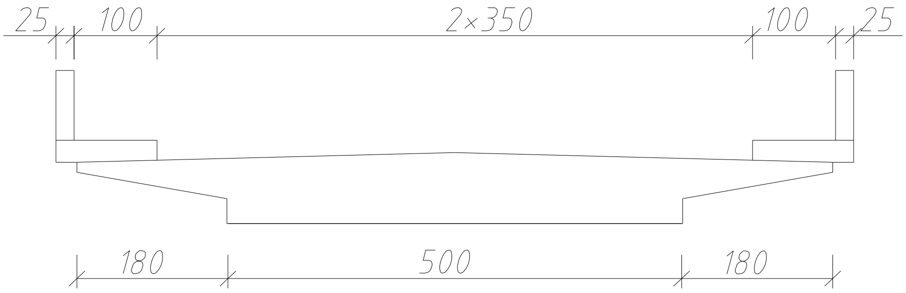
In this paper, the flexural-shear coupling performance of a slab bridge under random traffic loading is focused. The framework structure of this paper is organized as follows. Section 2 introduces the basic description of the studied slab bridge. Section 3 calculates the flexural-shear resistances of bridge slabs according to the MCFT, where several critical girder sections are analyzed. Section 4 presents the modelling of flexural-shear coupling load effects caused by random traffic loading, which is the first time that such an issue is focused. Wherein, the extreme values of flexural-shear load effects are extrapolated using peaks-over-threshold (POT) based bivariate extreme value theory. Section 5 probabilistically evaluates the failure path incorporating the flexural-shear coupling load effects and resistances. This study provides a general framework using probabilistic reliability theory to evaluate the flexure-shear performance of bridges under random traffic loads. Since bridge components are generally subjected to combined forces such as bending, shear, compression, and torsion. This study will provide an approach basis for clarifying the probabilistic reliability evaluation of bridge components under such complicated loading states.

DESCRIPTION OF THE BRIDGE

In the study, a continuous concrete slab bridge is investigated. It has four equal spans with an arrangement of 4×16m. The bridge structure was designed according to the Chinese standard drawings of JT/GQB008-96. Figure 1 shows the layout of the bridge. The bridge carried two lanes of unidirectional traffic. The grade of the constructed concrete is C25, and the bridge deck is paved with 20~110mm concrete cushion and 40mm asphalt concrete pavement. It is noted the studied beam sections shown in Figure 1(a) are sequenced number of 1 to 7 from the left to the right, where the section 4# is the cross-section of the left second-side support.



(a) Vertical diagram



(b) Cross-section diagram

Fig. 1 –The vertical and cross-section diagrams of the studied bridge (unit: mm)

To evaluate the bridge performance under random traffic loading, 90 days of weigh-in-motion (WIM) data from another site are used for the analysis. It is found from the statistics of the WIM data that the 2-axle truck has a proportion of 65.21%, and that for 3-axle, 4-axle, 5-axle, and 6-axle trucks are 6.79%, 10.86%, 2.29%, and 15.03%, respectively. The daily truck flow of the WIM data is 4000 veh/d. To study the performance of the flexural-shear performance of the slabs, 7 sections are studied as shown in Figure 1(a). For these cross-sections, bending moment and shear force are equally significant both under dead loads and live loads.

FLEXURAL-SHEAR COUPLING CAPACITY

Modified compression field theory

The flexural capacity of the normal section of beams is generally obtained based on the limit state equation where the longitudinal bars yield. However, in the design of inclined sections, excessive longitudinal bars are used in the beam bottom to prevent the failure of the normal section. In doing so, the longitudinal rebars generally not yield even though the beam failed. This calculates larger flexural capacities of normal sections. Several investigations have shown that the performance of longitudinal reinforcement is influenced by the shear forces in inclined sections. Therefore, if the design equations for the bearing capacity of section bending and shear are considered separately, the results are too conservative and the design cost is increased. The MCFT shows that cracked concrete is considered as a new material. Residual tensile stresses between cracks should be taken into account despite the cracking of the concrete and the effect of longitudinal reinforcement on its shear should not be overlooked. In MCFT, the constitutive equation can well reflect the interaction of the bearing capacity between bending and shear, which is mainly established by the average stress and strain. In MCFT, the shear force of the beam section is shared jointly by the shear-compression region of concrete and the stirrup intersecting with the inclined cracks. The contribution of the stirrup is based on the variable-angle frame model, while the contribution of concrete is provided by the tensile stress of the inclined cracked concrete. AASHTO (2012) [9] adopted MCFT for the design of concrete beams. In many national design codes, including the Chinese design specifications, the flexural-shear coupling effect is not taken into account for the design of concrete beams. In the following study, the flexural-shear relationship of the structure is established using the MCFT as specified in AASHTO (2012) [9].

In MCFT, for reinforced concrete beams subjected to flexural-shear coupling, the shear performance of the skewed section mainly depends on the shear-to-span ratio, the reinforcement ratio of ordinary longitudinal and hoop bars, the strength of concrete, and the aggregate biting force. Therefore, when cracks appear in the diagonal section of a concrete beam, its shear bearing capacity is mainly borne by the tensile stress of the diagonally cracked concrete in the shear-compression zone, the aggregate occlusion force in the tension zone, the shear force of the bending-up reinforcement, and the shear force provided by the configuration of the hoop reinforcement. Therefore, the shear capacity of the concrete beam is determined as the minimal value between Equations (1) and (2), as specified in AASHTO (2012) [9]:

$$V_n = 0.25 f_c b_v d_v + V_p \tag{1}$$

$$V_n = V_c + V_s + V_p = 0.083\beta\sqrt{f_c} b_v d_v + \frac{A_v f_y d_v (\cot\theta)}{s} + V_p \tag{2}$$

where V_n denotes the minimum flexural capacity value of the sloped section; V_p denotes the vertical component provided by the prestressing force; V_c denotes the component provided by the tensile stresses of the cracked concrete of the sloped section; and V_s denotes the component provided by the tensile stresses of longitudinal reinforcement in the concrete; f_c denotes the design value of compressive strength of the concrete; b_v denotes the actual width of the web; d_v denotes the actual shear height of the sloped section; θ denotes the angle of inclination of the bar stress; β denotes the tension transfer coefficient of cracked concrete in the diagonal section, where θ and β are mainly determined by the characteristics of the sloped section as well as the applied external load; s denotes the spacing of the configured hoops; A_v denotes the area of the configured hoops; and f_y denotes the yield strength of longitudinal reinforcement in the sloped section.

Affected by the flexural-shear coupling effect, reinforced concrete beam cross-section under external loading usually appears complex mechanical behavior, such as flexural-shear coupling, compression-flexural coupling, flexural-shear-torision coupling, etc., and then produce a certain amount of external bending moments; with the increase of sustained load, these external moments in the beam cross-section is gradually differentiated into tensile stresses, and the resulting tensile stresses are mainly borne by the longitudinal steel reinforcement. Therefore, the tensile strength of the longitudinal reinforcement should be considered when calculating the shear capacity of the beam section under the flexural-shear coupling, and its tensile capacity should be calculated and verified whether it meets the requirements through equation (3).

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_n|}{d_v} + \left(\left|V_n - V_p\right| - 0.5 V_s\right)\cot\theta \tag{3}$$

where A_{ps} denotes the cross-sectional area of prestressing reinforcement; f_{ps} denotes the yield strength of prestressing reinforcement; A_s and f_y denote the cross-sectional area and yield strength of flexural reinforcement; M_n denotes the bending resistance; and V_n denotes the shear resistance.

Analyzing the coupled flexural-shear resistance based on the MCFT, the total cross-section strain generated in the concrete beam section under flexural-shear coupling is equal to the sum of the cross-section strains generated under bending moments and pure shear, and the same is true for the total reinforcement strain. In practical calculations, the strain produced by the longitudinal reinforcement is equal to twice the strain in the beam section. The strain in the longitudinal reinforcement can be obtained from equation (4).

$$\varepsilon_x = \frac{\frac{|M_n|}{d_v} + 0.5|V_n - V_p|\cot\theta - A_{ps}f_{ps}}{2(E_sA_{s+E_p}A_{ps})} \quad (4)$$

where ε_x denotes the total strain of the flexural reinforcement on the bending side of the member; E_s and E_p denote the modulus of elasticity of the flexural and prestressing reinforcement respectively.

Based on Equations (3) and (4), the flexural capacity of normal section for beams can be obtained. It is shown that the coupling of flexural and shear behavior of concrete beams is considered by the MCFT. The coupling effect makes the beam in the risk state. When the beams under loads, the coupling effect of the flexural-shear behavior could be considered.

For the parameters of stirrups and concrete strength, the values of θ and β under different ε_x can be obtained based on AASHTO (2012)[9]. The shear capacity can be calculated by Equations (1) and (2). The flexural capacity can be calculated by Equations (3) and (4), approximately. Finally, the coupling curve of flexural-shear resistance is obtained.

Flexural-shear resistance curve

In the framework of probabilistic reliability, material property and geometric sizes are random variables. It is generally assumed that these parameters are normally distributed. The statistical distribution of these parameters can be referred to Wang et al [3], which are shown in Table 1. Based on these variables, the flexural-shear resistances under any εx can be calculated using Monte-Carlo modelling. 108 groups of variables are modelled to determine the resistance model of any given beam section, and the sole flexural resistance, shear resistance, and flexural-shear resistance are obtained. For instance, the mean value of flexural resistance of section 4# is 1.385×10^4kN·m, and the standard deviation is 1.155×10^3 kN·m. The mean shear resistance of section 4# is 7.525×10^3kN, the standard deviation is 8.361×10^2 kN. For the special concern of flexural-shear resistance, Fig. 2 gives the results. It is found from the figure that the flexural-shear coupling reduces the sole flexural (or shear) capacity. It is, therefore, very important to consider the coupling behavior of the flexural-shear resistance curve in the evaluation of beam performance.

Tab. 1 - The random variables of material property and geometric sizes [3]

Parameters	Unit	Deviation	COV	Nominal value
fcu	MPa	1.5868	0.1928	17
fy	MPa	1.0821	0.1211	235
fy	MPa	1.0849	0.0719	335
h0	mm	1.0124	0.0229	628
b	mm	1.0013	0.0081	5400
A	mm²	1.0000	0.0350	70224
As	mm²	1.0000	0.0350	20944
as	mm²	1.0000	0.0350	1621
a	mm	1.0179	0.0496	60
s	mm	1.0000	0.1000	100

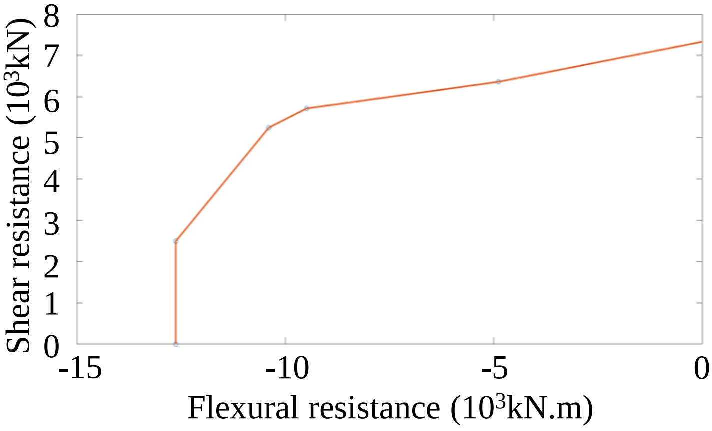


Fig. 2 – Flexural-shear resistance curve based on mean values

FLEXURAL-SHEAR COUPLING LOAD EFFECTS

Time-history of bending moments and shear forces under random traffic loads

The internal forces for beams are time-varying under random traffic loading. They are also correlated since their time-history is induced by the same traffic fleet. In conventional bridge design, only the most unfavorable bending moment or shear is concerned. However, for girder sections, they do not present the most unfavorable bending moment or shear, but the significant bending moment and shear concurrently, which are also drawn as the flexural-shear resistance curve as shown in in Figure 2. The magnitude of the correlation between bending and shear in any beam section is determined by their corresponding influence lines. To ensure the safety of given girder sections subjected to bending moment and shear force, the calculated concurrent bending moment and shear force should be within the envelope of the bend-shear resistance curve. It is generally assumed the bending moment and shear force under dead loading are constant with very small variability. Therefore, the safety assessment of flexural-shear behavior is to determine the failure path of coupling bending moment and shear force under traffic loading based on Figure 3.

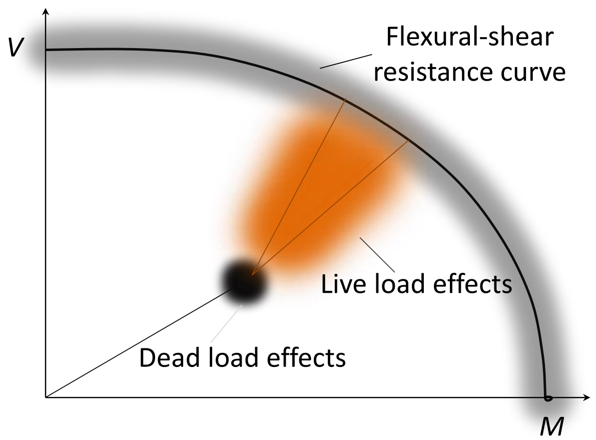


Fig. 3 –Schematic diagram of failure path determination of flexural-shear coupling

To obtain the most unfavorable load effects of the section, it is necessary to consider that the symbol of internal forces generated by dead load and live load is the same. Under this circumstance, the failure probability may be the largest. Fig. 4 shows the time-history curve of the bending moment and the shear force for section 4# under traffic loading. It can be seen clearly that the bending moment and the shear force have a strong correlation The bending moment and the shear force are likely to reach their large values concurrently. It is also can be seen that the bending moment is positive and the shear force is negative caused by traffic loads. Therefore, the negative bending moments and positive shear forces produced by random traffic loading are extracted as the underlying data for further analysis.

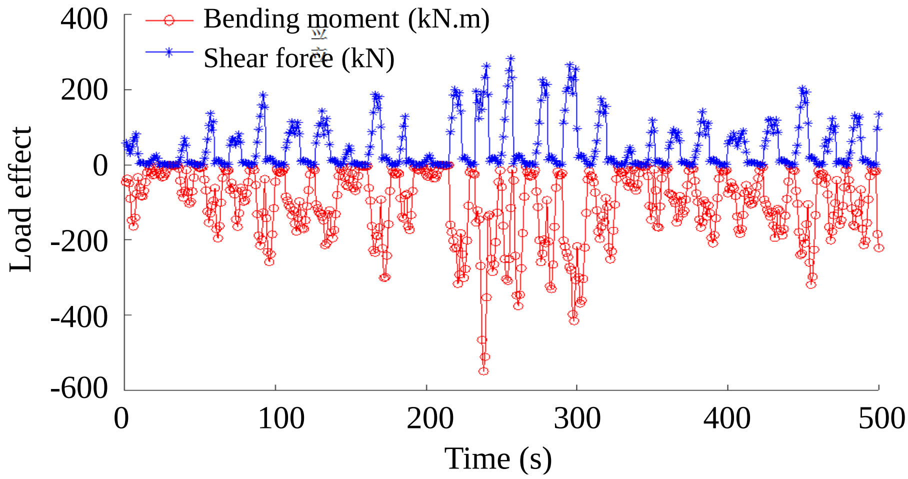


Fig. 4 –Parts of time-history of bending moment and shear force of section 4# under traffic loads

Extreme extrapolation of combined bending moments and shear forces

The safety assessment of flexural-shear coupling, as shown in Figure 3, is to determine the closest path between dead load effects and the flexural-shear resistance curve. Due to the randomness of live load effects caused by traffic loading, it is important to infer the characteristic traffic load effects of high return periods based on limited traffic measurements. Therefore, the bivariate extreme value extrapolation algorithm is required to determine the characteristic value [10].

Bivariate extreme value theory is to solve the modelling of extremely small probability events with bivariate variables reaching their maximum concurrently. Obviously, the extrapolation of extreme bending moment and shear force under random traffic flow is to establish their coupling probability model using this approach. The bivariate extreme value theory is developed from the classical extreme value theory, which can be approximated by using the block maxima based generalized extreme value distribution extrapolation, or POT based GPD (generalized Pareto distribution) extrapolation. The POT theory is more suitable for the extreme value modelling of limited underlying samples than the block maxima theory. In the paper, the bivariate POT theory is used to analyze the probabilistic extreme value model of coupling load effects of bending moment and shear under traffic loading.

According to the classical POT theory, the distribution function of the load effects X caused by traffic loading is F_x. For a sufficiently large threshold u_x, the tail of X can be approximated by GPD, given by

$$F_x(x) = 1 - \varsigma_x[1 - G_x(x)] = 1 - \varsigma_x\left(1 + \xi_x \frac{x-u_x}{\sigma_x}\right)^{-1/\xi_x}, x > u_x \tag{5}$$

where G_x is the GPD function of x; ξ_x, σ_x, and u_x are the shape parameter, scale parameter, and position parameter (threshold) of G_x respectively; $\xi_x = \Pr(x > u_x)$ is the probability of sample x exceeding u_x.

For two load effect samples, X and Y, the distribution functions are F_x and F_y respectively. It is necessary to find the case where x and y reaching their maximum concurrently. That is to analyze the tail characteristic of their joint distribution $F(x, y)$. According to Eq. (5), the marginal distributions for X and Y could be approximated by GPD when the threshold u_x and u_y are large enough. The

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relevant parameters for G_x and G_y are (ξ_x, σ_x, u_x, ς_x) and (ξ_y, σ_y, u_y, ς_y) respectively. Making the following changes [11], we have

$$\bar{X} = -\left(\log\left\{1 - \varsigma_x\left[1 + \xi_x \frac{X-u_x}{\sigma_x}\right]^{-1/\xi_x}\right\}\right)^{-1}, X > u_x \tag{6-1}$$

$$\bar{Y} = -\left(\log\left\{1 - \varsigma_y\left[1 + \xi_y \frac{X-u_y}{\sigma_y}\right]^{-1/\xi_y}\right\}\right)^{-1}, Y > u_y \tag{6-2}$$

$\bar{X}$ and $\bar{Y}$ can be approximated to the standard Fréchet distribution. Provided the distribution function of $(\bar{x}, \bar{y})$are $\bar{F}(\cdot)$, $F(x, y)$ can be expressed as follows when $X > u_x$ and $Y > u_y$,

$$F(x,y) \approx G(x,y) = \bar{F}(\bar{x},\bar{y}) = \exp\{-V(\bar{x},\bar{y})\} = \exp\left\{-\left(\frac{1}{x}+\frac{1}{y}\right)A\left(\frac{x}{x+y}\right)\right\} \tag{7}$$

where $A(t) = \int_0^1 \max\big(w(1-t), t(1-w)\big)\, dH(w)$, V is the dependent function of the bivariate Fréchet marginal distribution H. $A(t)$ is the Pickands convex dependent function, satisfying $\{t, (1-t)\} < A(t) < 1$ where t is the coefficient of correlation.

Therefore, the extreme value models of coupling bending moment and shear force under traffic loading could be established using GPD and its dependent function.

Moment-shear coupling load effects

According to the bivariate POT theory, the optimal threshold based on marginal data can be used as the joint threshold of the bivariate distribution. For the studied bridge, the best thresholds of bending moment and shear effect of live load are determined by the Kolmogorov–Smirnov test and maximum likelihood approach. The GPDs for the bending moment and shear force can be obtained from the above thresholds. Figure 5 shows an example of using GPD modeling of marginal traffic load effects of section 4#. It is seen from the figure that the GPD well captures the tail tendency of the underlying data.

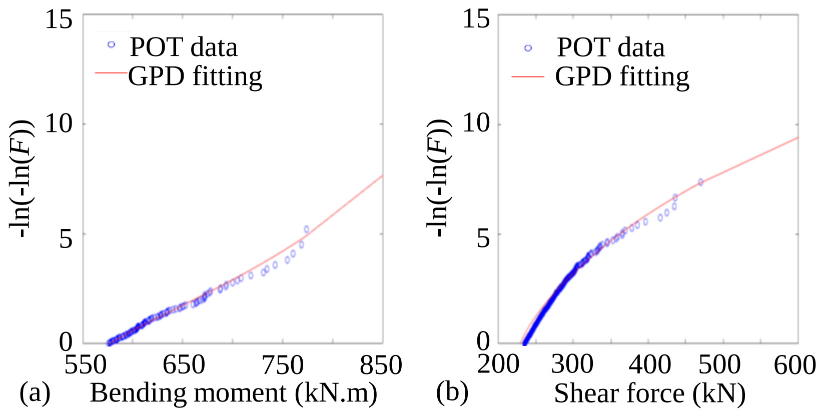

Fig. 5 –GPD modeling of marginal traffic load effects of (a) bending moment and (b) shear force: an example of section 4#

Using the bivariate thresholds, dependent functions can be used to model the tail tendency between the bending moment and shear force. Herein, many dependent functions provided in the R package are analysed [12]. It is found the bivariate logical copula was better to describe the dependency between bending moment and shear force. Fig. 6 shows an example of modelling the dependence between bending moment and shear force caused by traffic loading using the bivariate logical copula. The relative fitting parameters for section 4# are given in Table 2. The fitting results show that the bending moment has a strong tail correlation with the shear force. Based on these

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analyses, the maximum distributions of bending moment and shear force under traffic loading can be modelled, which can be further used for structural safety assessment.

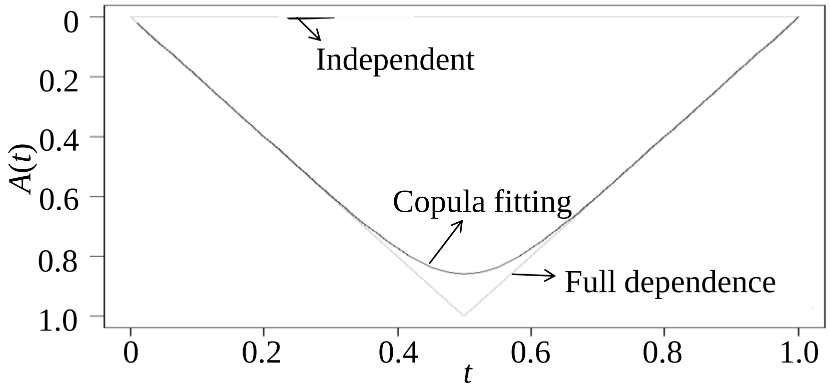

Fig. 6 –Copula fitting on tail dependence of bending moment and shear force caused by traffic loading: an example of section 4#

Tab. 2 - The fitting parameters of marginal GPDs and bivariate logical copula of section 4#

Marginal distribution						Dependence structure
Bending moment (kN•m)			Shear force (kN)			
σ	ξ	u	σ	ξ	u	t
51.6	0.099	550	19.6	0.139	220	0.19

PROBABILISTIC RELIABILITY ASSESSMENT

Limit state equation

Considering the dead load effects and traffic load effects, the limit state equation for flexural-shear assessment is given by

$$Z = R(M,V) - S_d(M,V) - S_q(M,V) \tag{8}$$

where R is the resistance; S_d is the dead load effect; S_q is the traffic load effect.

The flexural resistance is coupled with the shear resistance, as shown in Figure 2. However, the bending moment and shear force under dead loads could be regarded to be independent of each other. Figure 6 and Table 2 are also shown that the bending moment is strong coupled with the shear force under traffic loads. Therefore, the probability of failure is calculated based on $\Pr(Z < 0)$. In the paper, Monte-Carlo modelling is used to calculate the failure probability and reliability index.

Safety assessment

The reliability indexes of the concerned 7 cross-sections are calculated. Three failure modes with their corresponding reliability indexes are analyzed, which are flexural failure, shear failure, and flexural-shear failure. The results are given in Table 3.

The results show reliability indexes of flexural and shear are both greater than 5.61, indicating the failure probability is less than 10^{-8}. This is because most bridges in China have a good safety margin. However, for other engineering cases, the results might be different. However, when considering the coupling of flexural and shear both in resistances and load effects, the calculated reliability index is much lower, especially for section 4. The lowest probability reliability index for flexural-shear failure mode is 4.53, which is much lower than those calculated by flexural failure and shear failure. It illustrates the flexural-shear failure mode controls the safety of the beam section.

The conventional consideration of flexural checking or shear checking is unsafe, and it is required to consider the flexural-shear coupling both in traffic load effects and structural resistances.

Tab. 3 - Reliability indexes of concerned beam sections under different failure modes

Section	Failure mode		
	Flexural	Shear	Flexural-shear
1	>5.61	>5.61	>5.61
2	>5.61	>5.61	>5.61
3	>5.61	>5.61	4.83
4	>5.61	>5.61	4.53
5	>5.61	>5.61	5.07
6	>5.61	>5.61	5.51
7	>5.61	>5.61	>5.61

CONCLUSION

Safety assessment of bridges subjected to traffic loads is a research focus owing to the availability of site-specific traffic data. However, these studies mainly focused on the probabilistic evaluation of simple failure mechanisms of bridge components, such as flexural failure or shear failure. However, the critical components of bridges are often subjected to combined forces, such as flexural-shear, compressive-flexural, flexural-shear-torsion, etc. This paper proposed a novel framework for the probabilistic reliability evaluation of bridges subjected to combined forces. The flexural-shear performance of bridge girders is focused. Based on an exampled continuous concrete slab bridge, the flexural-shear coupling resistance and the flexural-shear coupling load effects under random traffic loading are investigated. Special attention is paid to the bivariate extreme value modelling of coupling bending moment and shear force under random traffic loading, which is to determine the most adverse failure path. Finally, the performance of bridge girders subjected to flexural failure, shear failure, and flexural-shear failure is investigated and compared. Main conclusions are as follows:

(1) According to the MCFT specified in AASHTO, the flexural resistances are coupled with the shear resistances. The existence of the bending moment (or shear force) reduces the shear (or flexural) resistance.

(2) Due to the randomness and uncertainty of traffic loads, the safety assessment of flexural-shear performance is to find the most adverse failure path based on the coupled resistance and dead load effects. It belongs to the scope of bivariate extreme value modelling.

(3) It is found from the studied case that the reliability index subjected to flexural-shear failure is much lower than that subjected to flexural failure or shear failure. It highlights the conventional safety checking of flexural or shear performance is unsafe, and it is required to consider the flexural-shear coupling effect both in traffic load effects and structural resistances for precise structural assessment, which is not covered by current design standards.

This study provides a methodology for estimating the structural safety of concrete beam bridges under random traffic loads, considering coupled flexural-shear failure. To obtain a more comprehensive understanding of the benefits of considering coupled flexural-shear failure compared to conventional flexural failure and shear failure, more example cases are required for thorough investigations.

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