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Research on extending the fatigue life of railway steel bridges by using intelligent control

Jia Liu ^{1*}, Weilian Qu ¹, Nikolaos Nikitas ^{2*}, Zeliang Ji ³

1. Hubei Key Laboratory of Roadway Bridge and Structure Engineering, Wuhan University of Technology, Wuhan 430070, China;

2. School of Civil Engineering, University of Leeds, Leeds, LS2 9JT, UK;

3. School of Civil Engineering and Architecture, Wuhan University of Technology, Wuhan 430070, China

Abstract: This paper investigates the potential of a vibration control-inspired method towards extending the fatigue life of railway steel bridges. Based on coupled thermal-mechanical and vehicle-track analysis, both the residual stresses from welding and these from traffic on the bridge are obtained. Subsequently, a multi-scale approach with a shell Finite Element (FE) model of the whole bridge and a solid FE model of its critical joints is put forward. The equation of motion is established for the controlled bridge, equipped with a Magnetorheological-Tuned Mass Damper (MR-TMD) system, while the combination of excitation, welding and control effects is practiced through own-developed packages and commercial software sub-model routines. The framework is showcased for the study of the Poyang Lake Railway Bridge in China. After obtaining the controlled stress states at the critical welded joint, the fatigue crack initial life is evaluated by using the critical plane method and the linear cumulative damage theory. Simulation results indicate that the multi-scale modelling approach followed, meets the accuracy needs for capturing the cracking process of the welded joint with high computational efficiency. The MR-TMD system, even when moderately reducing the critical joint stress amplitudes, can improve substantially the overall bridge fatigue resistance over the uncontrolled structure.

Keywords: Sub-model/multi-scale method; railway steel bridges; fatigue; structural control.

Notation

		<u>Section 2.1</u>		
k_x	the x directional thermal conductivity factors	M_c	weight of the train car	
k_y	the y directional thermal conductivity factors	J_c	inertia of the train car	
k_z	the z directional thermal conductivity factors	M_t	weight of bogies	
λ	efficiency of the heat source	J_t	inertia of bogies	
U_w	welding voltage	M_{wi}	the wheel's weight	
I_w	welding current	v_{wi}	vertical displacement of each wheel	
V_H	volume of the welding unit	v_{ti}	vertical displacement of the bogies	
ρ	the parent material density	φ_{ti}	rotation angle of the bogies	
C	specific heat capacity	v_c	vertical displacement of the repeating train car	
T	the joint temperature generated by the welding	φ_c	rotation of the repeating train car	
t	the independent time variable	u_1, u_2	the rail vertical displacements	
N_x	the perpendicular to the boundary of x direction cosines	u_3, u_4	the railway bridge sleeper vertical displacements	
N_y	the perpendicular to the boundary of y direction cosines	u_5, u_6	the ballast vertical displacement	
N_z	the perpendicular to the boundary of z direction cosines	k_{y1}	the elastic coefficients of the fastener	
h_c	the heat transfer coefficient of convection	k_{y2}	the elastic coefficients of the ballast	
h_r	the heat transfer coefficient of radiation	k_{y3}	the elastic coefficients of the railway bridge sleeper	
q_s	boundary heat flux	c_{y1}	the damping coefficients of again the fastener	
T_r	temperature of radiation	c_{y2}	the damping coefficients of again ballast	
T_∞	surrounding temperature	c_{y3}	the damping coefficients of again the railway bridge sleeper	
ff	the heat source distribution of the double ellipsoid model for front heat source	\mathbf{M}_v	the mass matrices of the train	
fr	the heat source distribution of the double ellipsoid model for rear heat source	\mathbf{C}_v	the damping matrices of the train	
qf	the heat source of the double ellipsoid model for front heat source	\mathbf{K}_v	stiffness matrices of the train	
qr	the heat source of the double ellipsoid model for front heat source	$\mathbf{\ddot{a}}_v$	the acceleration vector	
v	welding speed	$\mathbf{\dot{a}}_v$	the velocity vector	
x_0	the x coordinate of the welding initial position	\mathbf{a}_v	the displacement vector	
a_1	arc welding parameter	\mathbf{Q}_v	the force vector	
a_2	arc welding parameter	\mathbf{M}_1	the mass, damping vector of the track	
b	arc welding parameter	\mathbf{C}_1	the damping vector of the track	
c	arc welding parameter	\mathbf{K}_1	the stiffness matrices vector of the track	
$\{d\sigma\}$	vector of stress	\mathbf{Q}_1	the load vector of the track	
$\{d\varepsilon\}$	vector of strain	e	the element identifier of the 2D track beam	
dT	temperature increment	k_H	the equivalent spring stiffness	
$[D]$	elastic or elastic-plastic constitutive law matrix	u_i	the interpolating function of u_1 and u_2 of the nodal displacement	
$\{C\}$	temperature dependence vector	\mathbf{M}_C	the mass matrices of the coupled system	
		\mathbf{C}_C	the damping matrices of the coupled system	
		\mathbf{K}_C	the stiffness matrices of the coupled system	
		\mathbf{Q}_C	the load vector	
		$\mathbf{C}_{VI}, \mathbf{C}_{IV}$	the coupling damping	
		Ω	spatial angular frequency variable	

Ω_c	the truncated spatial angular frequency of the vertical profile irregularity	F_{dd}	the voltage sensitivity of controllable Coulomb damping force of the damper
Ω_r	the truncated spatial angular frequency of the alignment irregularity	K_{dd}	the voltage sensitivity of equivalent axial stiffness of the damper
A_v	the high interference roughness coefficient	η	the time coefficient of the damper's magnetic hysteresis response
<u>Section 3.1</u>			
Z_r, Z	the vertical displacements of the bridge deck and TMD mass respectively	I	the applied current intensity
$\ddot{\mathbf{x}}$	the acceleration vector of the bridge discrete model	u	the internal variable reflecting the relationship between model parameters and current intensity.
$\dot{\mathbf{x}}$	the velocity vector of the bridge discrete model	<u>Section 3.2</u>	
\mathbf{x}	the displacement vector of the bridge discrete model	$F_{d\max}$	the maximum coefficient
\mathbf{M}	the mass matrices	$F_{d\min}$	the minimum coefficient
\mathbf{C}	the stiffness matrices	ξ	the adjustment factor of the Coulomb damping force
\mathbf{K}	the damping matrices	<u>Section 3.4</u>	
\mathbf{H}	locator matrix of installed MR-TMD system	ε_x	the x -directional normal strain
$\mathbf{F}(\mathbf{t})$	the external force input	ε_y	the y -directional normal strain
\mathbf{U}_{MR-TMD}	the control force vector provided by the MR-TMD system	ε_z	the z -directional normal strain
K_{Ti}	the stiffness properties of the i^{th} TMD in the MR-TMD	ε_{xy}	the xz -directional shear strain
C_{Ti}	the damping properties of the i^{th} TMD in the MR-TMD	ε_{yz}	the yz -directional shear strain
F_{MRi}	the damping force of the i^{th} MR damper	ε_{xz}	the xz -directional shear strain
C_d	the viscous damping coefficient	$\varepsilon_{zz}, \varepsilon_{xz}, \varepsilon_{yz}$	the intermediate step calculation parameters
F_d	the coefficient of controllable Coulomb damping force	θ, Φ	the calculated angles
K_d	the equivalent axial stiffness of the damper	$\Delta\varepsilon_{eq}^{cr}$	the multi-axis fatigue damage parameter of equivalent stain
f_{0i}	the output force deviation caused by the damper accumulator	ε_n^*	the normal strain amplitude obtained by the cycle counting
e_b	the Bingham sliding displacement	$\Delta\gamma_{\max}$	the relevant shear strain amplitude
C_{ds}	the coefficient of viscous damping of damper on the condition of zero electric field strength	$\Delta\varepsilon_a$	the total strain amplitude
F_{ds}	the coefficient of controllable Coulomb damping force of damper on the condition of zero electric field strength	$\Delta\varepsilon_e$	the elastic strain amplitude
K_{ds}	the equivalent axial stiffness of damper on the condition of zero electric field strength	$\Delta\varepsilon_p$	the plastic strain amplitude
C_{dd}	the voltage sensitivity of viscous damping of the damper	σ_f'	the fatigue strength coefficient
		ε_f'	the fatigue plastic coefficient
		E	the material elastic modulus
		m	the elastic fatigue strength index
		n	the plastic fatigue strength index
		N_f	the cycle counts of fatigue life
		$\frac{\sigma_m}{\sigma_m}$	the average stress
		σ_m	the mean value of calculated redistributed welding residual stress
		σ_s	the yield strength of steel
		$D=1$	the critical failure fatigue threshold

1. Introduction

Railway steel bridges are very common long span bridges, constituting a large sector of the traffic communication network. Owing to their long time and frequent train traffic, cumulative damage may occur in their welded regions. In particular, stress concentrations can easily be generated under the combined action of multi-axial dynamic-excitation stress and welding-owed residual stress. Such could further lead to fatigue damage that sets off as crack initiation and, in a worst case scenario, could progress to failure of parts of the bridge, putting a serious concern in terms of life cycle design. Collapse and damage of steel bridges caused by cumulative fatigue action have been frequently reported across the world up to this date [1-3]. Therefore, it is important to propose and adopt effective means to prolong the fatigue crack initial life of steel bridge parts; such could subsequently extend the service life of railway steel bridges, and reduce the risk, economic losses and casualties caused by potential damage.

In the past decades, many scholars have carried out considerable theoretical and experimental research in the field of bridge fatigue life assessment. Namely, within the achievements produced in the field one can quote the establishment of fatigue damage models, the study of initial crack mechanisms and the evaluation of a structure's overall fatigue life [4,5]. Li and Chan, for instance, applied continuous damage mechanics to their dynamic constitutive model, and combining modelling with strain measurements from a state-of-the-art online health monitoring system, evaluated the fatigue life of the Tsing Ma Bridge under vehicle vibration [6]. Xu et al. established a fatigue damage evolution model based on continuous damage mechanics, and evaluated the fatigue life of the key steel parts of the Tsing Ma Bridge under wind load according to the so-called hot spot stress [7]. Guo et al. researched the effect of environmental temperature on the fatigue life of the

23 welded steel bridge deck of the Runyang Suspension Bridge, finding that the relationship between
24 temperature and fatigue life of welded nodes is linear within a certain temperature range ^[8].
25 Righiniotis studied the fatigue life of railway steel bridges under increasing train traffic load, and
26 concluded that when the train load increased to a certain degree, the fatigue life was drastically
27 reduced ^[9].

28 However, little of the literature addressed pathways to extend the fatigue life of steel bridges, and
29 particularly the research which adopts intelligent control techniques to do so for railway
30 applications is relatively sparse. Thus, a method is herein proposed to mitigate the responses of civil
31 engineering structures and reduce the stress cycle amplitudes by using a Magnetorheological -
32 Tuned Mass Damper (MR-TMD) control system. It is well known that TMDs can mitigate the
33 vertical responses in the middle span of a bridge without auxiliary supporting structures and with
34 minimal aesthetic intrusion to the architectural shape. Besides, MR dampers are among the most
35 promising control devices that can adjust in an active fashion the provided damping force in order
36 to yield increased vibration mitigation performance. In addition, there are many relevant studies on
37 the use of intelligent materials, intelligent configurations, and intelligent algorithms ^[10-12]. For the
38 case of a pedestrian bridge, that vibration monitoring showed prone to human-induced vibrations,
39 Caetano et al. proposed an effective TMD control system and demonstrated experimentally the
40 effectiveness of it ^[13]. Closer to the railway and fatigue focus of this paper, Andersson et al.
41 developed a procedure to account for detuning effects associated with passive TMD systems of
42 train-loaded bridges. Using an incremental frequency estimation technique for adjusting a variable
43 stiffness TMD, they researched numerically the cumulative fatigue damage on a case study bridge
44 equipped with different mass damper systems, ultimately establishing that adaptive systems can
45 significantly outperform passive equivalents ^[14]. Museros et al. researched the control effect on

46 moving loads' dynamic stresses for a simply supported girder bridge when using viscous fluid
47 dampers ^[15]. The same research team further proposed the potential of extending their methodology
48 to applications of railway bridges, and indeed two railway bridges in Spain were theoretically
49 studied using this method, obtaining excellent vibration/stress control performance ^[16]. Wang et al.
50 researched the classical problem of defining optimal parameters for a TMD, presenting an
51 optimisation technique for a steel railway bridge under different train traffic scenarios. For the
52 underlying numerical analysis of their TMD's control efficiency, they considered parameters such
53 as the static displacement of the bridge middle span, the TMD's mass block expenditure and the
54 difficulty of the damper manufacturing ^[17]. Luu et al., similarly on optimisation grounds, presented
55 a method where the TMD system parameters are directly tuned based on an objective function
56 taking into account the H_2 norm and the constraints of all peaks of the same height over the
57 frequency range of interest. The result is multi-modal, robust vibration control directly applicable to
58 multi-span railway bridges ^[18]. Younesian et al. beyond studying the optimal frequency and
59 damping ratio of a TMD system, also attempted to uncover the effect of boundary conditions on
60 TMD performance for the indicative case of a Timoshenko beam ^[19]. Kahya et al. designed a series
61 multiple tuned mass damper (STMD) device consisting of two in series connected
62 spring-mass-damper units. The assembly was shown that can effectively suppress the resonance
63 introduced from high-speed trains (HSTs), being at the same time economical and sufficiently
64 robust to detuning effects ^[20]. Yau proposed a string-type tuned mass damper composed of a
65 distributed TMD subsystem and a stretched string acting on a simple beam. The outcome for this
66 ensemble is a TMD with an adjustable tuning frequency, which could more efficiently mitigate
67 resonant response due to moving loads ^[21]. Lin et al. numerically studied the effect of multiple
68 tuned mass dampers (MTMDs) on the suppression of train vibrations on real Taiwan High Speed

69 Rail (THSR) bridges. For the narrow band train traffic loads they recovered that MTMDs are more
70 effective, robust and reliable than a single passive TMD not only at resonance speeds but
71 throughout the operational range [22].

72 This paper also concentrates on a method to dynamically control, in practice, railway bridges for
73 subsequently extending the fatigue crack initial life of their welded joints. Based on coupled
74 thermal-mechanical and vehicle-track theories, the residual stresses, owing to welding, and the
75 dynamic train traffic actions exerted on the joints of the bridge are respectively obtained. A
76 multi-scale FE shell model of the whole railway steel bridge and a FE solid model of its critical
77 joint in the middle span have been set up jointly based on a sub-model realisation. Further, the
78 equation of motion for the controlled bridge installed with a MR-TMD apparatus is established. The
79 whole approach together with the required control force estimation is practiced by using a
80 commercial software sub-model solution and enriched by own routines. The Poyang Lake Railway
81 Bridge in China has become the testbed of the developed analytical framework. After recovering the
82 controlled stress states in the most critical welded joint location, the fatigue crack initial life is
83 evaluated by using the critical plane method and the linear cumulative damage theory.

84 **2. Simulation method for accurate joint stress calculations**

85 In order to evaluate the fatigue crack initial life of the bridge, it is essential to obtain the accurate
86 stress response of its welded joints. In this section, the simulation principle for evaluating the
87 welding residual stresses and the train traffic load is presented. Further, towards the accurate stress
88 modelling for the bridge joints, the sub-model interaction proposed is described.

89 **2.1 Load simulation**

90 **2.1.1 Simulation of welding residual stress**

91 The simulation of the welding residual stress on the joints of the steel bridge is practised in two

92 steps: In the first step, the double ellipsoidal heat source model ^[23] is used to calculate the real-time
 93 temperature fields in the welded area while using the actual welding parameters, and also taking
 94 into account the cooling process. In the second step, the recovered temperature fields are applied as
 95 external loads to the mechanical model of the joint, and the real-time stress field in the welded area
 96 is produced. The stress of the joint, after also cooling has elapsed, is taken forward in the analysis as
 97 the welding residual stress ^[24-26].

98 Indicatively the steps go as follows. The transient nonlinear heat conduction differential equation
 99 for the welding process can be expressed as ^[27]:

$$100 \quad \frac{\partial}{\partial x} \left(k_x \frac{\partial T}{\partial x} \right) + \frac{\partial}{\partial y} \left(k_y \frac{\partial T}{\partial y} \right) + \frac{\partial}{\partial z} \left(k_z \frac{\partial T}{\partial z} \right) + Q = \rho C \frac{\partial T}{\partial t} \quad (1)$$

101 where k_x , k_y and k_z are the x , y and z directional thermal conductivity factors, respectively;
 102 $Q = \lambda U_w I_w / V_H$ is the heat generation rate, λ is the efficiency of the heat source, U_w and I_w are
 103 respectively the welding voltage and current, V_H is the volume of the welding unit; ρ and C are the
 104 parent material density and specific heat capacity, T is the joint temperature generated by the
 105 welding and t is the independent time variable.

106 The initial value and boundary conditions of Eq. (1) are:

$$107 \quad T(x, y, z, 0) = T_0(x, y, z) \quad (2)$$

$$108 \quad k_x \frac{\partial T}{\partial x} N_x + k_y \frac{\partial T}{\partial y} N_y + k_z \frac{\partial T}{\partial z} N_z + q_s + h_c(T - T_\infty) + h_r(T - T_r) = 0 \quad (3)$$

109 where N_x , N_y and N_z are the perpendicular to the boundary of each direction cosines, h_c and h_r are
 110 the heat transfer coefficient of convection and radiation respectively, q_s is the boundary heat flux, T_r
 111 denotes the temperature of radiation; and T_∞ represents the surrounding temperature.

112 The functions of the double ellipsoidal heat source model can be expressed as ^[23]:

113

114 Front heat source

$$115 \quad q_f(x, y, z, t) = \frac{6\sqrt{3}f_f Q}{a_1 b c \pi \sqrt{\pi}} e^{-3(x-vt-x_0)^2/a_1^2} e^{-3y^2/b^2} e^{-3z^2/c^2} \quad (4)$$

116 Rear heat source

$$117 \quad q_r(x, y, z, t) = \frac{6\sqrt{3}f_r Q}{a_2 b c \pi \sqrt{\pi}} e^{-3(x-vt-x_0)^2/a_2^2} e^{-3y^2/b^2} e^{-3z^2/c^2} \quad (5)$$

118 where f_f and f_r are the heat source distribution of the double ellipsoid model for either front or rear
119 heat source, $f_f + f_r = 2$; q_f and q_r are the heat source of the double ellipsoid model for either front or
120 rear heat source; v is the welding speed; x_0 is the x coordinate of the welding initial position and a_1 ,
121 a_2 , b and c are arc welding parameters.

122 The Von Misses yield criterion is used to simulate the real-time stress fields ^[17]:

$$123 \quad \{d\sigma\} = [D]\{d\varepsilon\} - \{C\}dT \quad (6)$$

124 where $\{d\sigma\}$ and $\{d\varepsilon\}$ are the vectors of stress and strain, respectively; and dT is the
125 temperature increment, $[D]$ is the elastic or elastic-plastic constitutive law matrix and $\{C\}$ is the
126 temperature dependence vector.

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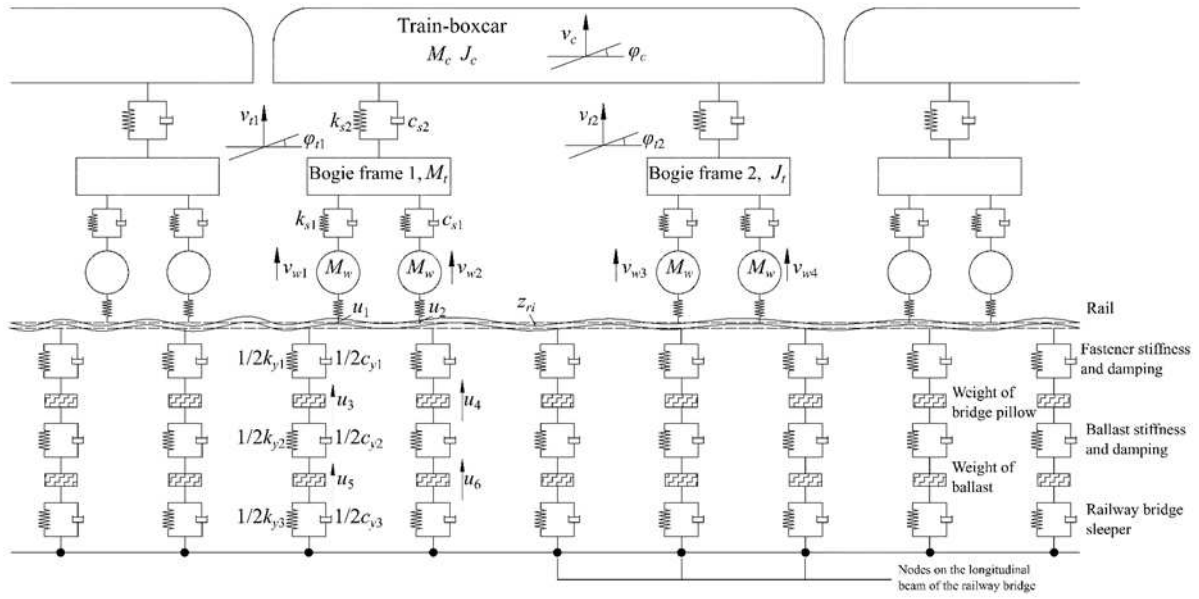
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136 **2.1.2 Simulation of train traffic load**



137
138 **Fig.1 Vehicle-track dynamic coupling model**

139 Based on rigid body kinematics, the vehicle-track dynamic coupling model is established to
 140 simulate the whole process of a train crossing the bridge [28]. Any interaction effect between the
 141 vibration response of the bridge and the track-vehicle system loads is not considered in the paper.
 142 The vehicle-track dynamic coupling model is shown in Fig.1. As it is depicted, M_c and J_c are the
 143 weight and inertia of the train car, M_i and J_i are the weight and inertia of bogies,
 144 $M_{wi}(i=1,2,3,4)$ are the wheel's weight, $v_{wi}(i=1,2,3,4)$ are the vertical displacement of each wheel;
 145 $v_{ii}(i=1,2)$ and $\varphi_{ii}(i=1,2)$ are the vertical displacement and rotation angle of the bogies, v_c and
 146 φ_c are the vertical displacement and rotation of the repeating train car, u_1 and u_2 are the rail
 147 vertical displacements, u_3 and u_4 are the railway bridge sleeper vertical displacements, u_5 and
 148 u_6 are the ballast vertical displacement; k_{y1}, k_{y2} and k_{y3} are the elastic coefficients of the
 149 fastener, ballast and railway bridge sleeper respectively, and c_{y1}, c_{y2} and c_{y3} are the damping
 150 coefficients of again the fastener, ballast and railway bridge sleeper respectively.

151 The train motion equation is established based on a Lagrangian formulation [29]:

$$152 \quad \mathbf{M}_v \ddot{\mathbf{a}}_v + \mathbf{C}_v \dot{\mathbf{a}}_v + \mathbf{K}_v \mathbf{a}_v = \mathbf{Q}_v \quad (7)$$

153 where \mathbf{M}_v , \mathbf{C}_v and \mathbf{K}_v are the mass, damping and stiffness matrices of the train, respectively, $\ddot{\mathbf{a}}_v$,
 154 $\dot{\mathbf{a}}_v$ and \mathbf{a}_v are the acceleration, velocity and displacement vectors respectively, and \mathbf{Q}_v is the
 155 force vector.

156 For the track model, the motion equation of the track can be obtained by using the three layers of
 157 the mass-spring-damper elastic system with discrete supports based on the Hamilton principle [29]:

$$158 \quad \mathbf{M}_1 \ddot{\mathbf{a}}_1 + \mathbf{C}_1 \dot{\mathbf{a}}_1 + \mathbf{K}_1 \mathbf{a}_1 = \mathbf{Q}_1 \quad (8)$$

159 In which $\mathbf{M}_1 = \sum [m]^e$, $\mathbf{C}_1 = \sum [c]^e$, $\mathbf{K}_1 = \sum [k]^e$ and $\mathbf{Q}_1 = \sum [Q]^e$ are the mass, damping,
 160 stiffness matrices and load vector of the track, respectively; e is the element identifier of the 2D
 161 track beam.

162 The \mathbf{Q}_v and \mathbf{Q}_1 load vectors in Eqs.(7) and (8) are functions of the wheel displacement v_{wi} , the
 163 track node displacements u_1 , u_2 and the track irregularity z_{ri} . Their elements can be expressed
 164 as:

$$165 \quad Q_{vi} = f_1(v_{wi}, u_{1i}, u_{2i}, z_{ri}) = k_H (v_{wi} - u_i - z_{ri}) \quad (9)$$

$$166 \quad Q_{li} = f_2(v_{wi}, u_{1i}, u_{2i}, z_{ri}) = -k_H (v_{wi} - u_i - z_{ri}) \quad (10)$$

167 According to the Hertz contact theory, the contact between wheel and rail is simplified to linear
 168 elastic, where k_H is taken as the equivalent spring stiffness for it. The displacement u_i of the
 169 track element can be expressed by the interpolating function of u_1 and u_2 of the nodal
 170 displacement [29].

171 Finally, the vehicle-track coupling dynamic equation is established as:

$$172 \quad \mathbf{M}_c \ddot{\mathbf{a}} + \mathbf{C}_c \dot{\mathbf{a}} + \mathbf{K}_c \mathbf{a} = \mathbf{Q}_c \quad (11)$$

173 where $\mathbf{M}_c = \begin{bmatrix} \mathbf{M}_v & \mathbf{0} \\ \mathbf{0} & \mathbf{M}_1 \end{bmatrix}$, $\mathbf{C}_c = \begin{bmatrix} \mathbf{C}_v & \mathbf{C}_{v1} \\ \mathbf{C}_{1v} & \mathbf{C}_1 \end{bmatrix}$ and $\mathbf{K}_c = \begin{bmatrix} \mathbf{K}_v & \mathbf{K}_{v1} \\ \mathbf{K}_{1v} & \mathbf{K}_1 \end{bmatrix}$ are respectively the mass,

174 damping and stiffness matrices of the coupled system; $\mathbf{Q}_C = \begin{Bmatrix} \mathbf{Q}_v \\ \mathbf{Q}_I \end{Bmatrix}$ is the load vector; \mathbf{C}_{vI} and \mathbf{C}_{IV}
175 are the coupling damping, with $\mathbf{C}_{vI} = \mathbf{C}_{IV}$, and \mathbf{K}_{vI} and \mathbf{K}_{IV} are the coupling stiffness matrix
176 block elements with $\mathbf{K}_{vI} = \mathbf{K}_{IV}$; their calculation can be practiced as in previous cases [29].

177 The train passage time-history on the railway is simulated by using the German [28] high
178 interference track irregularity power spectrum density function as provided for freight railway lines.
179 For the cases of this study, the vertical irregularity power spectral density function of the line is
180 expressed by the formula:

$$181 \quad S_v(\Omega) = \frac{A_v \cdot \Omega_c^2}{(\Omega^2 + \Omega_r^2)(\Omega^2 + \Omega_c^2)} \quad (12)$$

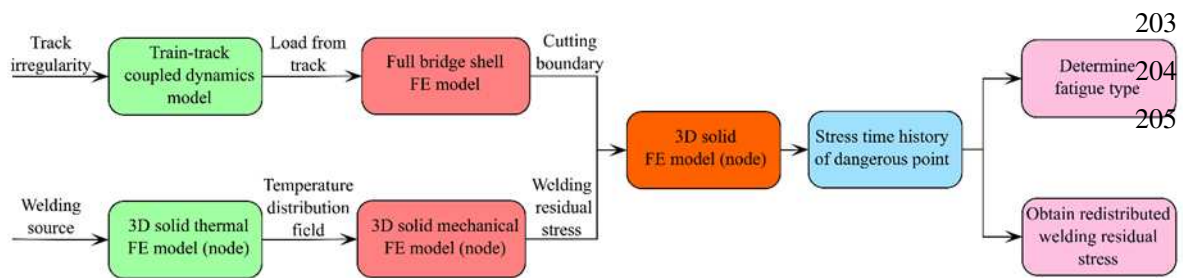
182 where Ω is the spatial angular frequency variable, Ω_c is the truncated spatial angular frequency of
183 the vertical profile irregularity, Ω_r is the truncated spatial angular frequency of the
184 alignment irregularity, and A_v is the high interference roughness coefficient.

185 Employing a step-by-step integration scheme, the real time-history curve of forces on the rail
186 nodes at any instant is produced. This can subsequently be considered as the load on the full-bridge
187 shell FE model scrutinised in the next section.

188 **2.2 Accurate overall stress response evaluation**

189 In order to obtain the accurate stress response of the welded joints of a railway steel bridge, a good
190 insightful numerical model is compulsory for all calculation and analysis purposes. On the one hand,
191 the welding residual stresses are mainly related to the distribution of the welding seam, so a fine
192 solid model should be used to analyse them. On the other hand, the train traffic load is more related
193 to the joint position on the full bridge. Thus, a single solid joint model is not practical to employ for
194 establishing the dynamic responses caused by train traffic. Besides, if a solid FE model was to use
195 for the full bridge, the complexity level would be too high to make practical and the overall

196 numerical efficiency would be classed as low. In order to get a balance between output accuracy and
 197 analytical efficiency, a multi-scale modelling approach based on the sub-model method is adopted
 198 [30]. The sub-model method is a detached modelling practice based on Saint-Venant's principle, also
 199 called as the cutting boundary displacement method. The displacements and loads on the
 200 corresponding sub-model boundary of the “coarse” shell full-bridge model are extracted and applied
 201 to the relatively fine solid sub-model of the bridge joint; this is seen as the best practical
 202 compromise.



206 **Fig.2 Flow chart to obtain the stress response of railway steel bridge nodes**

207 Fig.2 shows the flow chart for obtaining the stress response of the railway steel bridge joints.
 208 According to the vehicle-track coupling dynamics model, mentioned above, the train traffic loads
 209 acting on the nodes along the bridge are obtained by using the track irregularity as excitation. These
 210 are to be applied on the full-bridge shell FE model. Subsequently, the dynamic boundary responses
 211 of the shell full-bridge model in the position of the corresponding solid joint sub-model can be
 212 extracted. Meanwhile, the distribution of the welding residual stresses in the joints are captured
 213 through the thermal-mechanical fine solid model, which is acted by the same heat source of the
 214 actual welding process. The variation within the temperature fields are recorded at both rising and
 215 cooling temperatures and the residual stress maps can be obtained from the fine thermal solid model
 216 when exposing it to the recorded temperature fields. The boundary conditions of the solid model in
 217 the corresponding position of the shell model are applied as external loads while the welding

218 residual stresses are input too within the fine solid model. Consequently, the accurate dynamic
219 stress history of the examined joint within the railway steel bridge when considering the combined
220 welded residual stress and train traffic loads is calculated. The data recovered can be used to
221 estimate the overall fatigue life depending on whether the maximum combined stress will exceed
222 the yield threshold. If it does, the stress fatigue analysis method can be used for subsequent
223 estimations, otherwise the strain fatigue procedure is opted for. Additionally, the welding residual
224 stresses will be released under the external stimulus ^[31], allowing the process to also uncover the
225 redistributed welded stresses values.

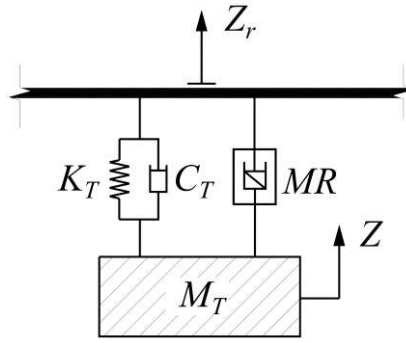
226 **3. Intelligent control towards extending fatigue crack resistance**

227 It is well known that the cumulative fatigue damage of railway steel bridges is mainly induced by
228 cyclic stresses under train running loads. Reducing the amplitude of the cyclic stress by vibration
229 control can evidently improve the fatigue crack resistance of such a bridge. Among all the control
230 devices, the MR damper is probably the most promising intelligent control device being popular for
231 vibration mitigation in civil engineering structures due to its quick response, low energy
232 consumption and good control performance ^[32]. On similar practicality-performance grounds,
233 TMDs are ideal for bridges because of their limited support needs, their many installations'
234 experience, and their non-aesthetically intrusive character. Therefore, a combined MR-TMD
235 intelligent control system is chosen in this remedy for controlling the cyclic stressing behaviour in
236 order to extend the overall structural fatigue life.

237 **3.1 MR-TMD equipped bridge; the controlled model**

238 Due to the maximum dynamic response generated in the middle span of the railway steel bridge,
239 when loaded by train traffic, the MR-TMD system is more effective to set up in the middle span.
240 Fig.3 schematically depicts the bridge mechanical model with the MR-TMD in place. Assuming

241 that vertical displacement of the bridge deck and the vertical displacement of the MR-TMD
 242 auxiliary mass are Z_r and Z respectively, the equation of motion for the bridge can be expressed as:



243
 244 **Fig.3 Schematic of the bridge MR-TMD system model**

245
$$\mathbf{M}\ddot{\mathbf{x}}+\mathbf{C}\dot{\mathbf{x}}+\mathbf{K}\mathbf{x}=\mathbf{F}(\mathbf{t})-\mathbf{H}\mathbf{U}_{MR-TMD} \quad (13)$$

246 where $\ddot{\mathbf{x}}$, $\dot{\mathbf{x}}$ and \mathbf{x} are the acceleration, velocity and displacement vectors of the bridge discrete
 247 model, respectively, \mathbf{M} , \mathbf{C} and \mathbf{K} are the mass, stiffness and damping matrices, respectively,
 248 \mathbf{H} is the locator matrix of the installed MR-TMD system, $\mathbf{F}(\mathbf{t})$ is the external force input, and
 249 \mathbf{U}_{MR-TMD} is the control force vector provided by the MR-TMD system. The latter for an ensemble of
 250 MR-TMDs can be expressed as:

251
$$U_{MR-TMDi} = K_{Ti} (Z_{ri} - Z_i) + C_{Ti} (\dot{Z}_{ri} - \dot{Z}_i) + F_{MRi} (Z_r - Z, \dot{e}) \quad (14)$$

252 where K_{Ti} and C_{Ti} are the stiffness and damping properties of the i^{th} TMD in the MR-TMD
 253 assembly and F_{MRi} is the damping force of the i^{th} MR damper; this can be expressed [33] as:

254
$$F_{MRi} = C_{di}\dot{e} + F_{di} \text{sgn}(\dot{e}) - f_{0i} = K_{di} (Z_r - Z - e) - f_{0i} \quad (15)$$

255 where C_d is the viscous damping coefficient, F_d is the coefficient of controllable Coulomb
 256 damping force, K_d is the equivalent axial stiffness of the damper, f_{0i} is the output force deviation
 257 caused by the damper accumulator, e_b is the Bingham sliding displacement and sgn designates the
 258 sign function. All the C_d , F_d and K_d parameters are related to the system's current intensity and
 259 can be expressed through:

$$\left. \begin{aligned} C_d &= C_{ds} + C_{dd}u \\ F_d &= F_{ds} + F_{dd}u \\ K_d &= K_{ds} + K_{dd}u \end{aligned} \right\} \quad (16)$$

$$\dot{u} = -\eta(u - I) \quad (17)$$

in which C_{ds} , F_{ds} and K_{ds} are the coefficient of viscous damping, the controllable Coulomb damping force and the equivalent axial stiffness of damper on the condition of zero electric field strength respectively, C_{dd} , F_{dd} and K_{dd} are the voltage sensitivity of viscous damping, controllable Coulomb damping force and equivalent axial stiffness of the damper respectively, η is the time coefficient of the damper's magnetic hysteresis response; the larger the value of η , the shorter the response time, with response time not taken into account explicitly in this paper; I is the applied current intensity, and u is the internal variable reflecting the relationship between model parameters and current intensity. There are eight unknown parameters in the mechanical model $(C_{ds}, F_{ds}, K_{ds}, C_{dd}, F_{dd}, K_{dd}, f_{0i}, \eta)$ which can be defined through parameter identification on actual test results for each MR damper. Performing this step, the mechanical model could be fully established.

3.2 Control algorithm of MR-TMD system

There are many options for the control algorithms to drive the MR damper system. Still, it is not the focus of this paper to research on the control algorithm efficacy. Therefore, the simplest and most practical fixed incremental control solution is adopted; this is abundant in engineering applications. The fixed incremental control algorithm dissipates energy^[33] through the limiting condition that when structural displacement and the TMD auxiliary mass velocity are in the same direction, then the adjustable Coulomb force is increased until it reaches the required current; on the contrary when the quoted response quantities are opposite, the adjustable Coulomb force is stepwise reduced until it shuts down. This principal is expressed through:

$$282 \quad F_d(t+1) = \begin{cases} \min(F_d(t) + \xi F_{d_{\max}}, F_{d_{\max}}) & Z_r \cdot \dot{Z} > 0 \\ \max(F_d(t) - \xi F_{d_{\max}}, F_{d_{\min}}) & Z_r \cdot \dot{Z} \leq 0 \end{cases} \quad (18)$$

283 where now $F_{d_{\max}}$, $F_{d_{\min}}$ are the maximum and minimum coefficients and ξ is the adjustment
 284 factor of the Coulomb damping force.

285 **3.3 Extending the fatigue crack initial life; the sub-modelling approach**

286 The key to enable improvements for the fatigue crack initial life of a railway steel bridge is to size
 287 accurately the dynamic stress response of the critical joint location after installing the MR-TMD
 288 control system. However, it is difficult to capture any stress updating directly by using off-the-shelf
 289 FE software because of the lack for example of control capabilities within such tools. Additionally,
 290 self-developed FE coding using sub-models of details (i.e. refined solid models) are hard to
 291 implement and could result in low computational efficiency. To this goal, a self-developed control
 292 program in FORTRAN was implemented and was combined to a multi-scale modelling scheme
 293 using commercial FE software in order to obtain the bridge critical point stress time-histories. The
 294 control model in FORTRAN is extracting sufficient modes from ANSYS in order to use the modal
 295 decomposition method for the response and control force calculations. As such, the bridge model in
 296 FORTRAN and ANSYS are expected to yield very close results. Fig.4 schematically depicts the
 297 methodology towards obtaining the controlled stress time-histories of the bridge with an added
 298 dynamic control system on. First, under railway load, the controlled actions and responses are
 299 evaluated through a FORTRAN package. These results are then fed as loading boundary input into
 300 the full bridge shell model, at the corresponding locations, of a commercial FE suite such as
 301 ANSYS. The subsequent calculations extract the controlled response at all bridge boundaries and
 302 particularly at the specific boundaries of the critical joint, for which a solid sub-model exists, a
 303 record is taken. The controlled stress-time history curve of the solid model is calculated after

304 combining data with the residual welding stress analysis. This information is further used as the
 305 basis for evaluating the impact of control interventions on the fatigue crack initial life of the
 306 bridge's most critical welded areas.

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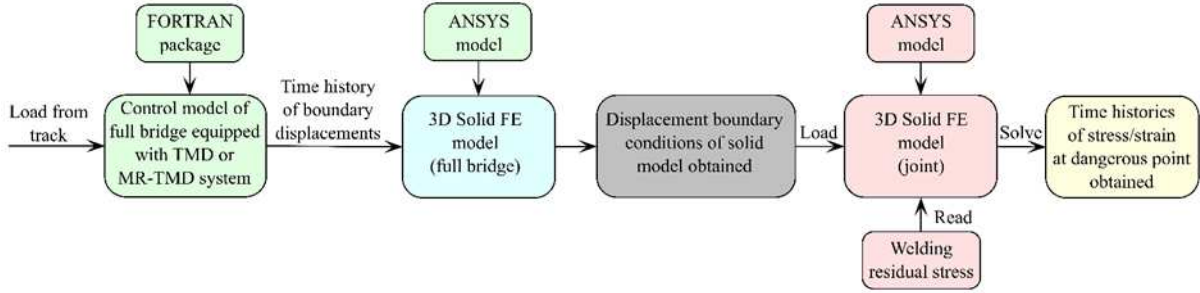


Fig.4 Flow chart for obtaining controlled stress time-history curve

315

3.4 Estimation of the fatigue crack initial life

316

Based on the microscopic study of material crack initiation, the critical plane method assumes a
 317 damage parameter, which here is the critical shear strain value of the material under cyclic loading.

318

The critical plane coincides with the fatigue crack initial plane of highest risk [5]. Using the six

319

strain time-history curves for the most severely loaded point, obtained in the previous section, the

320

orientation of the critical plane is searched through changing its angular position relative to the

321

reference plane in a step-like manner.

$$\begin{cases}
 \varepsilon_{z\dot{z}}(\theta, \Phi) = \sin^2 \Phi (\varepsilon_x \sin^2 \theta + \varepsilon_y \cos^2 \theta - \varepsilon_{xy} \sin 2\theta) + 0.5(\varepsilon_{xz} \sin \theta \sin 2\Phi - \varepsilon_{yz} \cos \theta \sin 2\Phi) + \varepsilon_z \cos^2 \Phi \\
 \varepsilon_{x\dot{z}}(\theta, \Phi) = 0.5(\varepsilon_x - \varepsilon_y) \sin 2\theta \sin \Phi - \varepsilon_{xy} \cos 2\theta \sin \Phi + \varepsilon_{xz} \cos \theta \cos \Phi + \varepsilon_{yz} \sin \theta \cos \Phi \\
 \varepsilon_{y\dot{z}}(\theta, \Phi) = 0.5 \sin 2\Phi (-\varepsilon_x \sin^2 \theta - \varepsilon_y \cos^2 \theta + \varepsilon_z + \varepsilon_{xy} \sin 2\theta) - \varepsilon_{xz} \sin \theta \cos 2\Phi + \varepsilon_{yz} \cos \theta \cos 2\Phi
 \end{cases} \quad (19)$$

323

where ε_x , ε_y , ε_z are the x , y , z -directional normal strains obtained by ANSYS, ε_{xy} , ε_{yz} ,

324

ε_{xz} are the xy , yz , xz -directional shear strains, $\varepsilon_{z\dot{z}}$, $\varepsilon_{x\dot{z}}$, $\varepsilon_{y\dot{z}}$ are the intermediate step

325

calculation parameters and $[\theta, \Phi]$ are the calculated angles.

326

Due to the random load, the shear strain and the normal strain on the plane of the material are

327

changing substantially, and they show continuous variation. Thus, a weight function is used to

328 determine the critical plane ^[33]. Thereafter, the normal and shear strain time-history curves of this
 329 critical plane are obtained by:

$$330 \quad \gamma(\theta', \Phi') = \sqrt{[\varepsilon'_{xz}(\theta', \Phi')]^2 + [\varepsilon'_{yz}(\theta', \Phi')]^2} \quad (20)$$

$$\varepsilon'_n(\theta', \Phi') = \varepsilon'_{z'z'}(\theta', \Phi')$$

331 In which, γ is the critical plane shear strain, ε'_n is the critical plane normal strain and $[\theta', \Phi']$
 332 are the critical interface angles calculated through the weight function method.

333 Meanwhile, the normal strain amplitude and shear strain amplitude are extracted by using the
 334 rain-flow counting method to make all fatigue estimations for the bridge. In order to effectively
 335 evaluate the multi-axial fatigue crack initial life, the Von-Mises criterion is adopted to reconfigure
 336 the normal and shear strain amplitudes into an equivalent strain amplitude on the critical plane; that
 337 is:

$$338 \quad \Delta\varepsilon_{eq}^{cr} / 2 = [\varepsilon_n^{*2} + \frac{1}{3}(\Delta\gamma_{max} / 2)^2]^{\frac{1}{2}} \quad (21)$$

339 where $\Delta\varepsilon_{eq}^{cr}$ is the multi-axis fatigue damage parameter of equivalent stain, ε_n^* is the normal strain
 340 amplitude obtained by the cycle counting, and $\Delta\gamma_{max}$ is the relevant shear strain amplitude. The
 341 fatigue strain variables include elastic and plastic parts, whereby the total strain can be written as:

$$342 \quad \Delta\varepsilon_a = \Delta\varepsilon_e + \Delta\varepsilon_p \quad (22)$$

343 where $\Delta\varepsilon_a$ is the total strain amplitude, $\Delta\varepsilon_e$ is the elastic strain amplitude, and $\Delta\varepsilon_p$ is the plastic
 344 strain amplitude. Subsequently, the amplitude of elastic and plastic strains can be expressed
 345 respectively by:

$$346 \quad \frac{\Delta\varepsilon_e}{2} = \frac{\sigma'_f}{E} (2N_f)^m \quad (23)$$

$$347 \quad \frac{\Delta\varepsilon_p}{2} = \varepsilon'_f (2N_f)^n \quad (24)$$

348 where σ'_f is the fatigue strength coefficient, ε'_f is the fatigue plastic coefficient, E is the
 349 material elastic modulus, m and n are the elastic and plastic fatigue strength index, and N_f is
 350 the cycle counts of fatigue life.

351 The fatigue initial life is assessed by the Manson-Conffin formula which considers in unity high
 352 cycle and low cycle fatigue^[34]. Considering the effect of the welding residual stress on the fatigue
 353 life, the average stress σ_m is introduced into the elastic strain, and the elastic fatigue strength
 354 coefficient σ'_f is modified to $\sigma'_f - \sigma_m$ ^[35]; this yields the modified Manson-Conffin formula:

$$355 \quad \frac{\Delta\varepsilon_{eq}^{cr}}{2} = \frac{\sigma'_f - \sigma_m}{E} (2N_f)^b + \varepsilon'_f (2N_f)^c \quad (25)$$

356 where $\sigma_m = \overline{\sigma_m} + 0.3\sigma_s$, $\overline{\sigma_m}$ is the mean value of calculated redistributed welding residual stress,
 357 and σ_s is the yield strength of steel.

358 The number of fatigue life cycles N_f of the railway steel bridge is obtained by substituting the
 359 equivalent strain amplitude into Eq.(24), while also the damage degree $D_i = 1 / \sum_{j=1}^n N_{f_j}$ is calculated
 360 for when a train crosses the bridge. Based on the Miner linear cumulative damage theory^[5], the
 361 fatigue crack initial life in any of the welded bridge joints for the combined complex loading –
 362 vibration control condition is designated by:

$$363 \quad D = \sum_1^k D_i \quad (i = 1, 2, \dots, k) \quad (26)$$

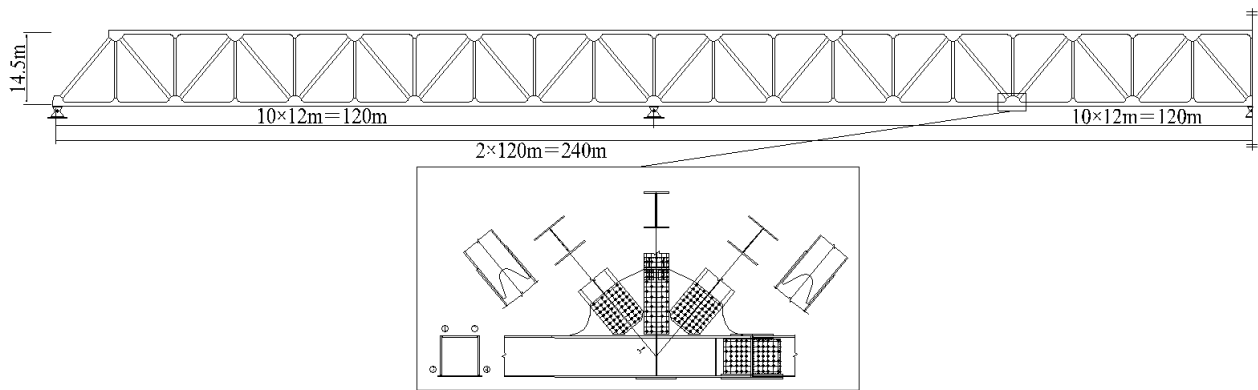
364 where k is the number of the train crossing bridge. Assuming that $D=1$ is used as the critical failure
 365 fatigue threshold, if D is greater than 1 fatigue occurs, while if D is less than 1, any fatigue damage
 366 is prevented^[36].

367 **4. Case study; the Poyang Lake railway bridge**

368 **4.1 Bridge background**

369 The Poyang Lake Railway Bridge, is a railway steel truss bridge located at the Tong-Jiu railway

370 freight line in China. The layout of its spans is 4×120 m, totalling 480 m in length. Each span is
 371 composed of 10 sections with each having 14.5 m height, 13 m width, and 12 m length, as
 372 illustrated in Fig.5, which schematically presents only half of the symmetrical railway truss bridge.
 373 The chord member of the truss is of box section with $0.8 \text{ m} \times 0.8 \text{ m}$ size. The material of the web
 374 member, the vertical and horizontal beam and the upper and lower flat of the truss is H-steel. In
 375 addition, the lower joint in the first span is 3.5 m long, 1.5 m high, and has a plate thickness of 0.04
 376 m. The joint is a welded integral joint with four fillet welds and each member is connected to it by
 377 bolts as shown in the detailed joint diagram inset to Fig.5.

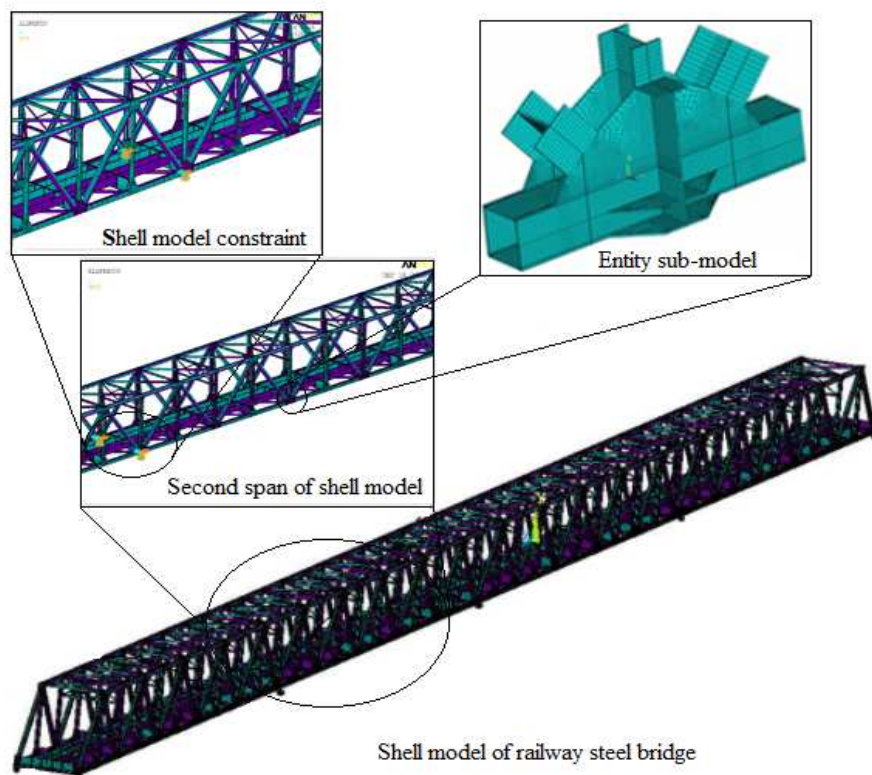


378
 379 **Fig.5 The schematic diagram of railway bridge structure**

380 **4.2 Multi-scale FE modelling of the bridge**

381 For any complex civil engineering structure such as the Poyang Lake Bridge, in order to perform its
 382 fatigue life estimation precisely, it is necessary to not only analyse the full bridge structure but also
 383 to obtain the accurate stress distributions locally. The approach where the full structure is simulated
 384 by a fine solid model would result in unreasonable calculating costs and low numerical efficiency.
 385 Thus, herein a hybrid scale approach with a coarse FE model for the full bridge and a fine solid
 386 model for its joints are established through the sub-model technique. For this variant, it takes nearly
 387 20 hours to obtain final results for one configuration, on a PC with WINDOWS 7 operating system,
 388 I7 CPU, 16 GB RAM memory, and 2TB high-speed SATA hard disk, However, if the full bridge is

389 fully modeled by a solid element model, this will result in hundreds of millions degrees-of-freedom,
390 that any typical home/office PC could not cope with. Fig.6 shows a schematic diagram of the
391 applied sub-model method philosophy. The welded thin steel plate of the steel bridge is simplified
392 as shell elements, and the ANSYS Shell63 unit is employed to set up the full-bridge model. This
393 results in a total of 106137 units that are meshed by using the quadrilateral element. Furthermore,
394 the corresponding thermal and mechanical analysis FE models of the joint are using the ANSYS
395 Solid 70 and Solid185 units respectively, and are meshed by using the high precision hexahedral
396 elements, resulting in a total of 28834 meshed elements for the one critical joint. The cross section
397 of the solid model is consistent with the design drawings of the steel bridge; those member bars
398 connected with bolts are simplified into rigid connection bars. In addition, in order to precisely
399 analyse the welding residual stress, the welded area of the box girder is divided in more scrutiny.



400
401

Fig. 6 Schematic diagram of railway steel bridge modelling based on sub-model method

402

403 **4.3 Load simulation**

404 **4.3.1 Welding residual stresses**

405 The FE simulation of the welding temperature field is using the double-ellipsoid heat source (Fig.7)
406 to establish the welding heat source effect. The welding parameters are same as the actual arc
407 welding parameters during construction. These are shown in Tab.1 [37]:

408 **Tab.1 Welding parameters of electric arc welding**

f_f	f_r	U_w (V)	I_w (A)	v (mm/s)	η (%)	a_1 (mm)	a_2 (mm)	b (mm)	c (mm)
1.33	0.67	270 V	10 A	6	70	5	10	3.5	5.0

409 The whole welding process is divided into the two stages, heating and cooling, and it is carried
410 out with the same speed of 6 mm/s as in the actual construction. There are four welds in the box
411 girder of the node, and the length of each weld is 4 m. The welding simulation is carried out in the
412 same manner as with the factory process; the heating time is 1720 s, the cooling time is 1500 s, and
413 the total welding time is 3220 s. Fig.8 shows the results of the temperature field on the joint at a
414 given time. It can be seen from the chart that the heat gradually moves along with time, and that the
415 welding temperature field distribution is non-uniform; the maximum temperature the heat source
416 reaches is 1829 °C, which is realistic representation of the actual welding temperature [38]; the
417 temperature decreases rapidly when the heat source passes through. Moreover, the isotherm near the
418 heat source is as prescribed close to an ellipse.

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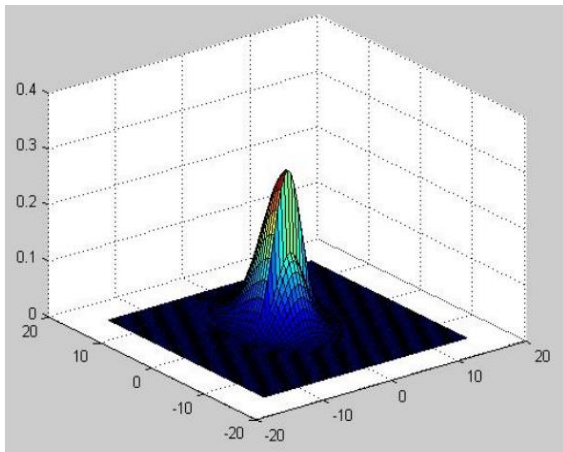
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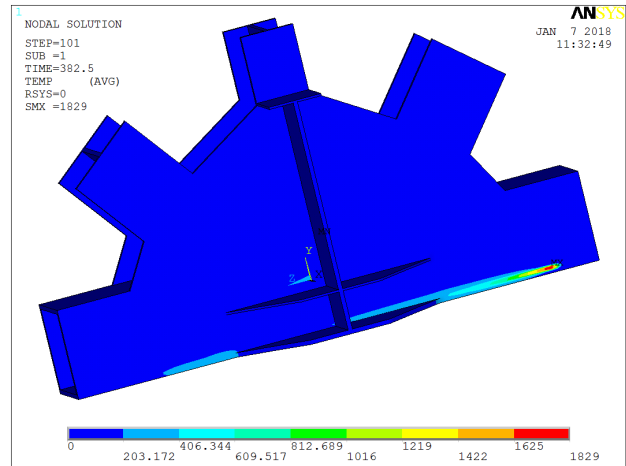
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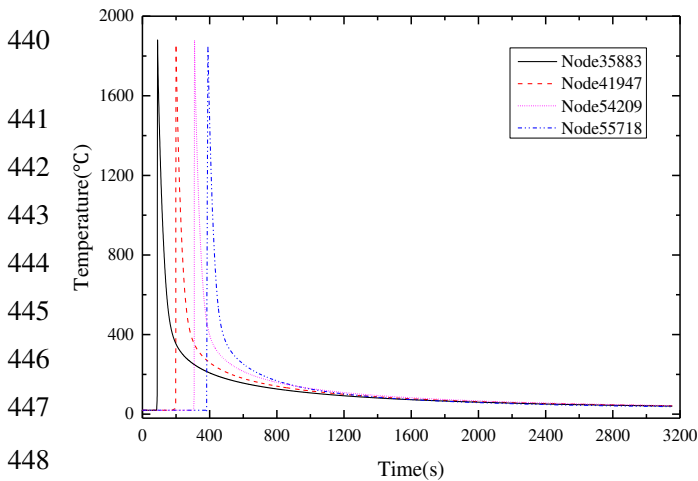


434 **Fig.7 Double-ellipsoid heat source model**

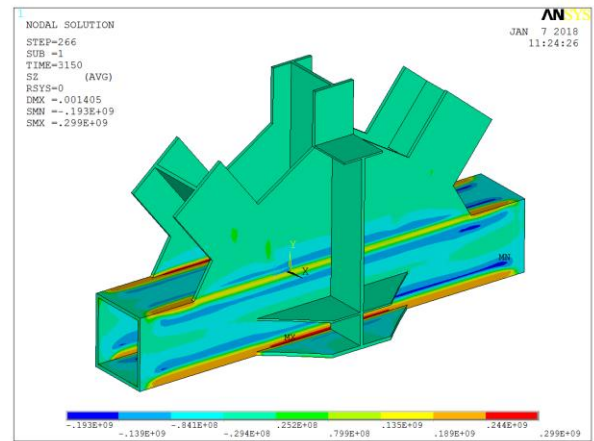


435 **Fig.8 Temperature distribution of nodes**

435 Fig.9 shows the temperature time-history curve of the four different points in the welded area of
436 the steel bridge joint during the welding process. It can be seen from the chart that in the heating
437 stage, the weld area temperature rises sharply at the welding instant, and the temperature reaches its
438 maximum value of around 1800 °C; in the cooling stage, the temperature drops rapidly, and
439 asymptotically tends to the room temperature.



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448 **Fig. 9 Temperature time-history curve**



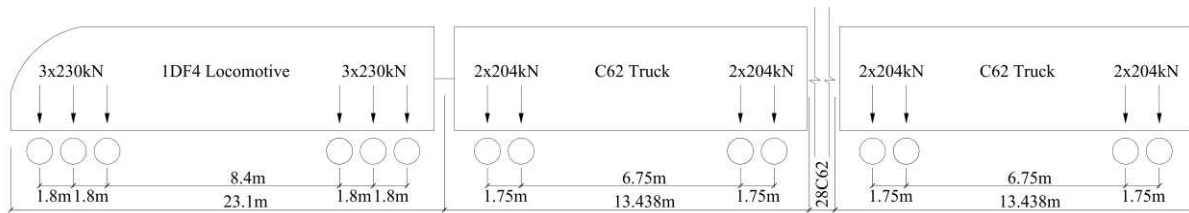
449 **Fig. 10 Nephogram of welded residual stress**

450 The residual stress field is analysed after the recovery of the temperature field. The temperature
451 field is applied as the load to the fine stress analysis numerical model, and the simulation of welding
452 residual stresses is achieved. In addition, the time course of the stress field analysis of the welding
453 residual calculation is the same as that of the temperature field analysis. Fig.10 shows the

454 nephogram of the welding residual stress, and as it can be seen from the chart these stresses are
 455 mainly distributed in the welds and their close vicinity. The welding residual stresses are tensile
 456 stresses, with a maximum value here of around 299 MPa. The location of the maximum welding
 457 residual stress is at the weld seam which connects the box girder and the node plate.

458 4.3.2 Train traffic load

459 According to the actual structure's engineering background, the Tong-Jiu line is a freight route.
 460 With an average of six trains per hour crossing the Poyang Lake Bridge, the speed of the
 461 characteristic train is 72 km/h. For all simulations, the interval time was selected as 0.02 s, making
 462 the total time for a train passing through the bridge 38.32 s. Reviewing actual information on
 463 realistic train traffic over the bridge, an indicative train formation with a DF4 locomotive and thirty
 464 C62 carriages was selected. For more details on the specific train grouping parameters one can refer
 465 to the Yang et al. [39]; Fig.11 shows the schematic diagram of the train formation.



466
467 **Fig.11 Schematic diagram of the train formation**

468 The track parameters are shown in Tab.2, and the average interval between adjacent track nodes
 469 are nearly 1.15 m; this is actually also the distance considered between nodes in the full bridge shell
 470 model.

471 **Tab.2 Track Parameters**

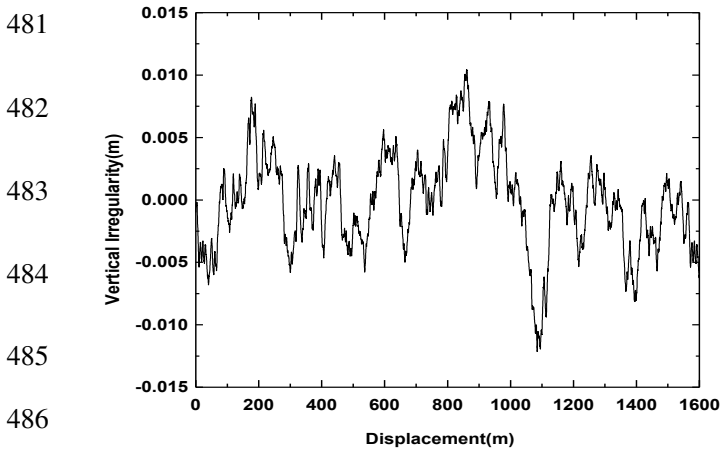
Parameters	k_{y1}	k_{y2}	k_{y3}	c_{y1}	c_{y2}	c_{y3}	k_H
	(MN/m)	(MN/m)	(MN/m)	(kN·s /m)	(kN·s /m)	(kN·s /m)	(MN/m)
Value	3.92	220	133	63	82	100	150

472 The power spectrum of Germany high interference track irregularity is used to simulate the train
 473 traffic loads due to its suitability for the freight trains in China. Tab.3 gives the track irregularity
 474 parameters herein used.

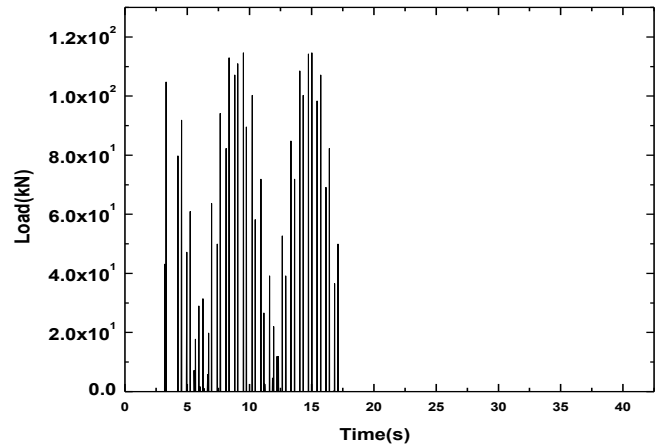
475 **Tab.3 Track irregularity parameters**

A_v ($m^2 \times rad/m$)	Ω_c (rad/m)	Ω_r (rad/m)
1.08×10^{-7}	0.8246	0.0206

476 The time-history curve of the vertical track irregularity is simulated by using a FORTRAN-coded
 477 program based on the triangular series superposition method [30]. Results can be shown in Fig.12.
 478 The load time-history curve acting on the nodes of the full bridge shell FE model is obtained
 479 through the vehicle-track coupling dynamic model developed. Fig.13 shows the case of the load
 480 time-history curve for the later considered critical node in the middle span.



487 **Fig.12 Time-history curve of vertical track irregularity**



487 **Fig.13 Time-history curve of load**

488 **4.4 Joint dynamic response; the vibration control scenario**

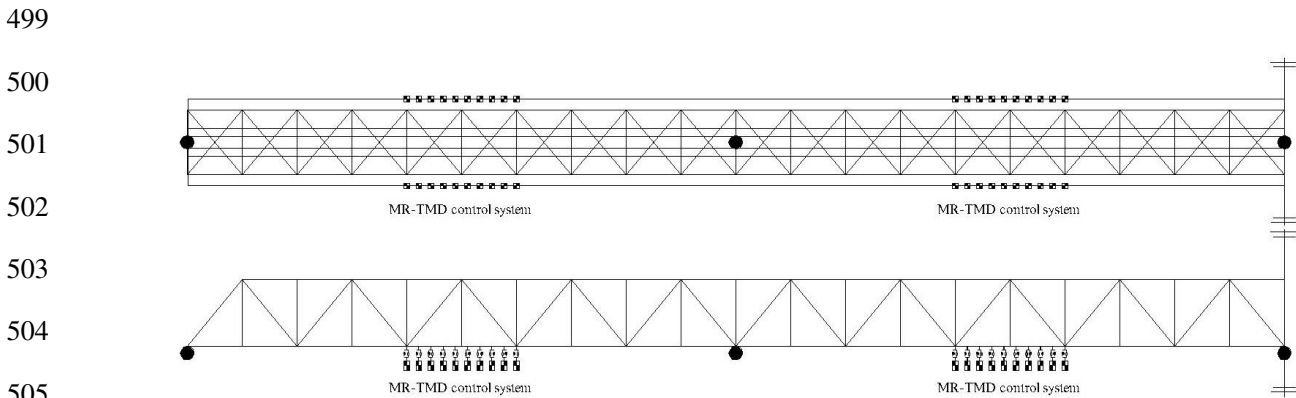
489 **4.4.1 Detailing of the MR-TMD control system**

490 Owing to the large vertical displacement response in the middle span of the railway bridge, the
 491 MR-TMD control system application is considered there, in order to obtain the best control
 492 performance effect against the train traffic load and without aesthetically affecting the bridge shape.
 493 To ensure that the MR-TMD installation does not affect the bridge shape and the trains passing over

494 the bridge, we use 20 mass damper units which are distributed uniformly along the middle span.
 495 Fig.14 presents the layout of the MR-TMD control system set on one of the spans of the actual
 496 bridge. Tab.4 gives the basic modelling parameters of the MR-TMD system. Additionally, for the
 497 passive TMD part the stiffness is 750kN/m, and the inherent damping is 1.2kNs/m.

498 **Tab.4 Parameters of each MR-TMD control system**

Mass block (kg)	Maximum force (KN)	MR diameter of cylinder (mm)	MR diameter of piston shaft (mm)	Effective length of piston (mm)
1200	100	140	70	120



506 **Fig.14 Layout of the MR-TMD control system in one of span of bridge; top: Front view; bottom: Side**
 507 **view**

508 4.4.2 Stress evaluation for the critical joint location

509 As discussed earlier, the accurate stress response on any joint of the railway bridge is obtained
 510 based on a sub-model realisation. Fig.15 gives the time-history of the vertical displacement at one
 511 node on the solid model boundary of the original structure from both the FORTRAN and ANSYS
 512 models. As it can be seen, the amplitude of the vertical displacement extracted in both cases are
 513 identical at 0.0326 m. Therefore, both models are consistent, giving an amplitude for vertical
 514 displacement that makes the fatigue damage easy to occur under the long time high frequency train
 515 traffic.

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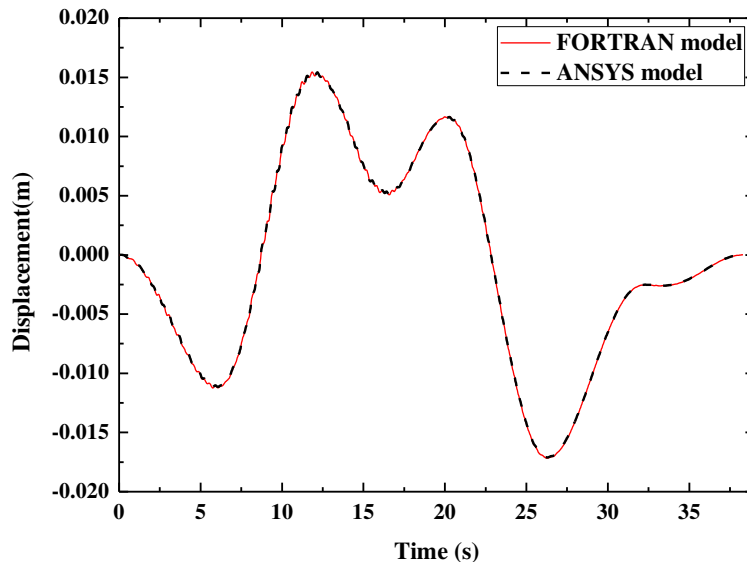


Fig.15 Time-history curve of vertical displacement

Fig.16 shows the Von-Mises stress contours after analysis of the original structure. What it shows is that the position connecting the edge of the gusset plate and the box girder has the largest welding stress and this is likely to produce stress concentration that could subsequently instigate the fatigue process. Therefore, this position is regarded as the most dangerous point to determine the fatigue crack initial life of the whole railway bridge and this is considered the same for all relevant later calculations.

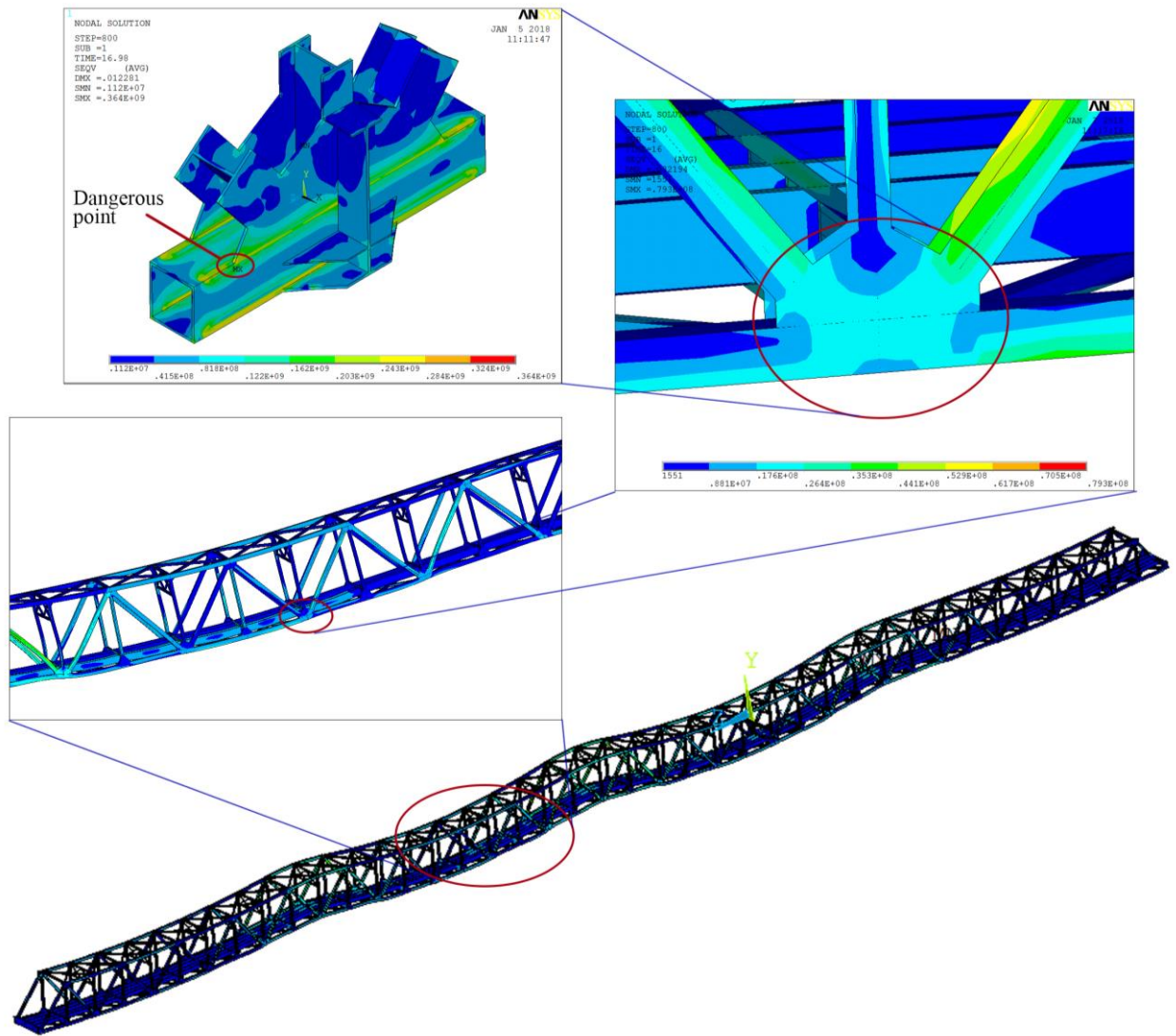
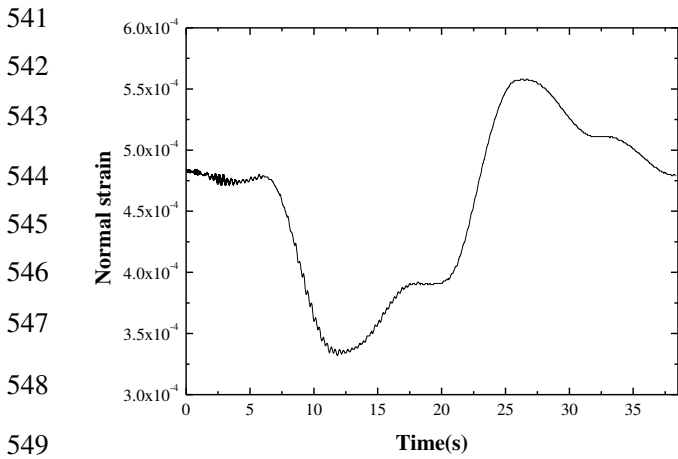


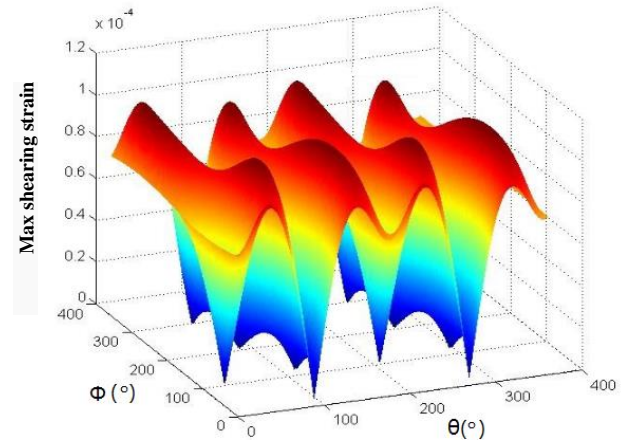
Fig.16 Von-Mises stress contour of the bridge

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Fig.17 presents the vertical normal strain time-history curve locally at the dangerous point. The maximum value of the normal strain is 560×10^{-6} , and this is the variable that gets the most influence on fatigue. The contribution of the vibration control method could impact on this and reduce it in a way that substantially prolongs the time to initiate any fatigue cracking condition.



550 **Fig.17 Vertical normal strain time-history curve**



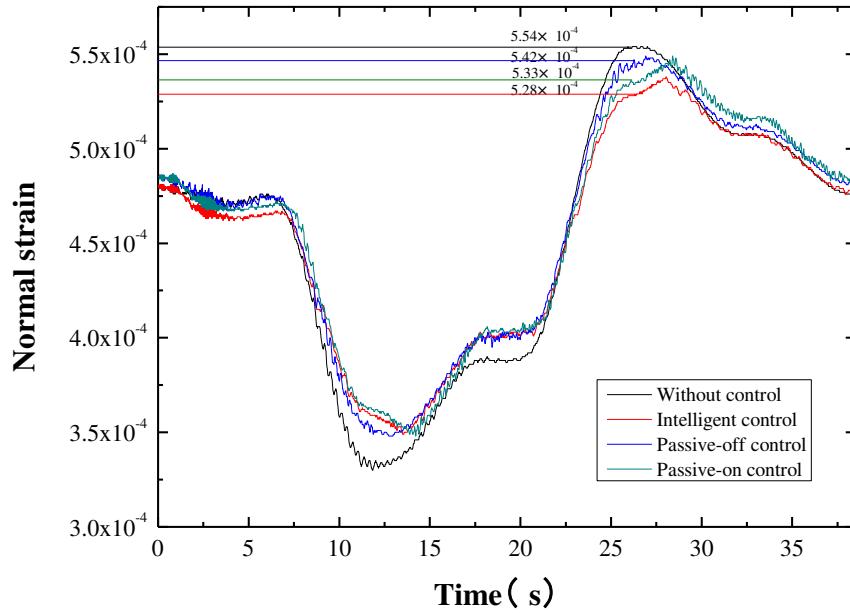
550 **Fig.18 Search results of critical damage plane**

551 Due to the welded joints of the steel bridge being in a multi-axial stress state, it is necessary to
552 determine the critical plane of the dangerous point for the fatigue life assessment based on the
553 fatigue damage critical plane method. Using the six components of the strain time-history curves, at
554 the point level as obtained in the previous section, the position of the maximum shear strain plane
555 derived parameter space at a given time is shown in the Fig.18. The orientation of the critical plane
556 calculated by the weight function method, for the illustration in hand, is $[\theta', \Phi'] = [262^\circ, 181^\circ]$.

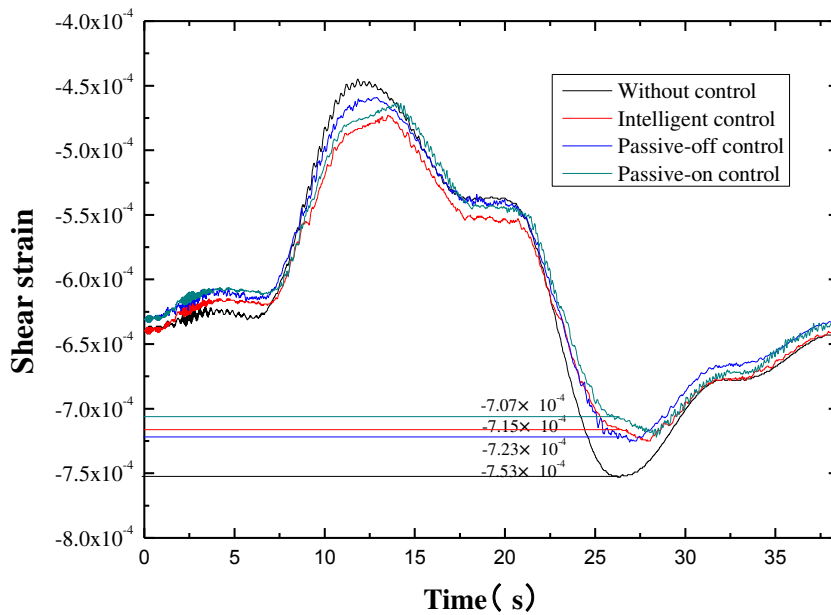
557 **4.4.3 Dynamic stress response under various control scenarios**

558 In order to analyse the effect of different control solutions on extending the fatigue crack
559 initiation life, first the dynamic response for the critical plane of the considered point under the
560 different control scenarios is obtained. The MR-TMD intelligent control system, with the fixed
561 increment algorithm prescribed, is used as the base towards accomplishing the vibration reduction
562 objective for the bridge response. For this, the fixed increment adjustment value is set at 20% of the
563 maximum current. At the same time, in order to research the effect of the control algorithm in the
564 MR-TMD system, two relevant uncontrolled cases were considered where the current was set
565 constant to the minimum and maximum values; these correspond to the MR damper being fully
566 closed or fully opened respectively. Subsequently, a designated state of passive-off control is

567 considered when functioning constantly at minimum current, while a passive-on control is
 568 considered when functioning repeatedly at the maximum current.



582 (a) Normal strain time-history curve



596 (b) Shear strain time-history curve

597 **Fig.19 The effect of different control option on strain**

598 Fig.19 shows the normal and shear strain time-history curves for the structure with passive-off,
 599 passive-on, intelligent control and without control when the freight train at 72 km/h speed passes
 600 through the railway bridge. It can be seen that, at the time 25.94 s, the normal strain of the structure
 601 with passive-off, passive-on, intelligent control and without control are 5.42×10^{-4} , 5.33×10^{-4} ,

602 5.28×10^{-4} , and 5.54×10^{-4} respectively. Similarly, at the time 26.24 s, the shear strain of the structure
603 with passive-off, passive-on, intelligent control and without control, are (in absolute values)
604 7.23×10^{-4} , 7.07×10^{-4} , 7.15×10^{-4} , and 7.53×10^{-4} respectively. It is observed that the bridge with all
605 control variants can show good control performance, giving evidence of potential for extending the
606 fatigue crack initiation life of the bridge. The control effect of the passive-on control is at instants
607 better than the intelligent control in terms of shear strain. However, holistically, the normal strain
608 with intelligent control is better than when with passive-on control, which also affects the fatigue
609 life through an equivalent strain measure.

610 Although these absolute strain values reduced only slightly there is a more significant effect on
611 amplitudes, on which the fatigue life directly depends on. Thus, it is also necessary to calculate the
612 strain cycle amplitude to judge the merits of the control effect.

613 **4.5 Sizing the extension of fatigue crack initiation life for the railway bridge**

614 **4.5.1 Calculation and analysis of equivalent damage parameter**

615 The normal strain and shear strain time-histories are accounted for by the rain-flow counting
616 method. The amplitude and frequency of each cycle are extracted as shown in the Fig.20. It can be
617 seen that the intelligent control has the best control effect among all different considered scenarios.

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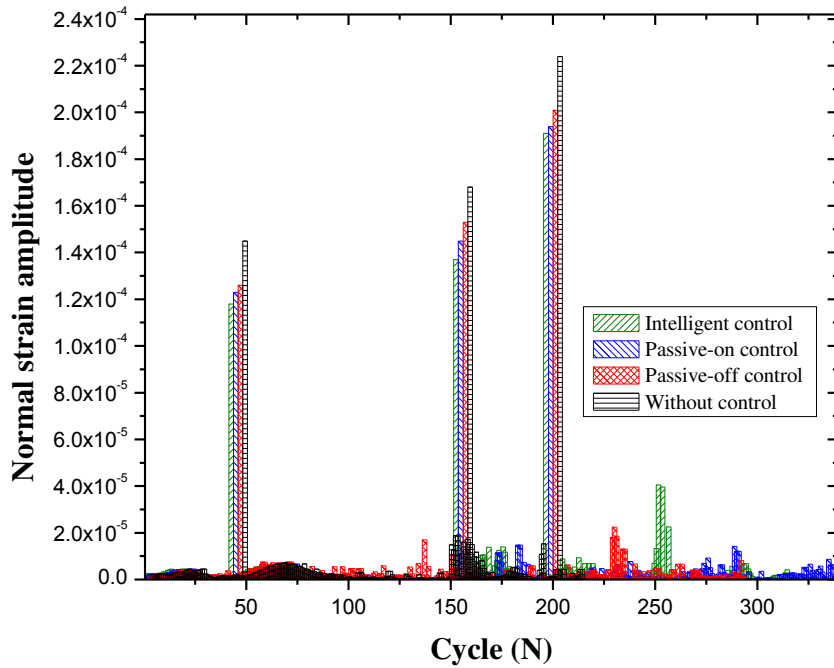
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(a) Diagram of normal strain amplitude

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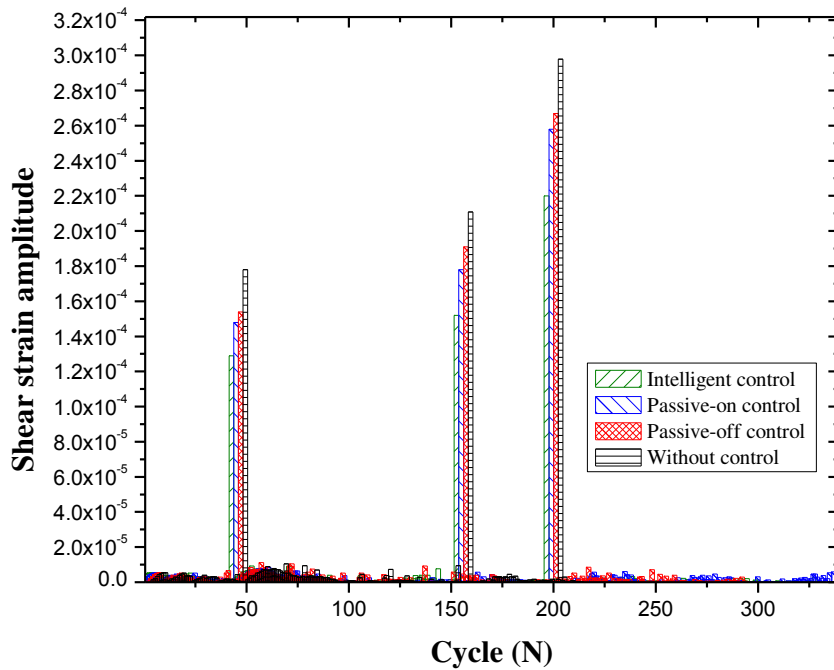
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(b) Diagram of shear strain amplitude

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Fig.20 Diagram of normal and shear strain amplitude

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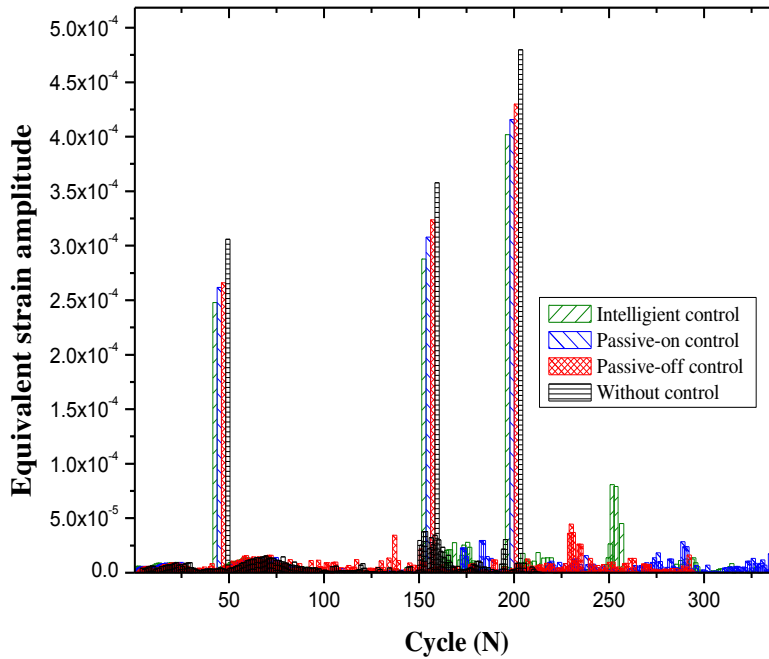
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According to Eq.6, the parameters of the normal and shear strain amplitudes on the critical plane are synthesized to the equivalent strain amplitude, and this is used as the damage control parameter. Fig.21 gives these equivalent strain amplitudes. It is shown that the maximum equivalent strain

652 amplitude of the structure without control is 4.80×10^{-4} , while for the passive-off, passive-on and
 653 intelligent control are 4.30×10^{-4} , 4.16×10^{-4} and 4.02×10^{-4} , respectively. This means that the
 654 maximum equivalent strain amplitude is reduced by 10.4%, 13.3% and 16.3% when compared to
 655 the original status. In addition, the rain-flow counting number of the original structure with
 656 passive-off, passive-on, intelligent control and without control are 293, 339, 333 and 211
 657 respectively. Although, there are differences in the number of cycles, the fatigue damage of the joint
 658 is produced only by the three larger strain amplitudes and all other cycles were found too small to
 659 cause any substantial damage. Thus, there is an evident control effect of the MR-TMD at all the
 660 aspects that affect the fatigue life later calculations.



669 **Fig.21 Diagram of equivalent strain amplitude**

671 4.5.2 Estimation of fatigue crack initial life

672 Finally, the Manson-Conffin formula of Eq.(25) is used in order to assess the fatigue crack
 673 initiation life of the bridge. For the basic involved parameters it was considered that $\sigma'_f - \sigma_m = 554$
 674 MPa, $b = -0.1632$, $c = -0.6824$, $\varepsilon'_f = 1.0118$ [40].

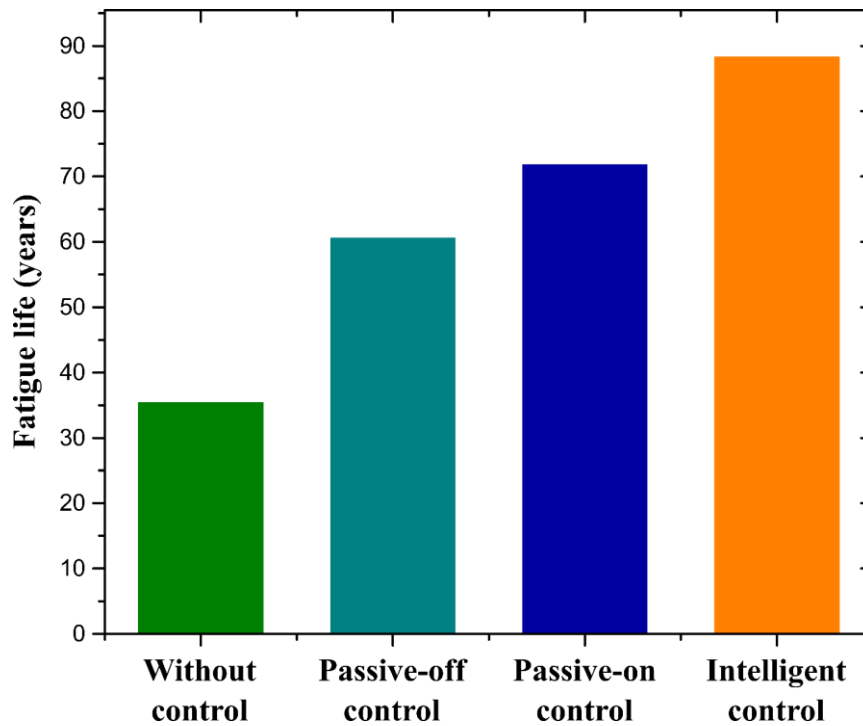


Fig.22 Comparison of fatigue life with various control

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According to the Miner's linear fatigue cumulative damage criterion mentioned in section 3.4, using the equivalent strain amplitude, the fatigue damage degree of the railway steel bridge without control is 5.367×10^{-7} , and the relevant number of cycles is 1863200. Similarly, the fatigue damage degree of the bridge structure with passive-off, passive-on and intelligent control are 3.137×10^{-7} , 2.648×10^{-7} and 2.153×10^{-7} , respectively while the associated number of cycles are 3187800, 3776400 and 4645300. According to the actual structural engineering details, the fatigue crack initiation life of the bridge is obtained as shown in the Fig.22. As it can be seen that the fatigue crack resistance of the bridge in-hand has improved to a high degree with all the various control options. Among them, the intelligent control solution brings the best results, whereby it raised the structural life estimate to 88.38 years, giving a relative increase, compared to the original structure, of the order of 150%.

690 **5. Conclusions**

691 This paper presented an analytical framework for sizing the fatigue life extension of railway steel
692 bridges when dynamically controlling their response. The tested of a real case bridge was employed
693 for showcasing the effectiveness and merit of the endeavour. The main outputs derived include

694 (1) The multi-scale analysis method devised for establishing the railway steel bridge model not only
695 meets the need of evaluating accuracy for the stress concentrations at the welded joints, but also has
696 high computational efficiency

697 (2) The MR-TMD intelligent control system proposed even when using an easy-to-implement fixed
698 increment algorithm can effectively control both the normal and shear strain amplitudes at the
699 fatigue-dangerous bridge point, reducing the equivalent strain amplitude to be used in fatigue
700 modelling.

701 (3) The degree of fatigue damage of the Poyang Lake railway steel bridges when using an
702 MR-TMD intelligent control system and under the combined realistic effect of train traffic load and
703 welding residual stress is 2.153×10^{-7} , the relevant number of cycles is 4645300, and the calculated
704 fatigue life 88.38 years. Compared to the original structure without any control this means an
705 increase by almost 150%. Therefore, the control effect of the MR-TMD control system makes it a
706 well-substantiated structural solution.

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712 Reference

- 713 [1] Committee on Fatigue and Fracture Reliability of the Committee on Structural Safety and Reliability of the
714 Structural Division. Fatigue reliability: Parts 1–4. *Journal of Structural Division, ASCE*, 1982,
715 **108**(ST1):3-88.
- 716 [2] Hansen B. Bridge Collapse Prompts Investigation, Raises Questions. *Civil Engineering, ASCE*, 2007,
717 **77**(9):12-14.
- 718 [3] Fisher JW. Fatigue and Fracture of Steel Bridges: Case Studies. Willey-Interscience, 1984.
- 719 [4] Chryssanthopoulos MK, and Righiniotis TD. Fatigue reliability of welded steel structures. *Journal of*
720 *Constructional Steel Research*. 2006, **62**(11):1199-1209.
- 721 [5] Shang D, and Wang D. Multi axis fatigue strength. Beijing: Science Press. 2007. (in Chinese)
- 722 [6] Li ZX, Chan THT, and Ko JM. Fatigue damage model for bridge under traffic loading: application made to
723 Tsing Ma Bridge. *Theoretical and Applied Fracture Mechanics*, 2001, **35**(1):81-91.
- 724 [7] Xu YL, Liu TT, and Zhang WS. Buffeting-induced fatigue damage assessment of a long suspension bridge.
725 *International Journal of Fatigue*, 2009, **31**(3):575-586.
- 726 [8] Guo T, Li AQ, and Wang H. Influence of ambient temperature on the fatigue damage of welded bridge decks.
727 *International Journal of Fatigue*, 2008, **30**(6):1092-1102.
- 728 [9] Righiniotis TD. Effects of increasing traffic loads on the fatigue reliability of a typical welded bridge detail.
729 *International Journal of Fatigue*, 2006, **28**(8):873-880.
- 730 [10] Tubino G, and Piccardo G. Tuned Mass Damper optimization for the mitigation of human-induced vibrations
731 of pedestrian bridges. *Meccanica*, 2015, **50**(3):809-824.
- 732 [11] Yang GQ. Large-scale magneto-rheological fluid damper for vibration mitigation: Modeling, testing and
733 control. PhD Dissertation University of Notre Dame, Indiana, USA. 2001.
- 734 [12] Fujino Y. Vibration, control and monitoring of long-span bridges-recent research, developments and practice

- 735 in Japan. *Journal of Constructional Steel Research*, 2002, **58**(1):71-97.
- 736 [13] Caetano E, Cunha Á, Magalhães F, and Moutinho C. Studies for controlling human-induced vibration of the
737 Pedro e Inês footbridge: Part 2. *Engineering Structures*, 2010, **32**(4):1082-1091.
- 738 [14] Andersson A, O'Connor A, and Karoumi R. Passive and Adaptive Damping Systems for Vibration Mitigation
739 and Increased Fatigue Service Life of a Tied Arch Railway Bridge. *Computer-Aided Civil and Infrastructure*
740 *Engineering*, 2015, **30**(9):748-757.
- 741 [15] Museros P, and Martinez-Rodrigo MD. Vibration control of simply supported beams under moving loads
742 using fluid viscous dampers. *Journal of Sound and Vibration*, 2007, **300**(1-2):292-315.
- 743 [16] Martínez-Rodrigo MD, Lavado J, and Museros P. Dynamic performance of existing high-speed railway
744 bridges under resonant conditions retrofitted with fluid viscous dampers. *Engineering Structures*, 2010,
745 **32**(4):808-828.
- 746 [17] Wang H, Tao T, Cheng H, and He X. Simulation Study on Train-Induced Vibration Control of a Long-Span
747 Steel Truss Girder Bridge by Tuned Mass Dampers. *Mathematical Problems in Engineering*, 2014, (2014),
748 506578:1-12.
- 749 [18] Luu M, Zabel V, and Könke C. An optimization method of multi-resonant response of high-speed train
750 bridges using TMDs. *Finite Elements in Analysis and Design*, 2012, **53**(6):13-23.
- 751 [19] Younesian D, Kargarnovin MH, and Esmailzadeh E. Optimal passive vibration control of Timoshenko beams
752 with arbitrary boundary conditions traversed by moving loads. *Proceedings of the IMechE, Part K: Journal*
753 *of Multi-body Dynamics*, 2008, **222**(2):179-188.
- 754 [20] Kahya V, and Araz O. Series tuned mass dampers in train-induced vibration control of railway bridges.
755 *Structural engineering and mechanics*, 2017, **61**(4):453-461.
- 756 [21] Yau JD. Train-Induced Vibration Control of Simple Beams Using String-Type Tuned Mass Dampers.
757 *Journal of Mechanics*, 2007, **23**(4):329-340.

- 758 [22] Lin CC, Wang JF, and Chen BL. Train-Induced Vibration Control of High-Speed Railway Bridges Equipped
759 with Multiple Tuned Mass Dampers. *Journal of Bridge Engineering*, 2005, **10**(4):398-414.
- 760 [23] Fu G, Gu J, Lourenco MI, Duan M, and Estefen SF. Parameter determination of double-ellipsoidal heat
761 source model and its application in the multi-pass welding process. *Ships and Offshore Structures*, 2014,
762 **10**(2):204-217.
- 763 [24] Zhao Q, Guo Z, Shen X, and Briseghella B. Test study on residual stress distribution of hybrid steel u-rib
764 stiffened plates. *Journal of Constructional Steel Research*, 2016, **121**:261-267.
- 765 [25] Armentani E, Pozzi A, and Sepe R. Finite-element simulation of temperature fields and residual stresses in
766 butt welded joints and comparison with experimental measurements, In: Proceedings of 12th Biennial ASME
767 Conference on Engineering Systems Design and Analysis (ESDA2014), Copenhagen, Denmark, 2014, pp.
768 V001T04A005.
- 769 [26] Cho DW, Song WH, Cho MH, and Na SJ. Analysis of submerged arc welding process by three-dimensional
770 computational fluid dynamics simulations. *Journal of Materials Processing Technology*, 2013,
771 **213**(12):2278-2291.
- 772 [27] Perić M, Tonković Z, Rodić A, Surjak M, Garašić I, Boras I, and Švaić S. Numerical analysis and
773 experimental investigation of welding residual stresses and distortions in a T-joint fillet weld. *Materials and*
774 *Design*, 2014, **53**:1052-1063.
- 775 [28] Xia H. *Vibration Engineering of Traffic Environment*. Beijing: Science Press. 2010. (in Chinese)
- 776 [29] Lei X. *New methods of orbital mechanics and Engineering*. Beijing: China Railway Press. 2002. (in
777 Chinese)
- 778 [30] Wang Y, Li Z, Wang C, and Wang H. Concurrent multi-scale modelling and updating of long-span bridges
779 using a multi-objective optimisation technique. *Structure and Infrastructure Engineering*, 2013,
780 **9**(12):1251-1266.

- 781 [31] Liljedahl CDM, Zanellato O, Fitzpatrick ME, Lin J, and Edwards L. The effect of weld residual stresses and
782 their re-distribution with crack growth during fatigue under constant amplitude loading. *International*
783 *Journal of Fatigue*, 2010, **32**(4):735-743.
- 784 [32] Qu WL, Qin SQ, Tu JW, Liu J, Zhou Q, Cheng H, and Pi YL. Intelligent control for braking-induced
785 longitudinal vibration responses of floating-type railway bridges. *Smart Materials and Structures*, 2009,
786 **18**(12), 125003.
- 787 [33] Brighenti R, Carpinteri A, Macha E, and Spagnoli A. Weight function method for determination of critical
788 plane position under multiaxial loading. In: 12th European Conference of Fracture (EFC12), Fracture from
789 Defects, Sheffield, UK, 1998, pp. 205-210.
- 790 [34] Ligaj B, and Szala G. Comparative analysis of fatigue life calculation methods of C45 steel in conditions of
791 variable amplitude loads in the low- and high-cycle fatigue ranges. *Polish Maritime Research*, 2012,
792 **19**(4):23-30.
- 793 [35] Shen W, Yan R, Xu L, Zhang X, and Geng L. Study on multi-axial fatigue damage of weld joints considering
794 residual stress. *Ship building of China*, 2014, **55**(2):67-76.
- 795 [36] Kim HI, Huh Y, and Seok CS. Fatigue Life Evaluation of an Actual Structure under the Irregular Loading
796 Using an Acceleration Test. *Solid State Phenomena*, 2007, **120**:9-14.
- 797 [37] ANSI/AASHTO/AWS D1.5M/D 1.5:2010. Bridge Welding Code.
- 798 [38] Liang W, Murakawa H, and Deng D. Investigation of welding residual stress distribution in a thick-plate
799 joint with an emphasis on the features near weld end-start. *Materials and Design*, 2015, **67**(67):303-312.
- 800 [39] Yang YB, Yau JD, and Wu YS. Vehicle-Bridge Interaction Dynamics: With Applications to High-Speed
801 Railways. World Scientific, 2004.
- 802 [40] Chen X, and Huang D. Low cycle fatigue of 14MnNbq steel welded joints. *Journal of Tianjin University*,
803 2001, 34(3):386-388. (in Chinese)