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Structural damage identification for railway bridges based on train-induced bridge responses and sensitivity analysis

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ABSTRACT

A damage identification approach using train-induced responses and sensitivity analysis is proposed for the nondestructive evaluation of railway bridges. The dynamic responses of railway bridges under moving trains composed of multiple vehicles are calculated by a train–bridge dynamic interaction analysis. Using the stiffness variation of the bridge element as an index for damage identification, the sensitivities of train-induced bridge responses to structural damage are analyzed and the sensitivity matrices are formed. By comparing the theoretical measurement responses of one measurement point in two different states, the damage indices of all elements are updated iteratively, and finally the absolute or relative damage is located and quantified. A three-span continuous bridge numerical example proves that the proposed dynamic response sensitivity-based FE model updating damage identification method is not only effective to detect local damage of railway bridges, but also insensitive to the track irregularity and the measurement noise.

1. Introduction

Damage in bridges can result in changes of their mechanical properties such as mass, stiffness, damping and boundary conditions, which can be reflected by changes in their global dynamic characteristics. The damage identification based on the global dynamic characteristics of structures has become currently a topic of very active research in civil and mechanical engineering. Various damage identification methods have been proposed by utilizing such parameters as natural frequencies [1,2], mode shapes [3,4], curvature mode shapes [5], modal damping [6], modal strain energies [7], frequency response functions [8] and stiffness or flexibility sensitivities [9,10]. Doebling et al. [11] comprehensively reviewed the literature, focusing on frequency-domain damage detection algorithms for linear structures. Zou et al. [12] summarized the methods on vibration-based damage detection and health monitoring for composite structures. Housner et al. [13] gave a good summary on state-of-the-art methods in control and health monitoring of civil engineering structures.

The fundamental principle of these methods is to compare the structural behavior in the damaged state with that in the undamaged state. In order to detect the damage locations and to determine the damage extents, it is necessary to model the undamaged state of the structure. A reliable method can be obtained by comparing the experimentally measured data

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of a structure in its initial state with those predicted by an initial mathematical model [14,15]. However, for an accurate model based damage assessment, often a lot of sensors and manual processing are needed, jeopardizing the online damage detection of structures in service.

From the view of structural online health monitoring, it is desirable to locate and quantify the damage directly from the time-domain dynamic responses of bridges under operating loads such as running vehicles. For this purpose, much research has been conducted. Liu and Chen [16] presented an inverse technique for identifying stiffness distribution in structures using the structural dynamic responses, where the sensitivity matrices of structural displacements with respect to the stiffness factors were calculated by Newton's method. Cattarius and Inman [17] detected the damage in smart structures from the time histories of structural responses. Chen and Li [18] and Shi et al. [19] proposed methods to identify both structural parameters and input loads from output-only measurements. Ling et al. [20] proposed an element level system identification method with unknown input with Rayleigh damping. Lu and Law [21, and Lu et al. [22] studied the features of dynamic response sensitivities under sinusoidal, impulsive and random excitations, and then used them in the structural damage identification. For large civil structures such as long-span bridges, it is usually difficult to excite them by impulsive or sinusoidal loads, so the passing vehicles are more suitable as excitation sources. Majumder and Manohar [23] proposed a time-domain approach for damage detection in bridges using both the vehicle response and the bridge response, in which the vehicle was considered as a single degree-of-freedom system with sprung and unsprung masses. Zhu and Law [24] studied the damage detection of simply supported concrete bridges, in which the moving forces and the damage indices are identified at the same time from the measured responses of multiple points.

In the above references, none is considering the damage detection of railway bridges from the dynamic responses due to passing trains composed of multiple vehicles. All papers also presume prior knowledge of the FE model in the undamaged state.

In this paper, a detailed train–bridge dynamic interaction model is established, in which the train is composed of multiple 4-axle vehicles with 10 degrees-of-freedom and the bridge is discretized by beam elements. The train-induced responses of the bridge in the damaged state are used as input data for damage identification and the response sensitivities with respect to the damage indices of the elements are calculated to establish the sensitivity matrix. Using the error between the measured response and the computed one as a minimization criterion, the sensitivity equation is solved by the least-squares method, and then the damage is located and quantified with the finite element model updating technique. In the proposed method, the influences of measurement noise and track irregularities on the analysis results are discussed. An example of a three-span continuous bridge numerical example proves that the local damage of railway bridges can be effectively identified using the train-induced response of a single measurement point.

2. Forward problem solution for train-induced bridge response

Since only the vertical response of the bridge is used in this study, a two-dimensional dynamic model of the train–bridge interaction system, composed of a train subsystem and a bridge subsystem, is established in the X–Z plane. The two subsystems are linked by the assumed wheel–track interactions.

The train subsystem model adopts the following assumptions:

(1) The train runs on the bridge at a constant speed.
(2) The train can be modeled as several independent vehicle elements. Each vehicle element is composed of a car body, two bogies, four wheel-sets and the spring–damper suspensions between the components.
(3) The car body, bogies and wheel-sets in each vehicle element are regarded as rigid components, neglecting their elastic deformations.
(4) The connections between a bogie and its wheel-sets are characterized by the first suspension system, which consists of springs and dampers with identical properties.
(5) The connections between a car body and its bogies are characterized by the second suspension system, which consists of springs and dampers with identical properties.
(6) The springs in vehicle elements are all linear, and the dampers all viscous.
(7) Each car body or bogie has 2 degrees-of-freedom in the Z and RY directions, while the longitudinal movement in the X direction is neglected.

Only the degree-of-freedom in the Z direction for the wheel-set is considered, thus each 4-axle vehicle element has 10 degrees-of-freedom (see Fig. 1).

Two-dimensional beam elements are used to model the bridge. In structural dynamics, the determination of the damping matrix is often difficult. The usual solution for this problem is to adopt the classical Rayleigh damping theory [25], in which the damping matrix $\mathbf{C}_b$ is expressed as a linear combination of the bridge mass matrix $\mathbf{M}_b$ and the stiffness matrix $\mathbf{K}_b$:

$$\mathbf{C}_b = \alpha\mathbf{M}_b + \beta\mathbf{K}_b \tag{1}$$

with $\alpha = 4\pi((\xi_1 f_1 f_2^2 - \xi_2 f_1^2 f_2)/(f_2^2 - f_1^2))$; $\beta = (1/\pi)((\xi_2 f_2 - \xi_1 f_1)/(f_2^2 - f_1^2))$.where f_1 and f_2 are the first- and the second-order natural frequencies (Hz). ξ_1 and ξ_2 are the first- and the second-order damping ratios of the bridge, respectively.