

一種考慮橋梁-土壤互制下高速鐵路橋梁的物理識別計算
A Physical Identification Algorithm for High-Speed Railway Bridges with Consideration of Bridge-Soil Interaction

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Abstract

This paper intends to develop a physical identification algorithm for high-speed railway bridges under braking and acceleration conditions of a single moving vehicle, and to investigate the dynamic characteristics of the railway bridges. The bridge-soil interaction is considered, and the dynamic interaction of the bridge-foundation with the underlying soil is taken into account. A linear model is used for the pier while an equivalent foundation model is chosen for the pile-soil system. The proposed algorithm can extract the actual system parameters rather than the mode ones. The parameters in the algorithm are more meaningful and useful. The proposed method provides not only the prediction of dynamic responses but also the determination of actual dynamic characteristics for the interaction system. Therefore, the actual bottom shear forces of piers also are able to be predicted, which are more helpful for the safety evaluation of high-speed railway bridges.
Keywords: physical identification, bridge-soil interaction, high-speed railway bridges, single moving vehicle

摘要

本文旨在發展一物理識別計算法，以識別出高速鐵路橋梁於受單一移動載具煞車力與加速度力作用下之動態參數。此橋梁系統將考慮橋梁-土壤互制，而此動態互制考慮橋梁基礎與周圍土壤間之動態互制作用。橋柱視為線性模型，橋梁基礎以等值基礎彈簧模擬之。此法有別於一般之系統識別理論，因一般之系統識別方法僅能識別出模態參數，但具有實際物理意義的系統參數則更具意義與實用性。本文所提出之識別方法，不但可提供互制系統動態反應之預測，且可獲得互制系統之實際動態參數，據此實際下傳至橋柱底之剪力即可被預測。此實有助於高速鐵路橋梁橋體之安全性評估。
關鍵詞：物理識別，橋梁-土壤互制，高速鐵路橋梁，單一移動載具

I. INTRODUCTION

The investigation of dynamic behavior of railway bridges caused by the moving vehicles has been a subject of research as early as the mid-nineteenth century. In the early research works, moving vehicles traveling along a bridge were usually modeled as moving loads or rolling masses [1, 2]. In the recent studies, the dynamic responses of railway bridges subjected to high-speed trains loading has been an interested for many engineers. Those studies focus on numerical simulations, including the effect of train mass inertia, coupling with the train cars suspension system, vehicle-bridge interaction element for modeling the vehicle-bridge interaction, dynamic condensation method to analyze the impact effects of simply supported beams, and the three-dimensional FEM to simulate the soil vibrations due to the moving high-speed train across bridges [3-9]. However much progress has been made recently in the development of analytical procedures for evaluate the dynamic response of high-speed railway bridges under moving vehicles loading. Application of

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such procedure is dependent on the incorporation of representative soil behavior in the analysis, also system identification rarely. In fact, system identification is an inverse problem of structural dynamic analysis. It involves the determination of the estimation of structural parameter values and provides the dynamic response prediction of structures on the basis of measured responses of structures under known disturbances [10].

Whenever a foundation element moves against the surrounding soil, stress wave originate at the contact surface and spread outward. These waves carry away some of the energy transmitted by the foundation into the soil. The magnitude of this damping depends on several factors, i.e. the frequency of excitation, the geometry of soil-foundation system, and the stress-strain characteristics of the soil. As a matter of fact, Knowledge on the subject of dynamic soil-structure interaction has derived primarily from studies for buildings and nuclear containment structures on mat foundation [11]. Although the dynamic response of pile supported bridge or similar structures has been the issue of some research efforts, much is yet to be learned on the crucial problem parameters on the response of bridges foundation on piles and pile groups in multi-layered soil [12,13].

In short, the usefulness of the theoretical analysis is limited by the degree of realistic representation of the formulated mathematical models. Obviously, a logical prelude to investigate the dynamic behavior of a high-speed railway bridge under a single moving vehicle braking and acceleration conditions is through the application of system identification techniques. This study intends to develop a physical identification algorithm to investigate the dynamic properties of the high-speed railway bridge in which bridge built on soft soil. The bridge piers are founded on piles embedded in the same site condition. The dynamic bridge-soil interaction is taken into account and is called “Interaction-system”. A linear model is used for the pier while a “Winkler beam” model is chosen to describe the foundational behavior of the bridge piers are founded on piles.

Unlike many other identification methods that generally identify the modal parameters, this paper investigates an identification process of a high-speed railway bridge with consideration of bridge-soil interaction is able to identify the actual system parameters. The parameters in the algorithm are more meaningful and useful. The reason is understandable. In the proposed method provides not only the prediction of dynamic responses but also the determination of actual dynamic characteristics for the interaction system. Therefore, the actual bottom shear forces of piers also are able to be predicted, which are more helpful for the safety evaluation of high-speed railway bridges.

In conclusion, the comparisons between the identified and measured responses of the high-speed railway bridge under a single moving vehicle braking and acceleration conditions show that the developed technique is feasible for identifying the interaction system.

II. MOTION EQUATION

A typical 3-span high-speed railway bridge under a single moving vehicle braking and acceleration conditions, and built on soft soil, as shown in Fig. 1, is considered. The deck is rigid and the substructure is assumed to be linear on account of the braking and acceleration force of the single moving vehicle is small number. Meanwhile, the bridge-pier system involves a one-column bent founded on a group of 5 rigidly-capped piles. A pile is deemed as a beam on “Winkler Foundation”. The role of pile-soil interaction is played through a set of continuously distributed soil-springs and soil-dashpots [11]. Those springs and dashpots connect the pile to the free-field soil. The motion of soil serves as the support excitation of the pile-soil system.

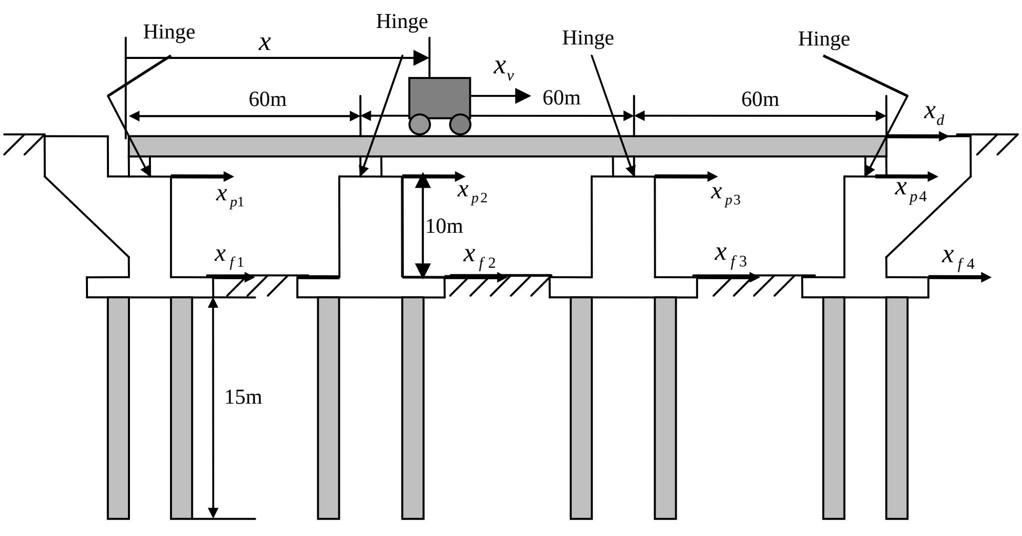

Fig. 1 High-speed railway bridge under braking condition of a single moving vehicle

Accordingly, the interaction system equation of motion can be expressed as

$$-\left(\frac{m_{pj}}{2}+m_{fj}\right)\ddot{x}_{fj}-R_{fj}\left(\dot{x}_{fj},x_{fj}\right)+R_{pj}\left(\dot{x}_{pj}-\dot{x}_{fj},x_{pj}-x_{fj}\right)=\{0\} \quad j=1\sim4 \tag{1}$$

$$-\frac{m_{pj}}{2}\ddot{x}_d-R_{pj}\left(\dot{x}_d-\dot{x}_{fj},x_d-x_{fj}\right)+f_j=\{0\} \quad j=1\sim4 \tag{2}$$

$$m_d\ddot{x}_d+\sum_{j=1}^{4}f_j=-m_v(\ddot{x}_v-\ddot{x}_d)\delta(x-vt) \tag{3}$$

in which m_{fj}, x_{fj} and R_{fj} are the mass, displacement and restoring force of the j-th foundation, respectively; m_{pj}, x_{pj} and R_{pj} are the lumped mass, displacement and restoring force of the j-th pier head, respectively; m_d and x_d are the mass and displacement of the bridge deck, respectively, the term $-m_v(\ddot{x}_v-\ddot{x}_d)$ present the contact forces acting between the wheels of the single moving vehicle and rail at the point x of the bridge, f_j is the internal force at the j-th hinge support.

There storing forces of foundation and substructure are respectively written as

$$R_{fj}\left(\dot{x}_{fj},x_{fj}\right)=C_{fj}\dot{x}_{fj}+K_{fj}x_{fj} \tag{4}$$

$$R_{pj}\left(\dot{x}_{pj}-\dot{x}_{fj},x_{pj}-x_{fj}\right)=C_{pj}\left(\dot{x}_{pj}-\dot{x}_{fj}\right)+K_{pj}\left(x_{pj}-x_{fj}\right) \tag{5}$$

where C_{pj} and K_{pj} represent the damping coefficient and stiffness of the j-th column pier, respectively, while C_{fj} and K_{fj} are the equivalent “dashpot” and “spring” coefficients corresponding to the j-th foundation. Physically, K_{fj} reflects the stiffness and inertia characteristics of the pile-soil system, while C_{fj} expresses the energy loss in the system.

III. EQUIVALENT FOUNDATIONAL STIFFNESS AND DAMPING COEFFICIENT

To determine the equivalent foundational stiffness at foundation, i.e. equivalent springs are played through the continuously distributed soil-springs at foundation, K_{fj}, one may obtain the horizontal-displacement profile, $y_s(0)$, of the pile subjected to a statically horizontal load of magnitude P_0. This coefficient is then directly computed as $P_0/y_s(0)$. The estimation of such a displacement profile can be done through the application of finite element method. At a particular node i, the reactive force from the surrounding soil was $K_h y_s(0)$. Note that K_h is constant of the soil-spring and is given as

$$K_h=k_hA_h \tag{6}$$

in which k_h is local subgrade reaction coefficient and A_h indicates the effective area. In the analysis, this area is computed by

$$A_h=b\left(\frac{l_{i-1}}{2}+\frac{l_i}{2}\right)=\frac{1}{2}b\left(l_{i-1}+l_i\right) \tag{7}$$

where l_{i-1} and l_{i+1} are length of elements adjacent to node i and b is pile diameter.

The equivalent foundational damping coefficient, i.e. equivalent dashpots done through the continuously distributed soil-dashpots at the foundation, is computed from the values of radiation damping and material damping distributed along piles. Radiation or geometric damping represents the radiation of energy by wave spreading geometrically away from the pile-soil interface. Its coefficient c_r can be obtained from appropriate wave propagation problem [14]. On the other hand, material damping refers to the hysteretic dissipation of energy in the soil. Its coefficient c_m is a function of the effective shear strain. The estimation of such strain was proposed by Kagawa and Kraft [15]. Once it is known, the hysteretic damping ratio, $\beta=\beta(z)$, in the soil can be assessed by widely available experimental data in the form of damping-versus-strain curve [16].

The radiation-damping coefficient associated with a circular cross section of diameter being b is expressed as [17]

$$c_r=2b\rho V_s\left\{1+\left[\frac{3.4}{\pi(1-\upsilon)}\right]^{5/4}\right\}\left(\frac{\pi}{4}\right)^{3/4}\left(\frac{\pi fb}{V_s}\right)^{-1/4} \tag{8}$$

in which $\rho=$ soil mass density, $V_s=$ S-wave velocity, $\omega=$ excitation frequency, and $\upsilon=$ Poisson’s ratio. Due to the generation of surface type waves instead of plane-strain body wave at shallow depths, i.e. z less than $2.5b$, the above equation should be modified as

$$c_r=4b\rho V_s\left(\frac{\pi}{4}\right)^{3/4}\left(\frac{\pi fb}{V_s}\right)^{-1/4} \tag{9}$$

The material damping coefficient at pile depth z related to the hysteretic damping ratio, β, is expressed as [17]

$$c_m\approx 2k(z)\frac{\beta}{\omega} \tag{10}$$

in which $k(z)$ is a secant modulus defined as the ratio of the static local soil reaction against a unit length of pile over the corresponding pile deflection, and can be obtained from the local soil Young’s modulus E_s as

$$k(z)=\delta E_s(z) \tag{11}$$

The coefficient δ is a function of the type of soil profile, the type of head loading and the ratio of pile stiffness to soil stiffness, E_p/E_s. Guidance for the selection of δ is provided in the literature [17]. It is also noted that

$$E_s = 2\rho V_s^2 (1+\upsilon)^2 \qquad (12)$$

Therefore, the equivalent foundational damping coefficient at the j-th foundation is evaluated by

$$C_{fj} = \int_0^l (c_m + c_r)\gamma_s^2(z)dz \qquad (13)$$

in which $\gamma_s(z) = y_s(z)/y_s(0)$, l is the pile length.

Finally, it is also noted that the equivalent foundational damping ratio of the j-th foundation is given by

$$\xi_{fj} = \frac{\omega}{2K(f)} = \frac{\pi f C_{fj}}{K(f)} \qquad (14)$$

where $K(f)$ denoted the dynamic equivalent stiffness at the foundation, i.e. the pile-head equivalent "spring" coefficient, and $f = 2\pi/\omega$ [13,15,17].

IV. IDENTIFICATION PROCESSES

When the accelerations of deck, piers, and foundations are measured, the system identification of the structure can be proceeded. Many system identification techniques are classified as output-error methods [18]. The system parameters are obtained by minimizing the discrepancy between the recorded responses and the theoretical responses of the system. The parameter value so evaluated is called an optimal.

The performance of identification starts with a fixed initial value of C_{p2}. Through several times of minimization process, an optimal K_{p2} is obtained. Then, K_{p2} is fixed at this value and the minimization process is proceeded to again an optimal C_{p2}. Meanwhile, the corresponding system parameters C_{p1} and K_{p1} are yielded. This constitutes one cycle of identification. A few cycles may be needed to ensure the convergence of an error index.

The above minimization process is further illustrated as follows. All available data are classified into four groups, i.e. piers, and foundations, and are called "partial measure-of-fit". When Eqs. (3) and (5) are substituted into Eq.(2), and using the set of data, i.e. the data is response of piers head (=deck) and foundations at time $= i\times\Delta t$, respectively, in which Δt is a time interval or time step. Now, we define the partial measure-of-fit for piers as

$$e_p = \sum_{i=1}\left[\begin{array}{l}\left(\frac{m_{p1}+m_{p2}+m_d}{2}\right)\ddot{x}_d + \\ \frac{m_v}{2}(\ddot{x}_v - \ddot{x}_d)\delta(x-vt) \\ + \sum_{j=1}^{2} C_{pj}(\dot{x}_d - \dot{x}_{fj}) + \\ \sum_{j=1}^{2} K_{pj}(x_d - x_{fj})\end{array}\right]^2 \qquad (15)$$

The optimal values of C_{p1} and K_{p1} can be obtained by solving

$$\frac{\partial e_p}{\partial(K_{p1})} = 0 \qquad \frac{\partial e_p}{\partial(C_{p1})} = 0 \qquad (16)$$

The associated value of e_p is calculated from Eq.(15). Similarly, a partial measure-of-fit for the j-th foundation is obtained by the Eqs. (1), (4) and (5) as

$$e_{fj} = \sum_{i=1}\left[\begin{array}{l}\left(\frac{m_d}{2}+m_{fj}\right)\ddot{x}_{fj} + C_{fj}\dot{x}_{fj} + K_{fj}x_{fj} \\ - C_{pj}(\dot{x}_d - \dot{x}_{fj}) - K_{pj}(x_d - x_{fj})\end{array}\right]^2 \quad j = 1 \sim 4 \qquad (17)$$

in which the update values of C_{pj} and K_{pj} are used, i.e. obtained by Eqs.(15) and (16). Then the optimal values of C_{fj} and K_{fj} can be determined by solving

$$\frac{\partial e_{fj}}{\partial(C_{fj})} = 0 \qquad \frac{\partial e_{fj}}{\partial(K_{fj})} = 0 \qquad (18)$$

Therefore, among those sets of C_{pj}, K_{pj}, C_{fj} and K_{fj}, the sets that yield a minimum partial measure-of-fit, i.e. piers and foundations, are regarded as the solutions. In other words, the C_{pj}, K_{pj}, C_{fj} and K_{fj} are the identified parameter values.

To compare the convergence and accuracy of the identification process, the error index is defined as

$$EI = \left\{\frac{\int_0^t \left[(\ddot{x}_{fj})_r - (\ddot{x}_{fj})_t\right]^2 dt}{\int_0^t \left[(\ddot{x}_{fj})_r\right]^2 dt}\right\}^{1/2} \qquad (19)$$

where $(\ddot{x}_{fj})_r$ is the recorded or measured acceleration response and $(\ddot{x}_{fj})_t$ is the theoretical or identified response. The latter is calculated from the identified parameter values and the recorded input. We noted that fj refers to the j-th foundation. When fj is replaced by d, Eq. (19) may yield the error index at deck.

V. NUMERICAL EXAMPLE

Numerical data include

1. Piers and deck:

$m_{p1}/2 = m_{p4}/2 = 96.0\times10^3 kg$,

$m_{p2}/2 = m_{p3}/2 = 48.0\times10^3 kg$,

$m_d = 2066.9\times10^3 kg$,

$K_{p1} = K_{p4} = 221.31 MN/m$,

$K_{p2} = K_{p3} = 110.66 MN/m$,

2. Foundations:

$m_{f1} = m_{f2} = m_{f3} = m_{f4} = 43.2\times10^3 kg$,

$K_{f1} = K_{f2} = K_{f3} = K_{f4} = 236.97 MN/m$, and

3. Vehicle:

$m_v = 64.0 \times 10^3 kg$,

$Max.\ddot{x}_v = \pm 3m/\sec^2$.

The damping ratio of reinforced concrete is assumed to be 5%. Meanwhile, the equivalent foundational damping ratio is given by Eqs. (8)~(14) and is equal to $\zeta_{fi} = 0.121 + 0.072 f^{3/4}$. By utilizing the concept of composite modal damping ratio and Rayleigh damping [19-21], all damping coefficients are estimated. Their values turn out to be $C_{p1} = C_{p4} = 1744\ kN.s/m$,

$C_{p2} = C_{p3} = 872\ kN.s/m$,

$C_{f1} = C_{f4} = 2381\ kN.s/m$,

$C_{f2} = C_{f3} = 2204\ kN.s/m$.

The linearity between restoring forces and displacements of pier 1 and pier 2, i.e. on the first and second pier head, are observed from Fig. 2, Fig. 3, respectively. The restoring forces-displacements relationship at foundation 1 and foundation 2, i.e. at the first and second foundation, is delineated in Fig. 4 and Fig. 5, respectively. They are approximately linear.

The responses of the interaction system under the braking and acceleration conditions of a single moving vehicle, which are calculated by Newmark's linear acceleration method, i.e. with $\beta = 1/6$ and $\gamma = 1/2$, for performing the step-by-step integration, i.e. time step=0.01sec. Therefore, the time-history responses, i.e. accelerations, velocities, and displacements, of the deck and foundations can be analyzed, respectively. Those acceleration values are treated as the measured ones.

In the first cycle of identification, the initial value of C_{p2} is set to be zero arbitrarily. The partial measure-of-fit for piers, i.e. e_p, as a function of K_{p2} is presented in Fig. 6, which reveals that the least squares estimate of K_{P2} is $110.00\ MN/m$. Then K_{P2} is fixed at this value and the minimization process is performed. As illustrated in Fig. 7, the optimal estimate of C_{p2} is $900\ kN.s/m$. In the meantime, the identified parameter values of pier 1 are $C_{p1} = 1707.0\ kN.s/m$, $K_{p1} = 222.18\ MN/m$. Those optimal values of related with foundations can be estimated by Eqs. (17) and (18), which are $C_{f1} = 2345.0\ kN.s/m$, $K_{f1} = 237.64\ MN/m$, $C_{f2} = 2259.0\ kN.s/m$, and $K_{f2} = 235.32\ MN/m$.

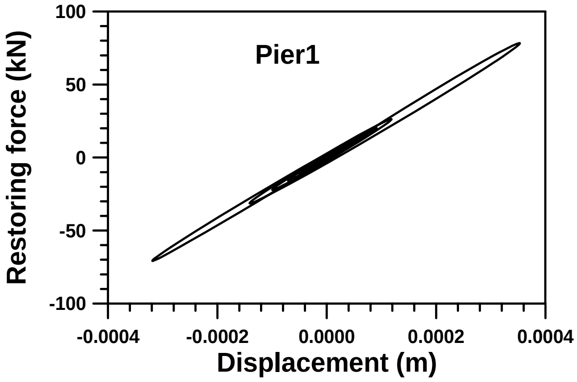

Fig. 2 Restoring force and displacement of pier 1

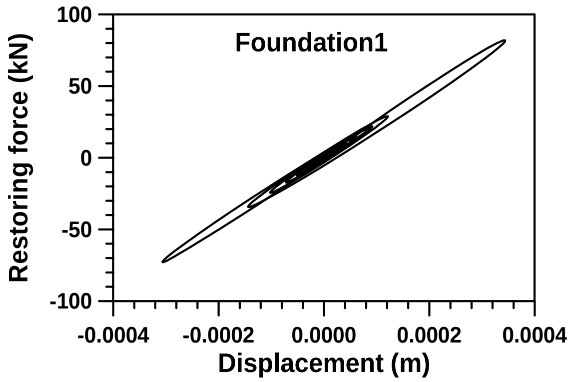

Fig. 4 Restoring force and displacement of foundation 1

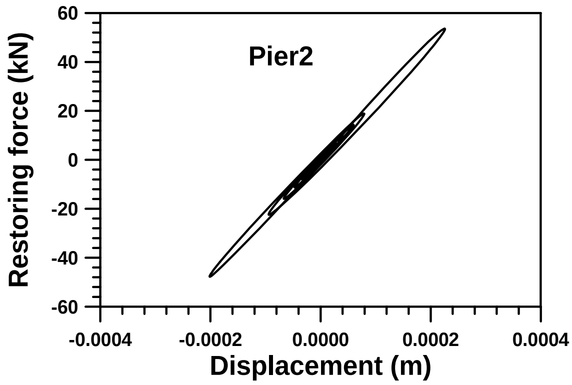

Fig. 3 Restoring force and displacement of pier 2

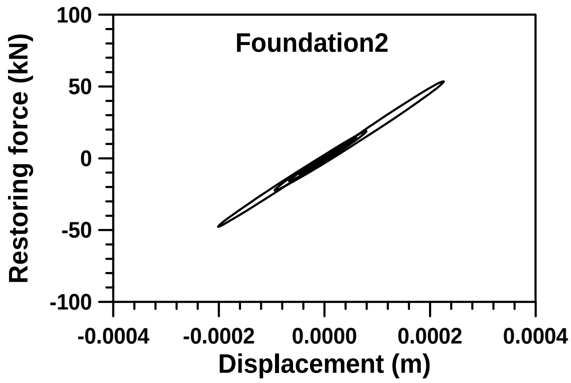

Fig. 5 Restoring force and displacement of foundation 2

The second identification cycle is then proceeded. The value of C_{p2} is fixed at $900.0 kN.s/m$. The minimization proceeded to again an optimal $K_{p2} = 110.50\,MN/m$, Fig. 8. Then, the value of K_{P2}

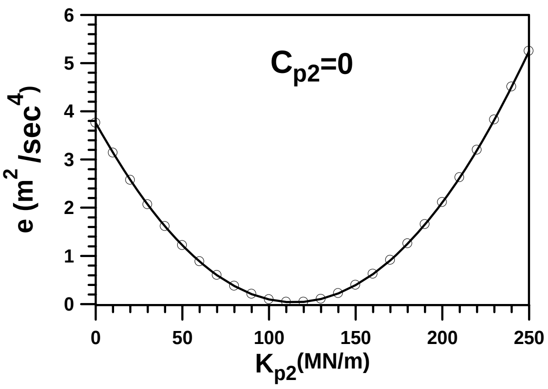

Fig. 6 Partial measure-of-fit for piers in the first cycle setting $C_{p2} = 0 kN.s/m$

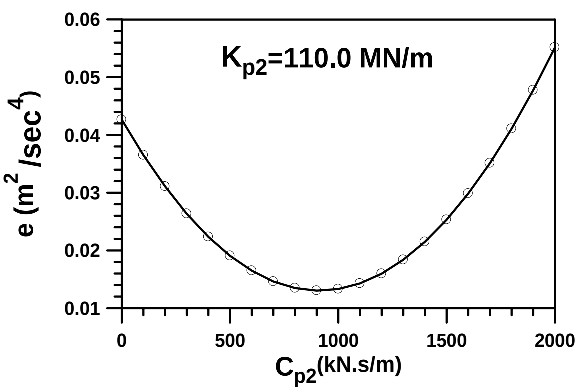

Fig. 7 Partial measure-of-fit for piers in the first cycle setting $K_{p2} = 110.0 MN/m$

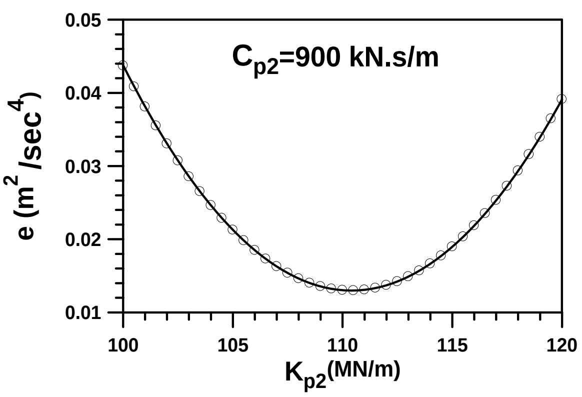

Fig. 8 Partial measure-of-fit for piers in the second cycle setting $C_{p2} = 900\,kN.s/m$

is fixed at $110.50 MN/m$. Minimizing e_p can be performed again, Fig. 9, the optimal estimated of C_{p2} is $890.0 kN.s/m$. Numerical calculation suggests that three cycles of identification are sufficient.

Column 2~3, column 4~5, in Table 1, respectively summarize the iterative identified parameter values of pier 1, pier 2, foundation 1, and foundation 2. From the above iterative identified values, however, the proposed algorithm offers a more reliable estimation.

The comparison between the identified and measured response is made. Figs. 10 and 11 show the acceleration and displacement responses at foundation 1. The identified and measured ones are virtually identical.

Similarly, the comparison of the acceleration and displacement responses of the deck is presented in Figs. 12 and 13, respectively. They also promise an excellent identification.

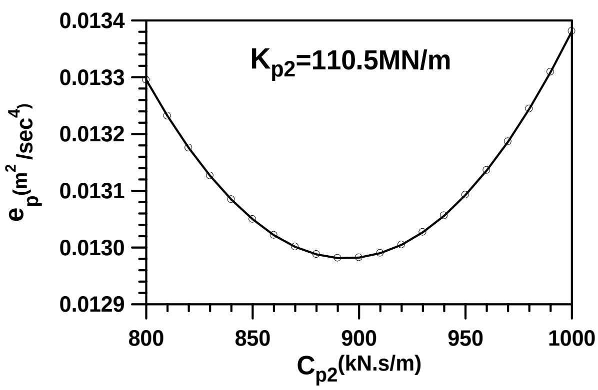

Fig. 9 Partial measure-of-fit for piers in the second cycle setting $K_{p2} = 110.5 MN/m$

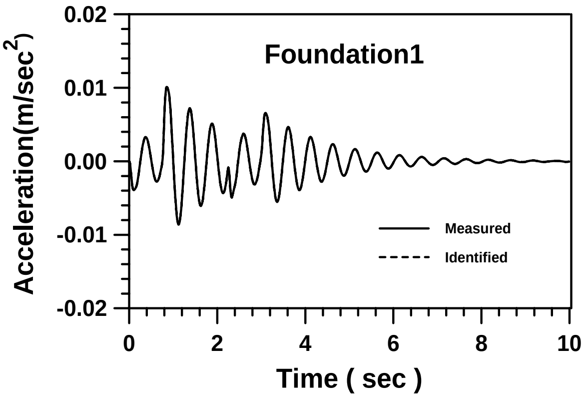

Fig. 10 Comparison between identified and measured accelerations of foundation 1

Table 1 Identified parameters

Number of cycle	C_{p1} kN.s/m	K_{p1} MN/m	C_{p2} kN.s/m	K_{p2} MN/m
1	1707.0	222.18	900.0	110.00
2	1721.0	221.52	890.0	110.50
3	1713.0	221.59	896.0	110.45
True value	1744.0	221.31	872.0	110.66
E. I.	Deck 2.73E-3			
Number of cycle	C_{f1} kN.s/m	K_{f1} MN/m	C_{f2} kN.s/m	K_{f2} MN/m
1	2345.0	237.64	2259.0	235.32
2	2358.0	236.94	2240.0	236.39
3	2350.0	237.01	2252.0	236.28
True value	2381.0	236.97	2204.0	236.97
E. I.	Foundation1 3.20E-3		Foundation2 3.82E-3	

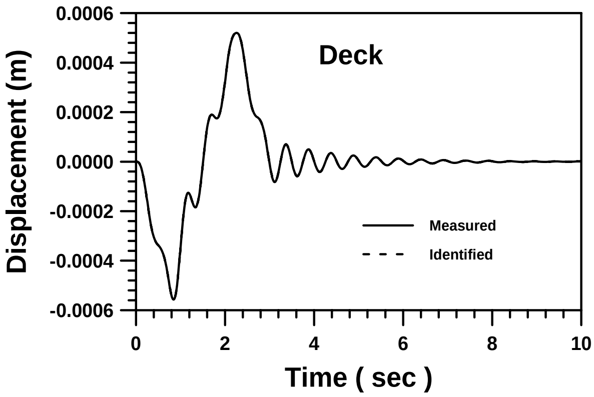

Fig. 13 Comparison between identified and measured displacements of deck

Finally, Figs. 14 and 15 present the actual shear forces at the bottom of the pier 1 and pier 2, respectively, and show that the shear forces are able to be predicted.

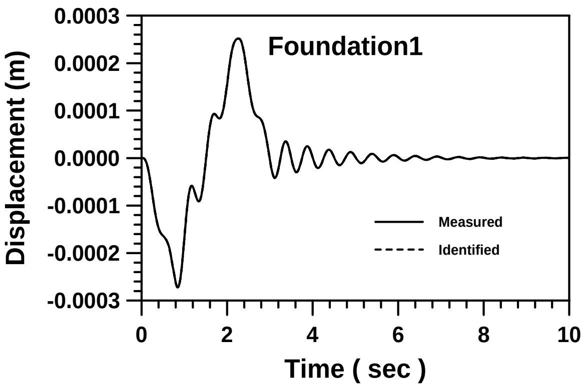

Fig. 11 Comparison between identified and measured displacements of foundation 1

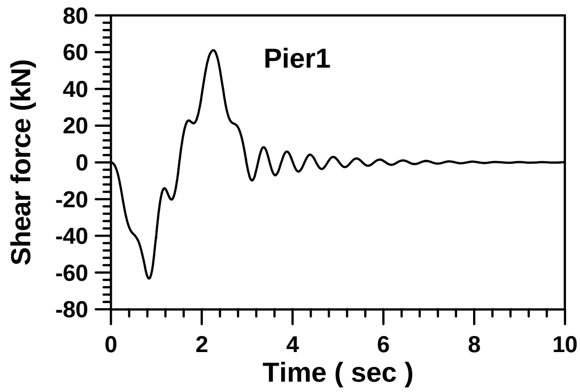

Fig. 14 Actual bottom shear forces of pier 1

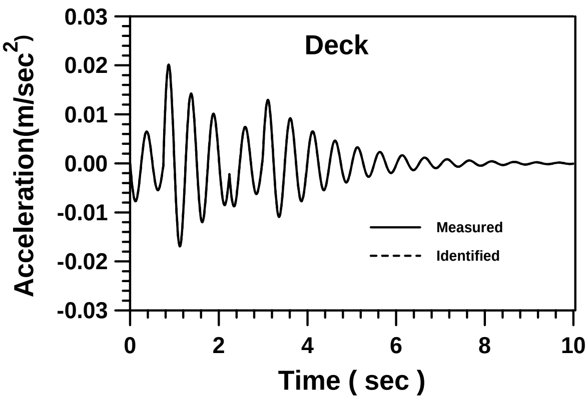

Fig. 12 Comparison between identified and measured accelerations of deck

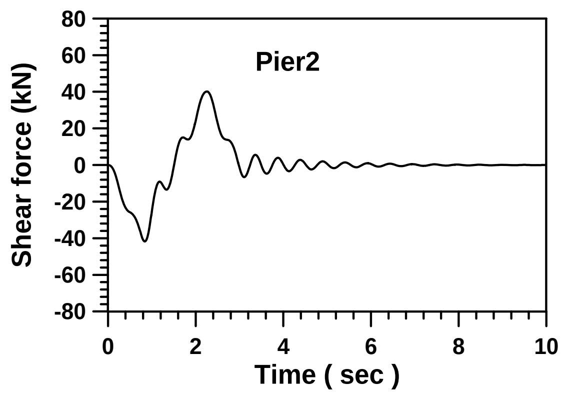

Fig. 15 Actual bottom shear forces of pier 2

VI. CONCLUSIONS

This paper develops an identification procedure to examine the dynamic characteristics of a high-speed railway bridge built on pile foundation. A linear model is used for the pier. To be more realistic, piles are founded on soft sit. The pile-soil interaction is taken into account by using a series of distributed soil-springs and soil-dashpots. The bridge-soil interaction system is assumed under a single moving vehicle braking and acceleration conditions.

The contribution of this paper can be evaluated from several ways. Unlike many other identification methods that generally identify the modal parameters, the proposed method is able to identify the actual system parameters. The parameters in the proposed method are more meaningful and useful. For instance, if degradation of pier stiffness is found, an appropriate retrofit can be executed. In this regard, the actual parameters obtained through the proposed identification process may provide a basis by which various warning system for high-speed railway bridges can be established.

In conclusion, the comparisons between the identified and measured responses at foundations and deck show that the developed technique is feasible for identifying a high-speed railway bridge, as well found on pile foundation. Above all, because of the proposed identification process provides not only the prediction of dynamic responses but also the determination of actual dynamic characteristics for the interaction system. Therefore, the actual bottom shear forces of piers also are able to be predicted, which are more helpful for the safety evaluation of high-speed railway bridges.

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