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Health Monitoring of Structures Under Ambient Vibrations using Semiactive Devices

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***Abstract* — *Structural health monitoring* (SHM) is the process of monitoring structural health and identifying damage existence, severity and location. Clear needs for SHM exist for various types of civil structures; for example, approximately 25% of U.S. bridges are rated as deficient and will require significant expenditures to rebuild or replace them (FHWA, 2002). Yet, the dominant method for monitoring the health of civil structures is manual visual inspection — a time-and labor-intensive procedure. Global vibration-based SHM techniques have been studied, but no approach has been well established and accepted due to limitations of ambient excitation sources for most civil structures.**

One approach that may help alleviate some of the SHM difficulties for civil structures would be to use variable stiffness and damping devices (VSDDs) — controllable passive devices that have received significant study for vibration mitigation — to improve damage estimates. In addition to providing near optimal structural control strategies for vibration mitigation, these low-power and fail-safe devices can also provide parametric changes to increase global vibration measurement sensitivity for SHM.

This paper proposes using VSDDs in structures to improve SHM, and demonstrates the benefits in contrast with conventional passive structures. It is shown that using VSDDs in identification gives parameter estimates that have better means and smaller variations than the conventional structure approach. The improvements in the identification process are even more effective when adding higher effective levels of stiffness or damping to a structural system, even though the resulting VSDD forces due to ambient excitation are small.

I. Introduction and Literature Review

Accurate diagnosis of structural health is a vital step in protecting structures. Whether caused by acute events, such as earthquakes or other natural disasters, or long-term degradation from environmental effects and human use (and abuse), structural damage can threaten both danger to human life and economic loss. The process of monitoring structural health and identifying damage existence, severity and location is generally termed *structural health monitoring* (SHM). Chang [6] defined structural health monitoring to be an "autonomous [system] for the continuous monitoring, inspection, and damage detection of [a structure] with minimum labor involvement."

By determining the model that best fits data taken from the structure, certain structural characteristics may be identified. With identification at different points in time — periodic or shortly after natural disasters — changes in these characteristics may be monitored. With damage models, changes in structural characteristics are used to predict damage severity and location. The focus of model parameter identification to achieve SHM is usually on local loss of stiffness as a proxy for local damage [2,3,5].

For most civil structures, the excitation is limited to ambient sources for SHM. Ambient excitation on civil structures takes a number of forms including wind, traffic, waves and microtremors. The ambient excitation approach has several advantages over those using forced vibration response. For example, for the low amplitude excitations typically experienced during ambient vibration, most structural systems are well characterized with linear models. In addition, continuous ambient vibration tests can be performed at a very low cost. However, while applications of ambient excitation identification techniques are more acceptable than active ones, the signal-to-noise ratios are small enough to make SHM difficult and results uncertain. Thus, solutions to these SHM difficulties must be sought elsewhere. One solution is to induce parametric changes into the structures through semiactive control devices in order to increase the sensitivity to detect model parameters changes.

A. *Passive, Active, and Semiactive Devices*

In general, control devices can be classified into passive, active, semiactive, and hybrid devices [21]. Hybrid devices are combinations of the other three classes. Passive devices can partially absorb structural vibration energy and reduce response of the structure [22]. These passive devices require no energy to function, and are relatively simple and are easily replaced. However, the effectiveness of passive devices is always limited due to narrow effective frequency

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range, their dependence on local information, and inability to be modified if goals change.

Active control devices can reduce structural response more effectively than passive devices because feedback and/or feed-forward control systems are used [15]. However, large power requirements hamper their implementation in practice. Further, active devices can inject dynamic energy into the structural system; if done improperly, this energy has the potential to cause further damage.

In contrast, "smart" devices are controllable passive devices that require small amounts of power to control certain passive behavior. These devices may only store and dissipate energy. Furthermore, they offer highly reliable operation at a modest cost and viewed as fail-safe as they default to passive devices should the control hardware malfunction [9].

This paper proposes using smart, controllable passive devices such as Variable Stiffness and Damping devices (VSDDs) in structures to improve SHM, and demonstrates the benefits over conventional passive structures. VSDDs can adjust the behavior of a structure by real-time modification of stiffness and damping at discrete points within the structure. By commanding different behavior for each VSDD in a structure, multiple structural configurations can be tested, each of which can be designed to increase the sensitivity to damage in different portions of the structure.

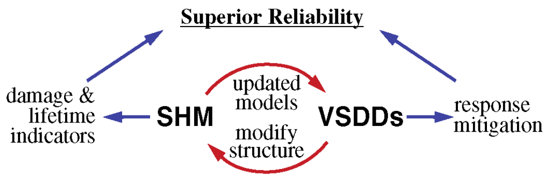

Figure 1. Mutual benefits of SHM and VSDDs

B. *Applications of Semiactive Control to Civil Structures*

VSDDs have been extensively researched for base isolation of structures and other structural control applications. Some researchers have investigated MR dampers for control of seismic response [*e.g.*, 9]. ER dampers were also studied for seismic response control [*e.g.*, 10,13,14] and others. Wind response mitigation using semiactive devices has also received attention, such as stay cable damping [16] and variable stiffness tuned mass dampers [22]. Patten *et al.* [19] reported the first successful full-scale demonstration of semiactive control technology, installing an Intelligent Stiffener for Bridges (ISB) on an in-service bridge on interstate I-35. The Kajima Corporation developed a semiactive hydraulic damper (SHD) and installed it in an actual building [17]. In these applications, the structural deflections were significantly reduced.

II. Parametric Frequency Domain ID with VSDDs

While using VSDDs to improve identification of parameters can be applied to a variety of techniques, the VSDD approach herein is introduced in the context of parametric frequency domain identification.

One method of identifying parameters of a dynamical system is by representing transfer functions (TFs) in the frequency domain as ratios of polynomials. The transfer functions generally are defined by the ratio between the output and input signals. For example, consider a linear structural model of the form:

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{C}_d\dot{\mathbf{x}} + \mathbf{K}\mathbf{x} = \mathbf{b}f\,, \qquad \mathbf{y} = \mathbf{C}_1\mathbf{x} + \mathbf{C}_2\dot{\mathbf{x}} + \mathbf{d}f + \mathbf{v} \qquad (1)$$

where $\mathbf{M}$, $\mathbf{K}$, and $\mathbf{C}_d$ are the mass, stiffness and damping matrices of the system, and $\mathbf{C}_1$, $\mathbf{C}_2$, and $\mathbf{d}$ are the output influence matrices for the displacement, velocity and the external force f. For simplicity of the method developed herein, the input force is assumed to be a single scalar force. Similarly, one can write the model in state-space form

$$\dot{\mathbf{q}} = \tilde{\mathbf{A}}\mathbf{q} + \tilde{\mathbf{B}}f\,, \qquad \mathbf{y} = \mathbf{C}\mathbf{q} + \mathbf{D}f + \mathbf{v} \qquad (2)$$

where $\mathbf{q} = [\mathbf{x}^{\mathrm{T}} \quad \dot{\mathbf{x}}^{\mathrm{T}}]^{\mathrm{T}}$ is the state vector, $\tilde{\mathbf{A}}$ is the system state matrix which is dependent on the mass, damping, and stiffness matrices, $\tilde{\mathbf{B}}$ is the input influence matrix, $\mathbf{C}$ is the output influence matrix for the state vector **q,** and $\mathbf{D}$ is the direct transmission matrix. In both equations, f is an excitation force, and $\mathbf{y}$ is an $m\times 1$ vector of measured responses corrupted by $m\times 1$ sensor noise vector $\mathbf{v}$.

Thus, the system can be represented by the $m\times 1$ transfer function matrix $\mathbf{H}(j\omega)$. Each element of $\mathbf{H}(j\omega)$ can be expressed as the ratio of numerator and denominator polynomials at a certain frequency with coefficients depending on matrices in Eqs. (1) or (2). It is important to state that, for many structural systems, the denominator polynomial is the same for all transfer functions from the same input. Therefore, identifying the denominator polynomial is crucial in defining the system dynamics. The transfer function vector from a single input to the outputs can, consequently, be written in polynomial ratio form as:

$$\mathbf{H}(j\omega) = \mathbf{B}(j\omega)/A(j\omega) \qquad (3)$$

where $\mathbf{B}(j\omega)$ and $A(j\omega)$ are the numerator and denominator polynomials, which may be expanded in the forms

$$\begin{aligned} B_r(j\omega) &= b^r_{n_B-1}(j\omega)^{n_B-1} + b^r_{n_B-2}(j\omega)^{n_B-2} + \ldots + b^r_0 \\ A(j\omega) &= a_{n_A}(j\omega)^{n_A} + a_{n_A-1}(j\omega)^{n_A-1} + \ldots + a_0 \end{aligned} \qquad (4)$$

where the b's and a's are real coefficients.

Assuming that the transfer functions have been determined experimentally through standard procedures from measured input and output data [5], then the experimental transfer function matrix,

$$\hat{\mathbf{H}}(j\omega_i)\,,\ i=1, 2, \ldots, n_\omega \qquad (5)$$

is known at various discrete frequency points. Therefore, the difference between the estimated theoretical transfer function $\mathbf{H}(j\omega)$ and the actual experimental one $\hat{\mathbf{H}}(j\omega)$

represent the residual error equation, which is then used in the identification process of the parameters.

Parametric frequency-domain methods to match such theoretical and measured transfer functions date back to the work of Levy [18] who parameterized a continuous-time TF by the coefficients of numerator and denominator polynomials. One approach to this problem is to follow Levy's procedure [18] in determining the polynomial coefficients and then, as a subsequent step, estimate structural parameters such as mass, stiffness and damping coefficients. In this study, however, the parameterization is chosen to be the structural parameters directly without calculating the coefficients of the polynomials as an intermediate step. In addition, it may be shown that the alternate error measure in [18], while simpler to solve, can be susceptible to strong bias from sensor noise in frequency ranges where $A(j\omega)$ is large (*i.e.*, often the case where $\mathbf{H}(j\omega)$ is small). To avoid this bias, and to avoid some complexity in solving the least-squares problem for the standard error measure **e**, an *iterative* method, described in the following section, is adopted here using an approximation to the denominator $A(j\omega)$.

For a structure with one or more variable stiffness and/or damping devices, the properties of which are determined through a local control system, some of the coefficients in the transfer function polynomials may be adjusted through changing the VSDD control algorithms. Thus, it is convenient to introduce notation to explicitly state that the transfer function polynomials are functions of unknown structural parameters, denoted by the $n \times 1$ vector $\boldsymbol{\theta}$, which is to be estimated, and of known controllable structural parameters, denoted by the vector $\boldsymbol{\kappa}$. The transfer function is, therefore, modified to be:

$$\mathbf{H}(j\omega) = \mathbf{B}(j\omega, \boldsymbol{\theta}, \boldsymbol{\kappa}) / A(j\omega, \boldsymbol{\theta}, \boldsymbol{\kappa}) \tag{6}$$

A. Least Squares Identification

For a given structural model, the A and $\mathbf{B}$ polynomials are specific known functions of their parameters. Substituting the measured TF in place of the exact TF leaves a residual error **e** that may be defined by

$$\mathbf{e}(j\omega_i, \boldsymbol{\theta}, \boldsymbol{\kappa}) = \frac{\mathbf{B}(j\omega_i, \boldsymbol{\theta}, \boldsymbol{\kappa}) - A(j\omega_i, \boldsymbol{\theta}, \boldsymbol{\kappa})\hat{\mathbf{H}}(j\omega_i, \boldsymbol{\kappa})}{A(j\omega_i, \boldsymbol{\theta}, \boldsymbol{\kappa})} \tag{7}$$

A conventional least-squares approach may be adopted to solve this problem, forming a global square error

$$\boldsymbol{\Delta}^2(\boldsymbol{\theta}) = \sum_i \mathbf{e}^*(j\omega_i, \boldsymbol{\theta}, \boldsymbol{\kappa}) \cdot \mathbf{e}(j\omega_i, \boldsymbol{\theta}, \boldsymbol{\kappa}) \tag{8}$$

where $(\cdot)^*$ denotes complex conjugate transpose. The optimal choice of the unknown parameters is found by minimizing the square error — *i.e.*, take the derivatives of the square error Eq. (8) with respect to the elements of unknown vector $\boldsymbol{\theta}$, set them equal to zero, and solve the resulting (generally nonlinear) equations. However, if there are known controllable structural parameters in a structure with multiple configurations — which is the case when using VSDDs, for example — the square error equation can be augmented by using several combinations of known controllable structural parameters

$$\boldsymbol{\Delta}^2(\boldsymbol{\theta}) = \sum_k \sum_i \mathbf{e}^*(j\omega_i, \boldsymbol{\theta}, \boldsymbol{\kappa}_k) \cdot \mathbf{e}(j\omega_i, \boldsymbol{\theta}, \boldsymbol{\kappa}_k) \tag{9}$$

where the symbol $\boldsymbol{\kappa}_k$ denotes multiple distinct sets of parametric changes to the structure. The error is, then, minimized simultaneously for all configurations.

B. Iterative-Least Squares Numerator Method

This method is an approximation to the conventional TF problem. Assume that iteration l in Eq. (9) begins with a starting approximation $\hat{\boldsymbol{\theta}}_{l-1}$ to the unknown parameter vector $\boldsymbol{\theta}$; then, the denominator of Eq. (7) is estimated based on the vector $\hat{\boldsymbol{\theta}}_{l-1}$ of estimated parameters and is no longer a function of these unknowns, but only in the frequency and the multiple distinct sets of parametric changes to the structure

$$\hat{A}_l(j\omega_i, \boldsymbol{\kappa}_k) \equiv A(j\omega_i, \hat{\boldsymbol{\theta}}_{l-1}, \boldsymbol{\kappa}_k) \tag{10}$$

and the error is, thus, formed as

$$\hat{\mathbf{e}}_l(j\omega_i, \boldsymbol{\theta}, \boldsymbol{\kappa}_k) = \frac{\mathbf{B}(j\omega_i, \boldsymbol{\theta}, \boldsymbol{\kappa}_k) - A(j\omega_i, \boldsymbol{\theta}, \boldsymbol{\kappa}_k)\hat{\mathbf{H}}(j\omega_i, \boldsymbol{\kappa}_k)}{\hat{A}_l(j\omega_i, \boldsymbol{\kappa}_k)} \tag{11}$$

and the squared error takes the form:

$$\boldsymbol{\Delta}_l^2(\boldsymbol{\theta}) = \sum_k \sum_i \hat{\mathbf{e}}_l^*(j\omega_i, \boldsymbol{\theta}, \boldsymbol{\kappa}_k) \cdot \hat{\mathbf{e}}_l(j\omega_i, \boldsymbol{\theta}, \boldsymbol{\kappa}_k) \tag{12}$$

Minimizing the sum of the square error in Eq. (12) will result in an updated estimate $\hat{\boldsymbol{\theta}}_l$ to the unknown parameter vector $\boldsymbol{\theta}$. The iterations continue until the relative differences between $\hat{\boldsymbol{\theta}}_{l-1}$ elements and the corresponding elements of $\hat{\boldsymbol{\theta}}_l$ are all below some threshold. (Absolute or relative norms of the difference could also be used.) A maximum number of iterations may also be set to stop the algorithm in the case that the iterative method does not converge (though this termination criterion was not required in this study as convergence always occurred within a limited number of iterations).

III. Illustrative Example

Consider a bridge structure such as one shown in Fig. 2, which is a typical elevated high- way bridge that consists of decks, bearings, and piers. The behavior of the bridge deck and piers, with a bearing between them, while complex, can be well approximated with the simple 2DOF model shown in Fig. 3c. This 2DOF model may be used to

Figure 2. General view of the construction of vehicle lanes [1]

represent a passive system with rubber bearings if the girder is continuous with one pier and one bearing, or for several piers and bearings with identical properties as Shown in Figs. 3a and 3b. Also, this model can be used for VSDD systems if the dev-ices are attached as shown in Fig. 3d and commanded to provide identical force levels.

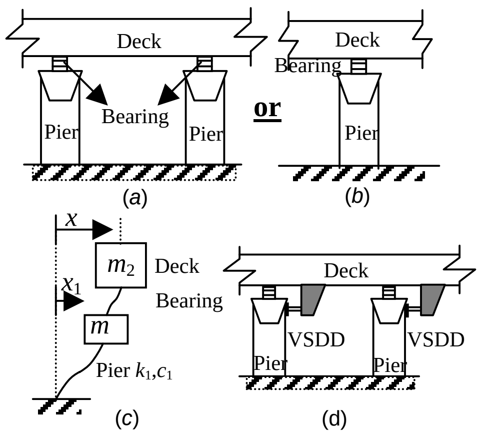

Figure 3. 2DOF bridge model and placement of VSDDs

The device is considered ideal (no internal device dynamics), the transfer functions are measured through standard means, and the iterative least-squares parametric frequency-domain identification technique is applied. The numerical quantities for this model of a full-scale bridge structure, are drawn from [12] where k_1 = 15.791 MN/m, k_2 = 7.685MN/m, m_1 = 100 Mg (tons), m_2 = 500 Mg, c_1 = 125.6 kN·s/m, c_2 = 196 kN·s/m. The experimental transfer functions are simulated in MATLAB® by using the exact transfer functions plus the Fourier transform of a Gaussian pulse process typical of band-limited Gaussian white sensor noise vector processes. The noisy transfer functions are shown in Fig. 4.

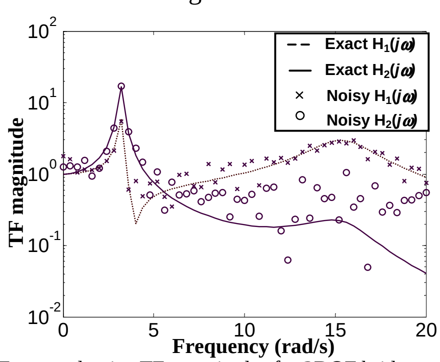

Figure 4. Exact and noisy TF magnitudes for 2DOF bridge model

Data is collected while the VSDDs are commanded to act in one of several discrete stiffness or damping modes, with different noise corrupting each subsequent data set. The conventional structure approach is provided with the same amount of data for fair comparison. The variation in identified structural parameters due to the effects of random noise are studied by performing these identifications 100 times, each with a different random seed to generate the noise. This gives a measure of the mean and the variance of the estimates.

The theoretical polynomial transfer function matrix $\mathbf{H}(j\omega,\kappa)$ of the 2DOF bridge model is defined as

$$\mathbf{H}(j\omega,\kappa)=\left[\frac{B_1(j\omega,\boldsymbol{\theta},\kappa)}{A(j\omega,\boldsymbol{\theta},\kappa)}\quad\frac{B_2(j\omega,\boldsymbol{\theta},\kappa)}{A(j\omega,\boldsymbol{\theta},\kappa)}\right]^{\mathrm{T}} \qquad (13)$$

containing the transfer functions from the ground acceleration to the absolute accelerations of the pier and to the bridge deck. The unknown parameter vector $\boldsymbol{\theta}$ is

$$\boldsymbol{\theta}=\left[\frac{k_1}{m_1}\quad\frac{k_2}{m_2}\quad\frac{c_1}{m_1}\quad\frac{c_2}{m_2}\quad\frac{m_2}{m_1}\right]^{\mathrm{T}} \qquad (14)$$

where k_1 is the stiffness of the pier and k_2 the stiffness of the bearing. It is assumed in this problem that the pier mass m_1 is known. The parameter κ denotes the additional stiffness or damping at discrete levels added by the VSDD connected between the per and the deck.

A. *Variable Stiffness Case Results: Small Forces*

The iterative least-squares parametric frequency domain identification is performed on this 2DOF bridge model, both with a VSDD in the isolation layer between the deck and pier and without. The stiffness levels induced by the device are 0%, 10%, 20%, 30% and 40% stiffness of the isolator; *i.e.*, κ_1 = 0.0, κ_2 = 0.1, …,κ_5 = 0.4 of the isolator stiffness coefficient.

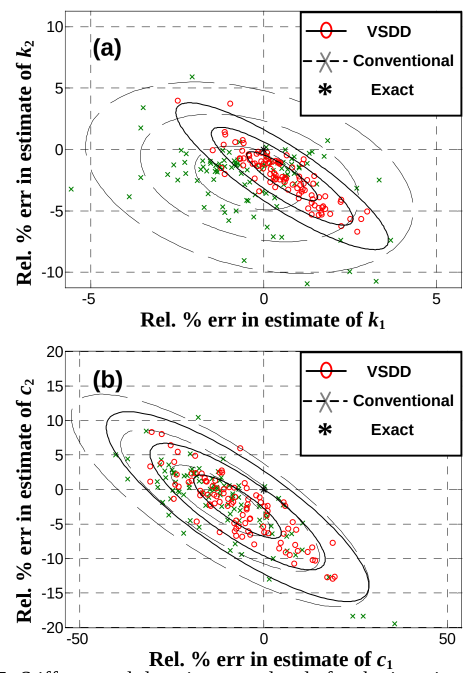

Figure 5. Stiffness and damping error levels for the iterative method with exact start in 2DOF model (variable stiffness mode)

The results show error reductions in both stiffness and damping estimates (though with the latter more modest than the former). The relative error in the stiffness estimates, shown in Fig. 5, have some small bias for both conventional and VSDD approaches — about 0.5% in the estimate of the pier stiffness and about 2% in that of the isolator. While the bias level is similar, the VSDD approach shows notable reductions in stiffness estimate variation, demonstrating that the VSDDs improve the identification. Similar observations may be made regarding damping estimates, as shown in Fig. 5b. The VSDD approach slightly decreases the bias in the pier damping coefficient estimate, and modestly decreases the variation in both pier and isolator damping estimates.

B. Variable Damping Case Results: Small Forces

Using variable damping would be of great interest since "smart" semiactive damping devices have received extensive study for vibration mitigation purposes and capitalizing on the synergies between control and SHM would be a cost-effective solution.

The results, shown in Fig. 6, indicate that the *VSDD* approach, with the damping levels described above, did not differ significantly from the *conventional structure* approach for low damping levels. The relative errors in stiffness estimates in Fig. 6 have similar bias in both approaches and a slightly larger variation with the VSDD damping device. Similar observations may be made about the damping estimates.

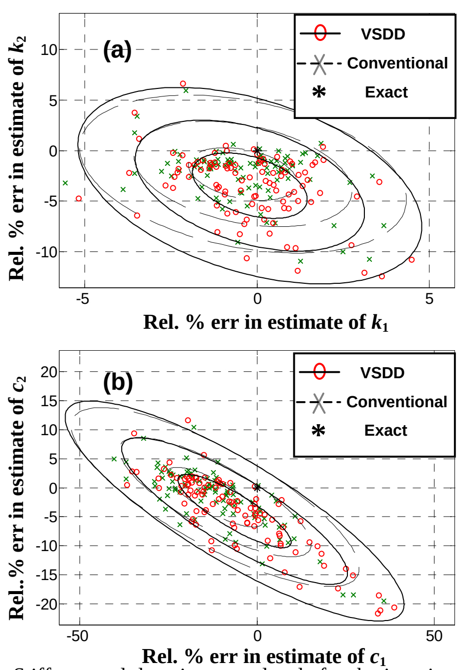

Figure 6. Stiffness and damping error levels for the iterative method with exact start in 2DOF model (variable damping mode)

One reason that the variable damping here did not provide any notable improvement is the very small force levels generated by the damping device. The damping forces in the isolation layer of this bridge model are about one order of magnitude smaller than the stiffness forces.

IV. LARGER VSDD STIFFNESS/DAMPING FORCES

To improve the advantages of the VSDD approach, larger VSDD stiffness/damping levels may be used. To verify this improvement, the identification is performed again with four sets of configurations: (*i*) adding {0,1,2,3,4} times the bearing stiffness, (*ii*) adding {0,5,10,15,20} times the bearing stiffness, (*iii*) adding {0,25,50,75,100} times the bearing damping, and (*iv*) adding {0,100,200,300,400} times the bearing damping. The device is considered ideal (no internal device dynamics), the transfer functions are simulated as discussed previously.

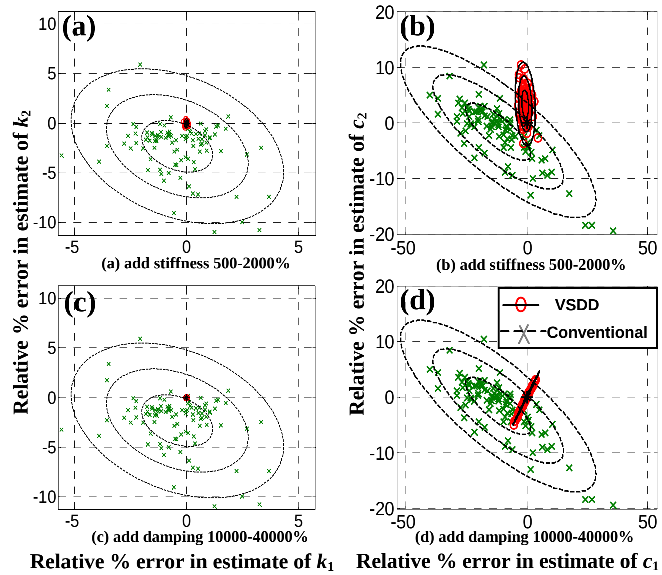

Figure 7. Comparison of stiffness and damping estimate error levels of higher VSDD induced stiffness/damping for 2DOF bridge model

Studying the results from these simulations, it is found that by increasing the level of stiffness that the VSDD induces at the isolator level, the variation of the relative error of stiffness coefficients estimation decreases extensively as shown in Fig. 7a. In addition, the variation of the relative error of the damping coefficients estimation reduces considerably as shown in Fig. 7c compared to cases of lower levels of induced VSDD stiffness, as in Figs. 5 and 6. For the case of varying damping coefficients, it is shown in Fig. 7b that the estimation of the stiffness coefficients are improved dramatically by increasing the damping levels that VSDDs induce to the structure. The damping estimates are also improved considerably. The results definitely confirm the improvement in the ID process using VSDDs.

One initial reaction to this approach is that the stiffness/damping levels sound unreasonable. However, it must be understood that these are effective levels of stiffness and damping forces exerted during low-level ambient excitation. The actual forces are well within the capabilities of current VSDDs. To verify that the force levels are reasonable, the response of the structure to a low-level earthquake excitation (Kanai-Tajimi filtered white noise with a 0.002*g* RMS ground acceleration) is computed. With the VSDD producing 20 times the bearing stiffness, the RMS pier and deck drifts are 1.5 mm and 0.125 mm, respectively, RMS absolute pier and deck accelerations are 0.0037*g* and 0.004*g*, respectively, and RMS VSDD force is 19.2 kN. This force level is quite small relative to the masses (500 ton deck, 100 ton pier). With the VSDD producing 400 times the bearing damping, the RMS pier and deck drifts are 1.44 mm and 0.074 mm, respectively, RMS absolute accelerations are 0.003*g* at both deck and pier, and the RMS VSDD force is about 15 kN, which is also small compared to the masses.

V. Conclusions

This paper demonstrates the effectiveness of using variable stiffness and damping devices to improve estimates of structural parameters for SHM and damage detection. Since VSDDs can be commanded to exert various force time histories, the response of a structure may be altered through the parametric changes affected by the VSDDs. The multiple "snapshots" of structural characteristics provided by the VSDD approach, can provide additional information to make structural parameter ID more accurate.

VSDD/SHM was investigated by identifying structural parameters — mass, stiffness and damping coefficients — based on measured absolute acceleration transfer function data, using a parametric frequency-domain least-squares identification method. The structural parameters were identified, first with VSDDs in the structure, and then with no VSDDs. In all cases, simulated sensor noise is added to the exact transfer function to replicate the noisy transfer functions. The variation in identified structural parameters due to the effects of random noise are studied by performing these identifications several times, each with a different random seed to generate the noise.

The iterative least-squares identification, with and without VSDDs, is applied to a bridge pier/deck model. The results indicate that using the VSDD approach with variable stiffness or variable damping can reduce errors in estimating structural stiffnesses by two orders of magnitude, and provide significant improvements in damping estimates as well. Thus, VSDDs can improve the effectiveness of ambient SHM for civil structures.

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