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SEISMIC AND VIBRATION MITIGATION ON THE HARBOR WHARF STRUCTURAL SYSTEM

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ABSTRACT

In recent developments of structural design, the performance based design method has been popularly applied, which is also known as Performance-Based Engineering, and for earthquake engineering, the Performance-Based Seismic Design or Performance-Based Earthquake Engineering (PBEE) has also been developed. Traditionally, wharf structures in harbors are counted as very basic infrastructure without much performance requirement except for safety. However, according to the seismic performance during recent earthquake events such as the 1990 Kobe earthquake in Japan and the 1999 Chi-Chi earthquake in Taiwan, many wharf structures were severely damaged beyond repair and needed to be reconstructed. The damage of the wharf structure also influenced many facilities built or based on the infrastructure of wharves. Since the infrastructure of wharf systems are basically subjected to dynamic loadings such as the sudden impact from berthing ships, the random wave forces and the loading impact from cargo ships, a multi-purpose design based on seismic resistance for the structure may also have good performance on the vibration mitigation too. In this study, the passively reacting mitigation devices are incorporated into the wharf structural system and responses are analyzed and compared to the traditional type of structural system. The response mitigation devices include both the high damping rubber (HDR) dampers and the viscoelastic (VE) damper system, of which the material model were constructed and simulated along with the dynamic responses of the structural system. It was found from the analytical analysis that after appropriate design, the seismic performance of the wharf structural system can be effectively upgraded by the incorporation of vibration mitigation devices as introduced.

Introduction

In recent developments of structural design, the performance based design method has been popularly applied, which is also known as Performance-Based Engineering, and for the earthquake engineering, the Performance-Based Seismic Design or Performance-Based Earthquake Engineering (PBEE) has also been developed. According to PBEE, the seismic performance of structures during the stages of design, construction, maintenance and monitoring on performance are all taken into account. Traditionally, wharf structures in the harbor are counted as very basic infrastructure without much requirement on the performance except for the safety. However, according to the seismic performance in the late events of earthquakes such as the 1990 Kobe earthquake in Japan and 1999 Chi-Chi earthquake in Taiwan, many wharf structures were severely damaged beyond repair and needed to be reconstructed. Due to the large

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size of wharf structures located under the water, the expenses of reconstructing the wharf structural system are inevitably huge. Damage of the wharf structure is also influenced by the many facilities built or based on the infrastructure of wharves.

According to the standard for the seismic design of wharf structures from International Navigation Association 2001, the damage status is categorized into four levels: Serviceable, the first level of damage, in which the structure is basically in good condition; Damage Controllable, the second level of damage, in which minor damages are repairable and controllable; Near Collapse, the third level of damage, in which the structure is severely damaged and long term interruption of service is inevitable; Collapse, the fourth level of damage, in which the functioning of structure is totally destroyed. However, based on the same level of intensity of earthquake, similar structure can also be designed with various levels of capacity as indicated, depending on the purpose of the design and performance requirements. Since wharf structures are essential for the mass transportation of important goods, particularly during or following disastrous earthquakes, sustainable functioning at least for some will be very important. Therefore, it is the idea and the purpose of this study to apply the concept of seismic and vibration mitigation to the wharf structural system to improve their seismic performance and to maintain their capacity to the second level, damage controllable and serviceable after major earthquakes.

Since the infrastructure of wharf systems are basically subjected to dynamic loadings such as the sudden impact from berthing ships, the random wave forces and the loading impact from cargo ships, a design based on seismic mitigation for the structure may also have good performance on resisting the vibration induced from the ship berthing or wave forces as a bonus of multi-purposes design. As we know, severe deflections and deformations occur subsequently after the vibration and then result in structural damage. In this study, the passively reacting mitigation devices are incorporated into the wharf structural system and responses are analyzed and compared to the traditional type of structural system. Due to the applicability to the infrastructure of wharfs, the response mitigation devices include both the high damping rubber (HDR) dampers and the viscoelastic (VE) dampers, of which the material model were constructed and simulated along with the dynamic responses of the structural system. The bridge type of wharf structure is a common type of infrastructure system being widely used in ports but has less rigidity compared to other types of wharf structural systems. Therefore, in this study, a typical bridge type of structure was redesigned with either HDR or VE damping devices and analyzed correspondingly when subjected to an impact loading and the strong ground motions. The purposes of this study are to develop an appropriate method for the application of the HDR or VE dampers to the bridge type of wharf structural system and further, to find the seismic resistant and vibration mitigation effect by using the developed material model and the application of the particular damper element in the nonlinear analysis.

Material Behavior of the Damping Devices

In order to adequately predict the behavior of a structural material subjected to dynamic loading, an analytical model must be capable of representing the typical material characteristics and adequately describing the dynamic behavior. Therefore, both the HDR and VE materials capable of dissipating input energy are introduced here.

Analytical Model for the High Damping Rubber (HDR) Damper

A hysteretic model, based on the viscoplastic mechanical model and modified with the restoring force model (Wen, 1976; Baber and Wen, 1981), capable of accounting for the degradation of stiffness and strength and the strain hardening effect, representing a relationship between the derivative of the stress over strain and the strain rate, is presented as (Lee 1996)

$$\frac{d\tau}{d\gamma} = \rho_k G_0 [\dot{\gamma} + \rho_p (\chi - y)|\dot{\gamma}| - (\rho_{s1}|\dot{\gamma}|y^n + \rho_{s2}\dot{\gamma}|y|^n)/\rho_f], \quad n = 1, 3, 5, .. \tag{1}$$

where G_0 is the original shear modulus of the nonlinear system, and x and y are the dimensionless strain and stress proportional to the yielding strain and stress respectively. Parameters $\rho_\bullet$ are to account for the nonlinear hysteretic characteristics. In terms of the dissipated energy, $\mathrm{E} = \int_t \tau d\gamma$ and a rate constant Δ, the parameters accounting for the degradation of the stiffness, strength and pinching such as

$$\rho_\bullet = \alpha_\bullet + \beta_\bullet \exp(-\Delta \mathrm{E}) \tag{2}$$

where $\alpha_\bullet$ and $\beta_\bullet$ are parameters corresponding to the original and the ultimate value for the data related, while the parameters corresponding to the rate of the variation of the nonlinear hysteretic characteristics of the loops are

$$\rho_{s1} = \alpha_{s1} + \beta_{s1} \exp(-\Delta \mathrm{E}) \quad \text{and} \quad \rho_{s2} = -(1-\rho_{s1}) \tag{3}$$

These parameters are determined from the experimental testing results. A list for the parameters corresponding to the HDR dampers under study is shown in Table 1. A comparison between the experimental data (Tanzo 1992) and the analytical results are shown in Fig.1, where very good agreements are obtained.

Table 1 Empirical parameters of HDR damper (Lee 1996).

parameters	α	β	Δ
ρ_k	0.50	0.50	0.01
ρ_p	0.20	-0.10	0.01
ρ_f	2/3	1/3	0.01
ρ_{s1}	0.50	0.15	0.01

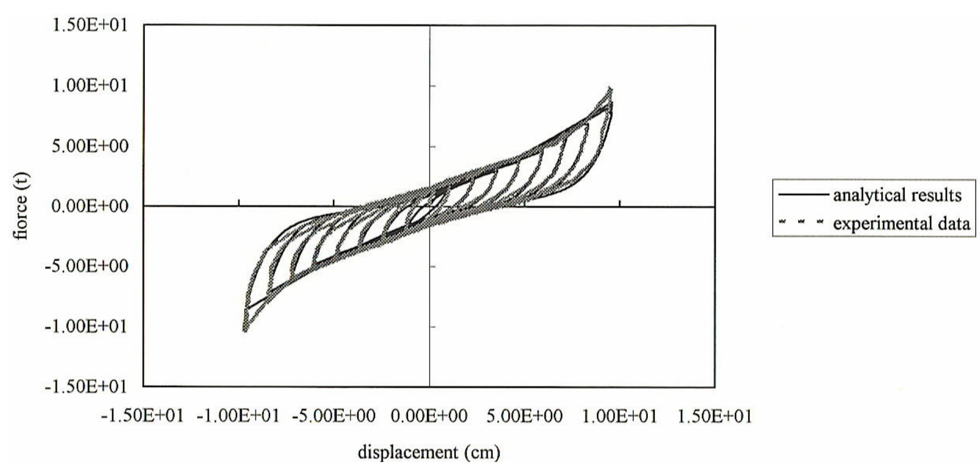

Figure 1. The comparison of hysteretic loops for the HDR dampers (Lee 1996)/

Analytical Model for the Viscoelastic Damper

Based on the molecular theory and the fractional derivative model, a nonlinear analytic model was derived and modified by using the available experimental results (Lee and Tsai 1992, 1994). The constitutional formula at time step $n\Delta t$, for the linear variation of the strain between two time steps, $(n-1)\Delta t$ and $n\Delta t$, is presented as

$$\tau(n\Delta t) = \left[G' + \frac{G''(\Delta t)^{-\alpha}}{(1-\alpha)\Gamma(1-\alpha)} \right] \gamma(n\Delta t) + \tau_p(n\Delta t), \qquad 0 < \alpha < 1, \tag{4}$$

where τ and γ are the stress and strain of the material; G' and G'' represent the shear modulus corresponding to the storage and the loss energy, respectively, and $\Gamma(1-\alpha)$ is the gamma function. The previous time effect of the strain $\tau_p(n\Delta t)$, is given by

$$\tau_p(n\Delta t) = \frac{G''\Delta t^{-\alpha}}{(1-\alpha)\Gamma(1-\alpha)} \left(W_0^{\,n}\gamma(0) + \sum_{i=1}^{n-1} W_i^{\,n}\gamma(i\Delta t) \right) \tag{5}$$

where $W_0^{\,n}$ and $W_i^{\,n}$ are functions corresponding to time step n. One of the comparisons between the analytical model and experimental data for the VE damper is also presented (Lee and Tsai 1994) in Fig.2, where very good agreements between analytical model (B) and experimental data (A) are also attained,

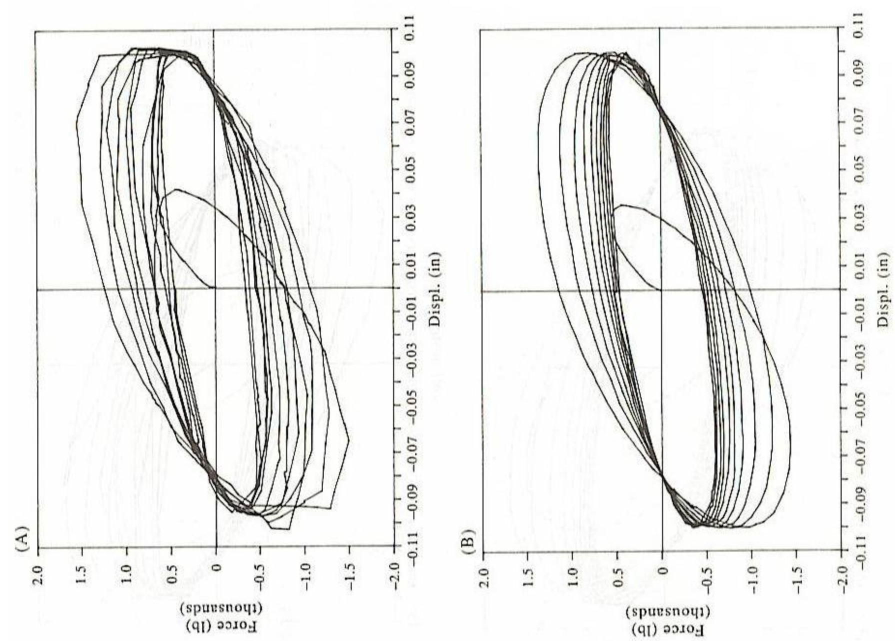

Figure 2. Comparison of experimental data and analytical model for the VE damper (Lee and Tsai 1994).

Wharf Structural System in the Marine Environment

The dynamic equation of motion for the engineering structural member with mass M, structural damping C, and stiffness K, subjected to the wave forces propagated in the normal direction of the structural member can be written as

$$M\ddot{X}(t) + \ddot{X}_g(t) + C\dot{X}(t) + KX(t) = P(t) \tag{6}$$

where $\ddot{X}(t)$, $\dot{X}(t)$ and $X(t)$ are the acceleration, velocity and displacement relative to the ground motion respectively and $\ddot{X}_g(t)$ is the ground acceleration. Taking into account the relative motion between the structures and fluids, the wave forces exerted on the body, $P(t)$ (Newman 1977, Isaacson 1979) is

$$P(t)=\rho C_m V^e \dot{U}_n(t)-\rho C_a V^e\left(\ddot{X}_n(t)+\ddot{X}_{gn}(t)\right)+\frac{1}{2}\rho C_d A^e\left|U_n(t)-\dot{X}_n(t)\right|\left(U_n(t)-\dot{X}_n(t)\right) \tag{7}$$

where $C_a = C_m - 1$, and U_n and $\dot{U}_n$ are the velocity and acceleration of the fluid normal to the structural member resulting from the horizontal and vertical motion of the fluid, respectively. C_m and C_d are coefficients corresponding to inertia and drag effect respectively. V^e and A^e are the displaced volume and the projected front area of the structural member, respectively. The last term in the equation representing the drag force due to the relative velocity of fluid is nonlinear. The nonlinearity of the drag term is retained through the use of the approximate relation derived by Penzien and Tseng (1978),

$$\left|U_t(t)-\dot{X}_n(t)\right|\left(U_t(t)-\dot{X}_n(t)\right)=\left|U_n(t)\right|U_n(t)-2\left\langle\left|U_n(t)\right|\right\rangle\dot{X}_n(t) \tag{8}$$

where $\left\langle\left|U_n\right|\right\rangle=\hat{U}_n$ represents the time average of $\left|U_n\right|$. Through the substitution of Eq. 8, Eq. 7 then becomes

$$P(t)=\rho C_m V^e \dot{U}_n(t)-\rho C_a V^e\left(\ddot{X}_n(t)+\ddot{X}_{gn}(t)\right)+\frac{1}{2}\rho C_d A^e\left|U_n(t)\right|\left(U_n(t)-\dot{X}_n(t)\right)-2\hat{U}_n(t)\dot{X}_n(t) \tag{9}$$

After the substitution of Eq. (9), Eq. (6) takes the form as

$$M^e\ddot{X}(t)+C^e\dot{X}(t)+K^e X(t)=C_m{}^e\dot{U}(t)+C_d{}^e U(t)-M^e\ddot{X}_g(t) \tag{10}$$

where $M^e = M+\rho C_a V^e B_1$; $C^e = C+\rho C_d A^e \hat{U}_n B_1$; $C_m{}^e=\rho C_m V^e B_1$; $C_d{}^e=\frac{1}{2}\rho C_d A^e\left|U_n\right|B_1$.

Now having the equations of motion and the forces exerted on the structural system ready, the analysis can be carried out by using the step-by-step integration schemes for the nonlinear structural system such as the Newmark-β method (Newmark 1962) and Wilson's method (Bathe and Wilson 1976). In this study, the Newmark method using average acceleration operator was adopted due to its stability advantage.

Numerical Results and Discussion

In the numerical analysis, a typical bridge type of 2-bay wharf structure is designed and shown in Fig. 3. The outer diameter and the thickness of the vertical steel pile is 91.4 cm and 1.27 cm respectively. The steel piles are assumed clamped on the sea floor. The dimension of the top deck of the wharf structure is shown in the drawing, while its thickness is assumed to be 1.7 m. The damping devices similar to bearings applied to the bridge are installed in between the deck and the steel piles of the wharf structure.

For the simulation of ship berthing impact, it is assumed that a 10000 D/W cargo ship is berthing at a 10 cm/sec speed. After part of the berthing induced energy is absorbed by the 1.5 m fender (Lee 2000), an impact force of 12.8 T/m is exerted on the deck in a short duration of 0.05 second. Two types of damping devices are applied to the transverse bearing system; the HDR damper and VE damper. The analysis is focused on the displacement induced by the input loading and the effect of displacement reduction when the dampers are applied. The results were obtained by carrying out the calculation for the coupled MDOF nonlinear system, and then plotted and represented in figures. In the earthquake simulation, the strong ground motion similar to the Chi-Chi earthquake 1999, was input, while the wave force and system damping were ignored and the maximum strain of the damper was confined to 100%.

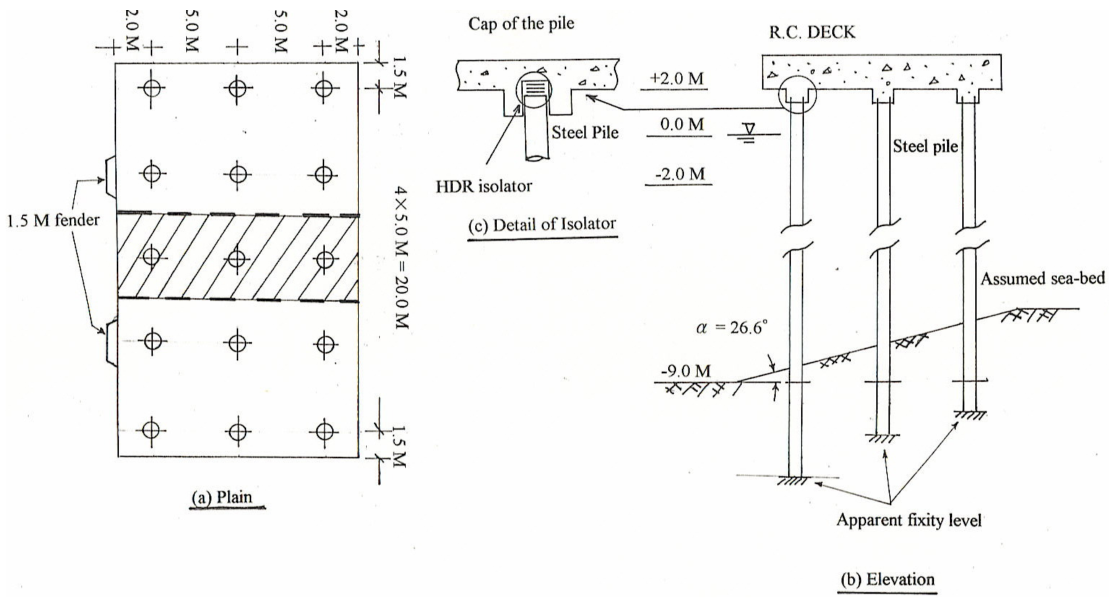

Figure 3. Schematic drawing of bridge type of wharf structure installed with damping devices.

Application of HDR Damper to the Wharf Subjected to Berthing Impact

The placement of the HDR damper is shown in the schematic drawing, where the thickness of the HDR damper is 4.8 cm and the diameter is 9.027 cm. The apparent fixed level for the clamped piles is assumed to be 6 times the diameter of the pile. The time-domain response of the deck subjected to the berthing impact of a cargo ship is shown in Fig. 4, where the dotted line indicates the motion of the deck without installation of the HDR damper and the solid line represents the motion of the deck with installation of the HDR damper. It is found that with the installation of the HDR damper the vibration of the deck is reduced tremendously after about one cycle of vibration.

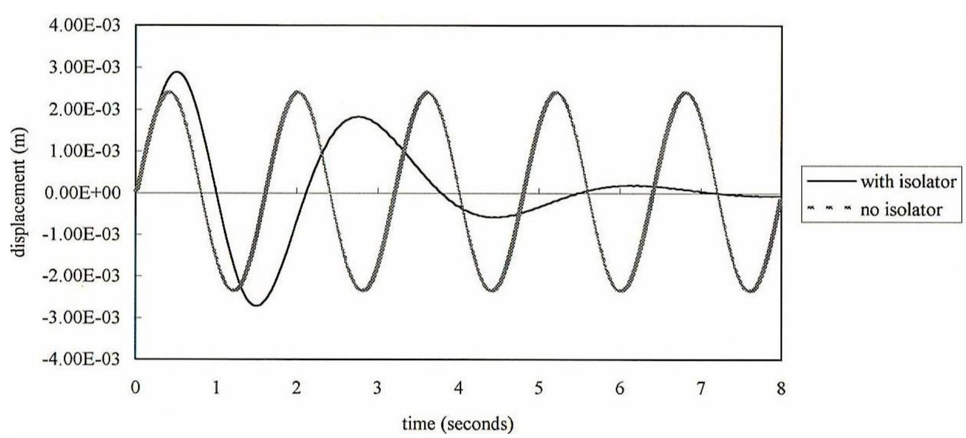

Figure 4. The response comparison of deck with and without HDR damper.

Application of VE Damper to the Wharf Subjected to Berthing Impact

In the second analysis of the VE damper applied to the wharf structure, a structural system with larger mass was applied, of which the natural frequency is reduced to about 0.38 Hz compared to 0.62 Hz in the first analysis. The VE dampers applied in the study were 8 cm thick and 0.16 m^2 in area. The time-domain response of the deck subjected to the berthing impact of a cargo ship is shown in Fig. 5, where similarly, the dotted and solid line indicates the motion of the deck without and with installation of the VE damper, respectively. It is found that with the installation of the VE damper, the vibration of the deck is reduced even more effectively compared to the previous case. After about one half cycle of vibration, the amplitude of displacement response is reduced to 30% and then decays quickly to a horizontal line.

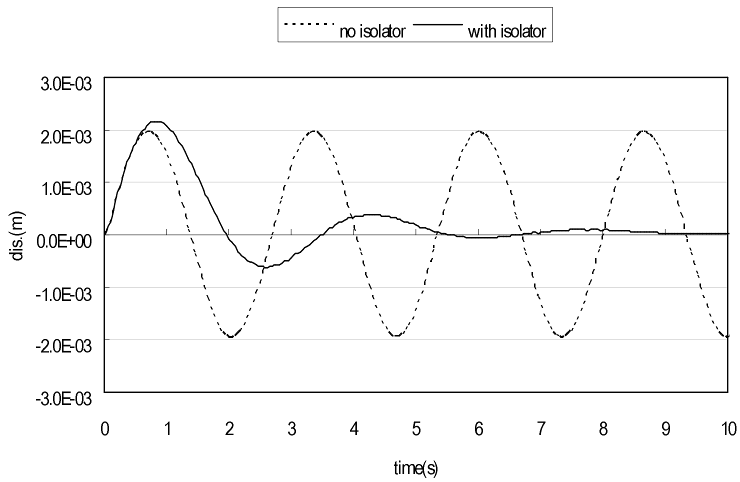

Figure 5. Response comparison for deck with and without VE damper.

Application of VE Damper to the Wharf Subjected to Earthquakes

For the earthquake simulation, a transverse strong ground motion similar to the Chi-Chi earthquake (Station TCU070, E-W) was applied as shown in Fig. 6.

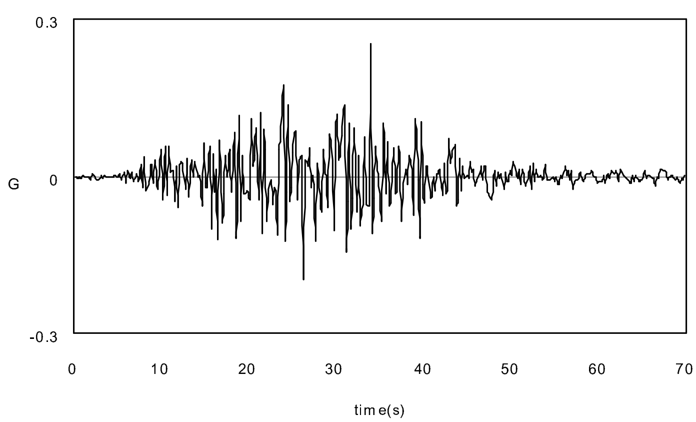

Figure 6. The ground motion of the Chi-Chi Earthquake, 1999 Taiwan.

The displacement response of the top deck in time domain is shown in Fig. 7. This figure shows that the displacement responses are effectively reduced from the installation of VE damping devices. For the wharf deck without damper, the large amplitudes of displacement response continue all the time until the last minute of the earthquake as shown in dotted line of Fig. 7, while the large amplitudes only last for about 30 seconds and then decay into small fluctuations for the wharf with VE dampers.

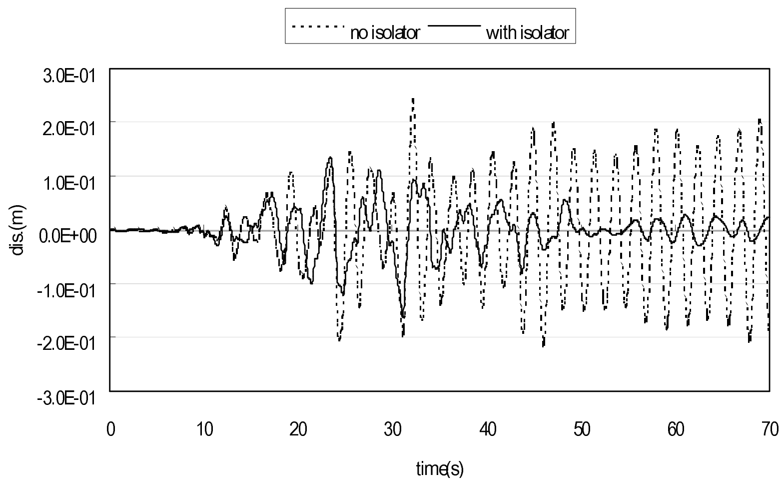

Figure 7. Seismic response comparison for deck with and without VE damper.

The comparison of seismic responses in the frequency domain is also presented in Fig. 8. This figure indicates that for a wharf deck without the installation of VE damping devices, a significantly large peak around 0.5 Hz frequency in the response is identified but the significant responses are not observed when the VE dampers are installed in the structural system.

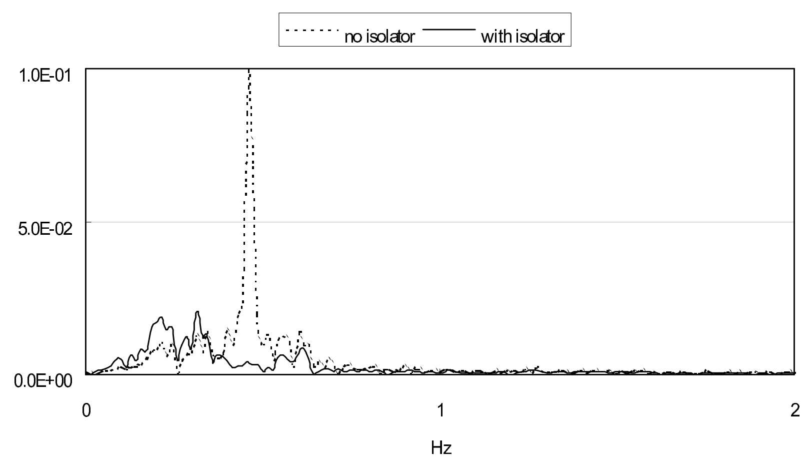

Figure 7. Frequency seismic response comparison for deck with and without VE damper.

Conclusions

In this study, two types of dampers both with energy dissipation capacity are incorporated into a bridge type of wharf structural system with the purposes to mitigate the responses induced from either strong ground motion associated with earthquakes or the sudden impact forces from the berthing motion of ships. For the analysis of the impact induced from berthing motion, both dampers have fairly good performance. Generally, after less than one cycle of vibration, the amplitude of displacement drops dramatically and decays into almost a flat line, which is not possibly achieved for the traditional fender system. For the seismic response, the wharf structures installed with dampers also show a very good result in dynamic responses. In the time domain responses, the mitigation on the displacement responses of wharf structure is clearly realized almost immediately when the strong ground excitation is applied and the response can be reduced by up to 80% during the later response stages. In the frequency domain analysis, a significant mitigation on the responses in the dominant frequency range is also observed and generally the displacement responses may shift to a lower frequency range when the VE dampers are installed in the system.

Therefore, it is concluded that after appropriate design, the installation of dampers with materials such as HDR and VE in between the piles and deck of a bridge type of wharf structural system may effectively improve its dynamic characteristics in terms of the displacement reduction, energy absorption and response within dominant frequencies.

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