Nonlinear Viscous Damper Application to Arch Bridge

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Abstract- **This paper presents nonlinear viscous damper (NVD) implementation to an existing highway arch bridge structure to prevent pounding effect on abutments. The nonlinear dynamic response analysis performed on finite element model of bridge structure due to severe earthquake excitations showed that the relative displacement response between the bridge deck and the abutments exceeds available distance in normal conditions through the longitudinal direction. As a remedy for suppressing seismic responses, nonlinear viscous dampers were approved to be installed at the end of the deck since they are simple to implement for existing structures in construction field. The transversal behavior has also been investigated to mitigate the corresponding responses by the installation of dampers in order to control the lateral behaviour of the bridge for each direction. The damper capacity in response to seismic responses through transverse direction was also obtained in the same manner as done in longitudinal direction.**

Keywords- Highway Arch Bridge; Pounding; Nonlinear Viscous Damper; Energy-equivalent Method; Single Degree-of-freedom System; Finite Element Model

I. INTRODUCTION

The 1995 Hyogo-ken Nanbu earthquake in Japan is the turning point which focuses attention on seismic performance of highway bridges all around the world. Considerable amount of researches states the destructive effects of this earthquake to highway bridges [1]-[4]. The innovative design and retrofitting strategies are intensely developed to improve seismic performance of the new and existing bridge structures since then.

One of the energy dissipation devices that has widespread application area in structural engineering field is the viscous damper. The efficiency for all modes of the structures and simple implementation in the field during construction make the viscous dampers superior and preferable among others. In principle they dissipate energy by forcing fluid through orifices. The force generated by fluid resists the excitation force given by,

$$F_d = c_d |\dot{x}|^{\alpha} sign(\dot{x}) \qquad (1)$$

in which F_d is damper force, c_d is damping coefficient, $\dot{x}$ is relative velocity between damper ends, sign is the signum function (1 for positive, -1 for negative and 0 for zero velocity values). The nonlinearity exponent α takes 1 for linear viscous damper and $\alpha < 1$ for nonlinear viscous damper. For $\alpha = 0$ the viscous damper behaves as a friction damper [5]. The advantage of nonlinear viscous dampers can be explained as having lower damper force even for excessive amounts of velocity by limiting the damper force in comparison with linear viscous dampers. Fig.1 illustrates the damper force-velocity relationship with regard to change in nonlinearity exponent of α. It is clear that the nonlinear dampers yield lower damper force even for large velocities as long as nonlinearity exponent have smaller values whereas the dissipated energy is identical in each scheme for a given structural system.

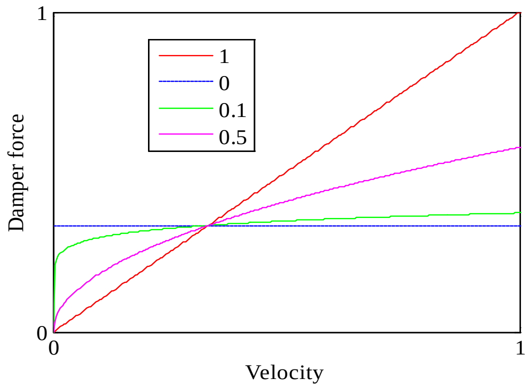

Fig. 1 Damper force-velocity relationship

The effective usage of NVDs is abundant in literature presenting numerous methods to show the non-linear damper efficacy and to obtain the damping coefficient e.g. energy equivalent method [5]-[7], power consumption method [8] etc. In [5], it is stated that providing supplemental damping ratio by viscous dampers should be the main purpose to reduce structural responses to desired values rather than considering the influence of velocity exponent. Reference [6] includes formulas for bridge structures with linear and nonlinear viscous dampers idealized as two degree-of-freedom system based on modal characteristics and damping ratios of structural components. In [8], the non-dimensional damper capacity is referred as normalizing the damper force by the weight of structure. Reference [10] offers formulations for friction damper by the concept of equivalent linear damper.

The retrofitting strategy of an existing highway arch bridge with nonlinear viscous dampers was the aim of this investigation to resist strong earthquake excitations. The description of the normalized damping coefficient which defines a proportion between idealized SDOF system and the bridge structure was proposed in order to adjust the output of SDOF system to the real structure. According to the time response analysis of finite element model of the bridge, the relative displacement responses (target displacement hereafter) was reduced to desired values by preventing pounding effect of the deck.

II. ARCH BRIDGE MODEL

A. Finite Element Model

The arch bridge investigated in this paper is a conventional upper-deck type steel arch bridge with reinforced concrete (RC) deck slab. The finite element model of the bridge is illustrated in Fig.2. The total length of the deck and available width between bearings are 90.0 m and 8.1 m, respectively. The twin arch ribs that are connected by lateral steel bracings have a span of 60.0 m and a rise at the crown of 12.0 m which gives a rise-span ratio of 1:5. RC deck slab is supported by two main longitudinal girders with transverse girders and diagonal members. The connection between main longitudinal girders and arch ribs is supported by 11 piers at the intersection joints of the main ribs and transverse bracings with pin connection except side piers which are fixed to the deck.

The steel members were all modelled as linear beam elements whereas the interaction between the deck and the abutments were assumed as bilinear spring elements of which stress-strain diagram is shown in Fig. 3. K_2 is the degraded stiffness after the member reach yield stress. The ground has been modelled as two nodal spring elements with 6 degree-of-freedom at each node of (F). Table I shows boundary conditions for abutments (A), foundations (F) and side piers (P). (F: Free, R: Restraint)

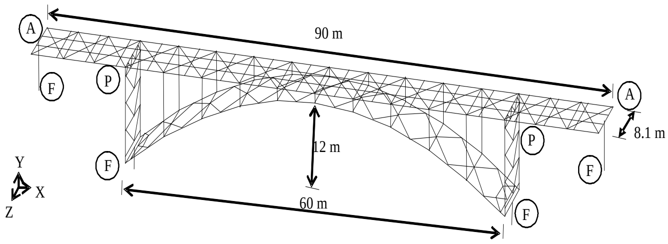

Fig. 2 Configuration of three-dimensional finite element model

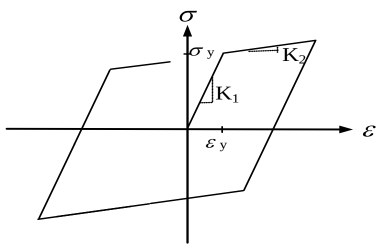

Fig. 3 Bilinear spring model

TABLE I BOUNDARY CONDITIONS

	X	Y	Z	θ_x	θ_y	θ_z
Abutments (A)	F	R	R	R	R	R
Foundation (F)	R	R	R	R	R	R
Piers (P)	R	R	R	R	F	F

In general the gap between adjacent structural components of the bridge structures is small to provide smooth traffic flow [11]. However, at the moment pounding can easily occur during strong earthquake motions. The lateral seismic movement of the deck along longitudinal axis may exceed available distance between the deck and the abutments during strong earthquake motions. To protect the structure from pounding, the NVDs were installed to the each end of deck. Therefore the seismic responses herein were presented as the relative displacement responses of the deck and abutments subjected to the longitudinal excitations whereas the behaviour of midspan was paid attention for transversal excitations.

B. Single Degree-of-Freedom Idealization

The SDOF idealization of large structures such as highway bridges, towers etc. is an effective approach for the sake of less computation labour. In particular regular shaped structures are well-represented with SDOF system by summarizing fundamental modal characteristics of the structure into SDOF system [12].

The SDOF system with supplemental damper in Fig.4 can be utilized for determination of damper characteristics for a given displacement limitation as carried out in this paper. The quantified damping coefficient from the SDOF system includes the sum of all damping coefficient of dampers installed.

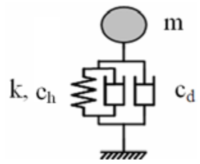

Fig. 4 Idealized SDOF model

III. NONLINEAR VISCOUS DAMPER

C. Energy Equivalent Method for Damper Coefficient

The closed-form solution for supplemental dampers can be obtained approximately to represent additional damping ratio in terms of structural damping ratio by equating dissipated energy to inherently damped energy over one cycle [10]. The solution of friction damper characteristics by energy equivalent method is well presented in [10].

To avoid the iterative and tedious calculations which require computer programming to find out the NVD damping coefficient for the structure which has response limitations, i.e. target displacements, energy equating is an effective solution. The equation of motion of SDOF system with viscous damper under harmonic force is given by,

$$m\,\ddot{x} + c_h\,\dot{x} + k\,x + c_\alpha\,|\dot{x}|\,sign(\dot{x}) = P_0 \sin \omega t \qquad (2)$$

where P_0 is force amplitude and ω is angular frequency of loading. Eq. (2) can be rewritten considering equivalent damping coefficient,

$$m\,\ddot{x} + (c_h + c_{eq})\dot{x} + k\,x = P_0 \sin \omega t \qquad (3)$$

The energy equation of the system is presented in Eq. 4. Since there is no change in kinetic energy, E_k and strain energy, E_s for one cycle [9], they are neglected. E_h is inherent viscous damping energy, E_{eq} is equivalent dissipated energy by supplemental damper, and E_E is excitation energy (harmonic force energy herein).

$$E_k + E_h + E_{eq} + E_s = E_E \tag{4}$$

$$E_h = \pi c_h \omega u_0^2 \tag{5}$$

$$E_{eq} = \pi c_{eq} \omega u_0^2 \tag{6}$$

$$\pi \left(c_h + c_{eq}\right) \omega u_0^2 = \pi P_0 u_0 \sin\phi \tag{7}$$

After substituting the damping ratios and frequency ratio $\left(\omega_r = \omega / \omega_n\right)$ into the Eq. 7,

$$2\pi k \left(\xi_h + \xi_{eq}\right) \omega_r u_0^2 = \pi P_0 u_0 \sin\phi \tag{8}$$

TABLE II SUPPLEMENTAL DAMPING RATIOS

Target Displacement	**10cm**			**8cm**			**6cm**			**4cm**		
Direction (Fundamental Mode)	**Lg (1st)**	**Lg (4th)**	**Tr (2nd)**	**Lg (1st)**	**Lg (1st)**	**Tr (2nd)**	**Lg (1st)**	**Lg (1st)**	**Tr (2nd)**	**Lg (1st)**	**Lg (1st)**	**Tr (2nd)**
Kobe_NS	27	1	4	42	1	20	64	6	42	94	16	78
Kobe_EW	25	---	---	37	0.4	3	56	5	29	93	15	68
Takatori_NS	0.4	---	---	8	---	1	30	3	8	97	11	30
Duzce_NS	---	---	2	4	---	6	14	---	12	35	14	26
ElCentro_NS	---	---	---	---	---	---	9	---	3	26	---	17

Note: --- means that the bridge does not need supplemental NVDs since the pounding effect does not occur.

Dissipated energy by viscous damper over one cycle is obtained by [7],

$$E_D = \oint f_D du = \int_0^{2\pi/\omega} f_D \dot{u}\, dt = \int_0^{2\pi/\omega} c_\alpha \left|\dot{u}\right|^\alpha dt \tag{9}$$

$$E_D = \pi \beta_\alpha c_\alpha \omega^\alpha u_0^{\alpha+1} \tag{10}$$

where the constant is $\beta_\alpha = \dfrac{2^{2+\alpha}\Gamma^2\left(1+\alpha/2\right)}{\pi\,\Gamma\left(2+\alpha\right)}$.

From $E_D = E_{eq}$,

$$\pi \beta_\alpha c_\alpha \omega^\alpha u_0^{\alpha+1} = \pi c_{eq} \omega u_0^2 \tag{11}$$

For non-harmonic excitations i.e. earthquake induced ground motions, equivalent damping ratio and damping coefficient are calculated by,

$$\xi_{eq} = \frac{\beta_\alpha c_\alpha \omega_n^\alpha u_0^{\alpha-1}}{2k} \tag{12}$$

$$c_\alpha = \frac{2k\,\xi_{eq}}{\beta_\alpha \omega_n^\alpha u_0^{\alpha-1}} \tag{13}$$

IV. RESULTS

A. Modal Characteristics

Fundamental modes through longitudinal and transversal directions are required to form SDOF system. Since the boundary conditions are restrained along transverse direction, the stiffness between the deck and abutments were vanished in order to be able to perform free vibration analysis of finite element model of the bridge along corresponding direction. Table II represents fundamental modal characteristics.

TABLE III FUNDAMENTAL MODAL CHARACTERISTICS

	Mode	**Frequency (Hz)**	**Period (s)**	**Effective Mass Ratio (%)**
Longitudinal Axis	1	1.52	0.657	15.0
	4	2.65	0.378	24.0
Transverse Axis	2	1.29	0.777	37.0

B. Establishment of SDOF Model

The three SDOF models have been formed for each fundamental mode. Each SDOF system constituted has fundamental periods of the bridge structure presented in Table II while the stiffness was calculated based on the mass and the period $\left(4\pi^2 m / T^2\right)$ of corresponding mode. The weight of the lumped mass was assigned as 1000kN to provide unity. For longitudinal direction the mean value of damping coefficients was utilized.

C. Implementation of NVDs

To perform the Eq. (13), required supplemental damping ratios to achieve target displacements which the SDOF system should have were obtained under each ground motion at first. Table III depicts the required supplemental damping ratios while structural damping ratio is ξ_h =0.05. (Lg is longitudinal, Tr is transversal direction).

The target displacements are 10 cm and lower values since the available distance between the deck and abutments is 10 cm in the investigated bridge.

The damping coefficients of SDOF systems were evaluated by the Eq. (14) so as to be adjusted for the bridge structure. The equation that is proposed herein includes effective mass of fundamental modes instead of the mass of entire bridge.

$$c_{\alpha} = R \times M_{eff} \div N \qquad (14)$$

whereas c_{α} is damper coefficient, R is normalized damping coefficient (damping coefficient of SDOF/mass of SDOF), M_{eff} is effective mass of fundamental mode and N is the amount of dampers (two dampers were implemented for each end of the deck, i.e. four dampers in total). The illustrations of normalized damping coefficient versus target displacement are given in Fig. 5. Bold values on the graphs show the sufficient damping coefficients which are adjusted for the bridge installation according to Eq. (14).

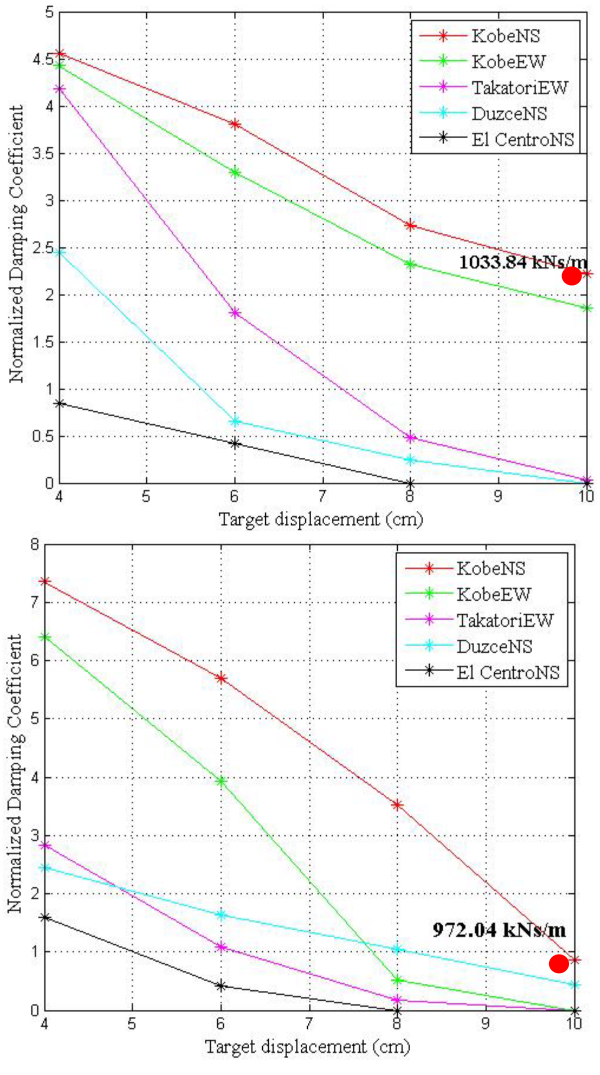

Fig. 5 Normalized damping coefficient vs. target displacement for longitudinal (top) and transverse (bottom) axis

The performance indices in this study were the displacement responses of the deck and midspan of the bridge structure. Fig. 6 and Fig. 7 illustrate the displacement time responses of the end of the deck and midspan with and without NVDs. The reduction in the displacement responses demonstrated the efficacy of NVDs. Furthermore the good agreement between damped systems of SDOF and the bridge is clear for both longitudinal and transverse direction verifying the adequacy of SDOF approach. However one can observe that the transversal response exceeds the target displacement range (10 cm). This arises from the distance between the location of NVDs at the end of the deck and midspan. Nevertheless the reduction in the responses which is almost half of those of undamped bridge is satisfying from the engineering viewpoint.

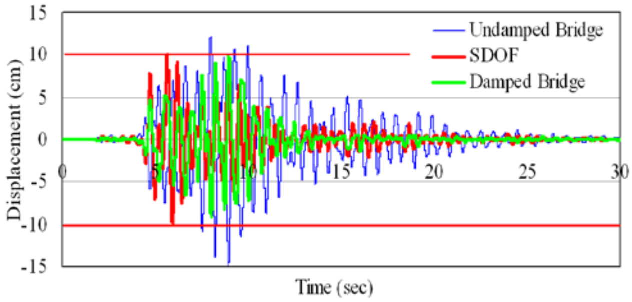

Fig. 6 Displacement time response for the deck under longitudinal Kobe-NS earthquake

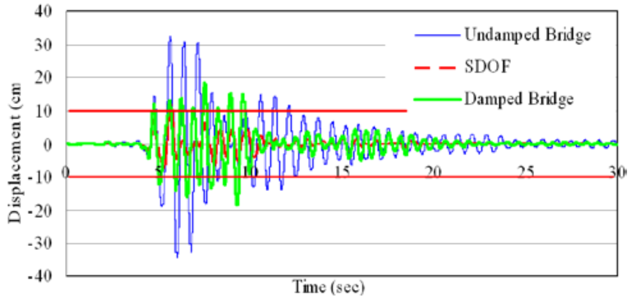

Fig. 7 Displacement time response for the midspan under transversal Kobe-NS earthquake

V. CONCLUSIONS

The SDOF approach to find out supplemental damper characteristics is very effective and the direct approach for preliminary analysis of seismic performance improvement of large structures. In addition to this, fast and efficient retrofitting strategy for large structures is very limited in literature with lack of detailed explanation. Therefore the nonlinear viscous damper implementation to an existing arch bridge structure has been investigated in this paper.

The damping coefficient of nonlinear viscous dampers was proposed to be computed from the effective mass of fundamental mode instead of whole mass of the bridge. The reduced displacement responses for given target displacement confirmed the adequacy of usage of effective mass. The damping coefficient of each NVD installed between the end of the deck and abutment is calculated by the square root of the sum of the squares (RSS).

The conclusions can be summarized as follows:

a) The superiority of nonlinear viscous dampers on constraining the damper force on structure even during large velocities provides better protection than linear damper while they both yield same dissipated energy. Based on this property, energy equivalent method which has widespread application area offers an accurate solution to figure out nonlinear viscous damper characteristics.

b) Single degree-of-freedom models established for each fundamental mode was utilized to figure out the damping coefficients which are essential to perform the equation of motion of finite element model.

c) The adjustment of damping coefficients obtained from the SDOF systems were carried out by the proportion of mass of SDOF system and fundamental effective masses of the bridge.

d) The SDOF idealization and adapting way of damping coefficient succeeded in terms of reduced displacement responses due to severe ground motions.

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