Evaluation of Seismic Performance and Damage Control of Highway Bridge

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This paper presents a retrofitting procedure of a five-span girder bridge with nonlinear viscous dampers installed to the abutments. The study focuses on providing required damping ratio by the nonlinear viscous dampers for the desired structural responses of existing bridge. The nonlinear viscous damper parameters are figured out iteratively by the simplified single degree-of-freedom (SDOF) system possessing the fundamental characteristics of the bridge including natural period and stiffness. This iterative solution is also compared with the energy equivalent method. In addition the usage of effective weight of the bridge structure has been indicated as sufficient for obtaining damper coefficient. The dynamic analysis results have confirmed that iterative method using SDOF system is effective in terms of the reduction of structural responses under severe earthquake waves.

Key Words: nonlinear viscous damper, damage control, dynamic analysis, highway bridge

1. INTRODUCTION

The turning point for the design and retrofitting developments of bridges in Japan is accepted as Hyogo-Ken Nanbu earthquake (1995) as stated in many studies¹³. Through the tough experience of this severe earthquake, conventional seismic energy dissipation methods counting on structural strength have been put aside and energy dissipating devices have been started to develop. These devices limits the structural deformation to the design values so that the stability during seismic excitation can be provided. One of them is viscous damper which dissipates energy by forcing fluid to flow orifices⁴. The damper force produced by viscous dampers is given by 

\[ F_D = c_d |\dot{x}|^\alpha \text{sgn}(\dot{x}) \]

where \( c_d \) damping coefficient, \( \dot{x} \) velocity between damper ends, \text{sgn} is the signum function. The viscous dampers have two kinds depending on the nonlinearity exponent \( \alpha \): \( \alpha=1 \) for linear viscous damper and \( \alpha<1 \) for nonlinear viscous damper. For \( \alpha=0 \) the viscous damper has no difference from friction damper⁵. The superiority of nonlinear viscous dampers over linear viscous
dampers has been studied and published by many researchers\(^5\)\(^-\)\(^6\). They present numerous methods to obtain the higher capacity but lower damper force by equating nonlinear viscous damper to linear viscous damper: energy equivalent method, power consumption method, response spectrum method\(^6\)\(^-\)\(^8\). Pekcan et.al (1999)\(^7\) defines the normalized damping force in their paper. Hwang and Tseng (2005)\(^5\) generates formulas for bridge structures with linear and nonlinear viscous dampers based on modal characteristics and damping ratios of structural components idealized as two degree-of-freedom system.

In this paper the girder bridge is assumed as two-dimensional since it has the fundamental behavior along longitudinal direction. In addition the boundary conditions at the abutments allow the deck to displace along corresponding direction. The nonlinear viscous dampers are implemented at the abutments. The nonlinear viscous damper capacity is obtained by means of idealized SDOF system iteratively where lumped mass will be substituted with effective mass of the bridge structure for calculation of damping coefficient. Iterative solution method for nonlinear dampers is also compared with energy equivalent method by equating energy dissipated in each cycle to the linear viscous dampers for previously defined damping ratios. Then the structures with and without nonlinear viscous dampers are compared to clarify the damper efficiency. The ground motions utilized are Level 2 type ground motions presented in Japan Highway Bridge Specifications\(^9\).

2. BRIDGE MODEL AND RETROFIT CASES

(1) The Bridge

The bridge studied in this paper is a five-span girder bridge with the length of 199 m as seen in Fig.1. The ground is Type II specified according to the Japan Highway Bridge Specifications\(^9\). Piers are prestressed concrete (PC). Additional internal forces and deflections of superstructure (creep, shrinkage, prestress) are neglected.

The finite element model of the bridge is seen in Fig.2. The longitudinal behavior has importance since the deck is movable along this direction. The piers have fixed bearings with the deck slab. The horizontal and vertical elements are rigidly attached to the main elements. Pier foundations are represented as nodal spring elements. Damping ratio for abutments is 5%.

Therefore the inherent damping ratio of SDOF will be assigned as 5%. According to the eigenvalue analysis, the first mode natural frequency is 1.702 Hz and effective mass is 70% of total mass.

(2) Retrofit Cases

In this paper the bridge performance measure is the potential seismic damage of the deck. The gap between abutment and deck is 10 cm. Larger displacement causes permanent deformation at the each end of the deck. The allowable displacement values will be indicated as target displacements herafter. The two dampers will be attached to the abutments of A1 and A2 in Fig. 1.

Four cases are studied: the bridge without dampers and with dampers having the target displacements of 10 cm, 7.5 cm, and 5 cm. Table 1 shows the required supplemental damping ratios where \(\zeta_s\) is inherent damping ratio, \(\zeta_d\) is supplemental damping ratio. (These supplemental damping ratios are for the preliminary design parameters of nonlinear viscous dampers. The lumped weight of SDOF is 1000kN and the design velocity is 1m/s.) Ground motions are the acceleration records of Hyogo-ken Nanbu earthquake; NS and EW component of Takatori and NS component of Fukiai are Type II-II-1, 2 and 3 respectively.

3. ESTABLISHMENT OF NONLINEAR VISCOUS DAMPER CAPACITY

(1) General Characteristics of Nonlinear Viscous Dampers
Nonlinear dampers have lower damper forces than linear dampers have during large structural velocities. Fig. 3 illustrates the damper forces for different values of nonlinearity exponent $\alpha$. It is clear that while their energy dissipation is equal (the area under curves), the damper forces becomes smaller together with the nonlinearity exponent $\alpha$ which takes real positive exponent ranging between 0.1 and 1. Lin and Chopra (2002) showed that nonlinearity has no influence on structural responses as long as the damping ratio in accordance with the limit design deformation is provided to the structure.

### (2) Nonlinear Viscous Damper Parameters

The SDOF system in Fig. 4 represents the nonlinear damper at the abutment of the bridge structure. $m$ is set as 1000(kN)/9.81 (m/s$^2$), $c_d$ is damping coefficient of supplemental viscous damper. The nonlinearity exponent $\alpha$ is 0.1, $c_s$ is the inherent damping coefficient of abutment with respect to the 5% damping ratio and $k$ is established according to the fundamental period($T$)($4\pi^2m/T^2$).

Firstly the energy equivalent method and the iterative solution method are compared to clarify the accuracy of the latter one. In iterative solution method: the equation of motion of a structure with nonlinear viscous damper is formulated as the following equation $m\ddot{x} + c_s\dot{x} + kx + c_d|x|^{\alpha}\text{sgn}(x) = -m\ddot{z}_g$. In this equation initial displacement is zero and design velocity is 1m/s. Maximum displacement is determined (target displacements) beforehand. $\ddot{z}_g$ is ground acceleration. $c_s, k$ are structural properties. The unknown nonlinear viscous damper coefficient is iterated until maximum displacement reaches to the target displacement. Since this approach requires computer programming, many methods to equate nonlinear viscous dampers to linear viscous damper have been recommended $^{5-6}$. Energy equivalent method has been applied where energy dissipated in a cycle of force-displacement curve is given by $^{6}$.

For nonlinear viscous damper ($\alpha < 1$),

$$E_D = \pi\beta_\alpha c_d \omega^\alpha x_0^{\alpha+1}$$

### Table 1 Study cases

<table>
<thead>
<tr>
<th>Cases</th>
<th>Target Displacement</th>
<th>$\zeta_x$ (%)</th>
<th>$\zeta_d$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Undamped structure</td>
<td>5</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>10 cm</td>
<td>5</td>
<td>8</td>
</tr>
<tr>
<td>3</td>
<td>7.5 cm</td>
<td>5</td>
<td>14</td>
</tr>
<tr>
<td>4</td>
<td>5 cm</td>
<td>5</td>
<td>43</td>
</tr>
</tbody>
</table>

### Fig. 3 Five-span girder bridge.

### Fig. 4 SDOF idealization

### Fig. 5 Comparison of iterative solution (red) with energy equivalent method (blue)
The constant, 
\[ \beta = \frac{2^{\alpha+1} \Gamma^2 \left(1 + \alpha / 2 \right)}{\pi \Gamma \left(2 + \alpha \right)} \]  
(1b)

For linear viscous damper (\( \alpha = 1 \)),
\[ E_D = \pi \ c_1 \omega \ x_0^2 \]  
(1c)

The nonlinear damper coefficient can be found from known linear viscous damper coefficient as following equation,
\[ c_a = \left( \frac{\omega \ x_0}{\beta} \right)^{-\alpha} c_1 \]  
(1d)

whereas \( \omega \) is natural frequency of the structure, \( x_0 \) is maximum displacement, \( \alpha \) nonlinearity exponent, \( c_a \) is nonlinear and \( c_1 \) is linear damper coefficient.

The displacement results of SDOF system under Type II-II-I are illustrated in Fig.5. Blue line is for energy-equivalent method, red one is for iterative method. The good agreement between responses is clear.

In Table 2, damping coefficients are presented for each case and each ground motions found by iterative solution. The damper capacity for whole bridge can be determined based on the results obtained from SDOF system for each target displacement for each earthquake wave. The equation for conversion of damper force from SDOF system to bridge structure is given in equation (1e).

Table 2  Nonlinear viscous damping coefficients

<table>
<thead>
<tr>
<th>Earthquake Wave</th>
<th>Target Displacement</th>
<th>Normalized damping coefficient</th>
<th>Viscous damping coefficient (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type II-II-1</td>
<td>0.10</td>
<td>0.126</td>
<td>4638</td>
</tr>
<tr>
<td></td>
<td>0.075</td>
<td>0.180</td>
<td>6626</td>
</tr>
<tr>
<td></td>
<td>0.05</td>
<td>0.345</td>
<td>12699</td>
</tr>
<tr>
<td>Type II-II-2</td>
<td>0.135</td>
<td>0.200</td>
<td>4969</td>
</tr>
<tr>
<td></td>
<td>0.215</td>
<td>7362</td>
<td>12920</td>
</tr>
<tr>
<td>Type II-II-3</td>
<td>0.094</td>
<td>0.194</td>
<td>3460</td>
</tr>
<tr>
<td></td>
<td>0.501</td>
<td>7141</td>
<td>18441</td>
</tr>
</tbody>
</table>

\[ c_a = R \times Y \times W / N \]  
(1e)

whereas \( c_a \) is nonlinear viscous damping coefficient, \( R \) is normalized damping coefficient (damping coefficient of SDOF/1000kN), \( Y \) is effective mass ratio of fundamental mode (1st mode), \( W \) is the weight of whole bridge, \( N \) is the amount of damper (2 dampers herein).

4. DYNAMIC RESPONSES OF BRIDGE

The nonlinear dynamic analysis is performed for the structure with and without dampers to evaluate the effectiveness of nonlinear viscous dampers using the software TDAP III (Three Dimensional Dynamic Analysis Program\( ^{10} \)). Time responses of the pier top and pier base are introduced.

(1) The Response of Pier Top

The dynamic responses of the pier top (P1) before and after installation of nonlinear viscous dampers to the abutments are quantified in terms of displacement ductility ratio (the ratio of displacement value to the yield displacement: \( \mu = \delta / \delta_y \)). Displacement value can be explained by Fig.6 and the following equation,
whereas \( y_1 \) and \( y_2 \) are horizontal displacement of pier top and pier base, respectively, \( l \) length of pier and \( \theta \) is the rotation angle of pier base. Ductility ratio of piers are \( \mu = 4.063 \) in this bridge.

In Fig.7, 8 and 9 the change in ductility ratio as time diagrams can be observed. All cases show that the pier does not fail even before damper installation (Case 1). However maximum ductility ratio of pier under each type of earthquake exceeds the ratio of 1. This means the pier has plastic deformation. After installation of damper to each abutment, ductility ratios reduce under 1 with the exception of Case 2 and Case 3 during Type II-II-3 ground motion. This points out the necessity for larger damper force.

(2) The Response of Pier Base
The bending moments and rotation angles are depicted by Fig.10, 11, 12. From the figures, the energy dissipation of nonlinear viscous dampers is clear. However the necessity for larger damper during Type II-II-3 in Case 2 and 3 can be observed again.
(3) Comparison of the bridge responses with SDOF system

The nonlinear viscous damper parameters have been obtained by SDOF system of simplified bridge structure. In the final step we observed the accuracy of this simplification with the bridge structure with nonlinear viscous dampers in terms of damping force-displacement and damping force-velocity relationship and displacement responses.

The determination of damper parameters can be found out by iteration of simplified method rather than using whole structure system. However the structure responses with damper differs from SDOF system as seen in Fig. 13 and Fig. 14. The velocity and damper force give larger values for bridge structure as expected since the bridge structural velocity and damper force are larger.

The nonlinear viscous damper has been designed to provide previously defined structural deformation limit (target displacements). Therefore the displacement responses of bridge structure (end of deck) and SDOF is expected to give the same results as in Fig.15.

5. CONCLUSION

In this paper the seismic retrofit of a girder highway bridge structure with nonlinear viscous dampers is studied. One damper is attached to each abutment (in total 2 dampers). First, the iterative method is compared with the energy equivalent method to check the accuracy of iterative method on SDOF system. The dynamic analysis is performed for severe ground motions of Level 2. It is aimed that bridge responses stay in elastic range. Therefore the responses were measured at the piers. The results can be summarized as follows;
• Damping coefficient for whole bridge structure can be obtained from simplified SDOF system by means of normalized damping coefficient. The sufficiency in using mass of fundamental mode of bridge structure is approved as observed in seismic responses of the bridge structure subjected to ground motions.

• The iterative solution for nonlinear viscous dampers is not preferable method since it is time consuming and complicated for complex structures. Instead the energy equating method for nonlinear dampers is more practical. However from another point of view as studied in this paper, repetitive solutions of iterative method can become simpler by idealized system of complex structure.

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REFERENCES


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