EFFECT OF DAMPER ON SEISMIC RESPONSE OF CABLE-STAYED BRIDGE USING ENERGY-BASED METHOD

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ABSTRACT

Bridges are a vital element of transportation networks. As any disorder in the maintenance or utilization of bridges may cause irrecoverable incidents, the vulnerability of bridges to earthquakes must be seriously considered. Dampers are extensively used in the handling of earthquake forces; however, their optimization and optimal location have not been clarified. Considering the unjustified use of force-displacement design methods in the seismic design of important structures, such as bridges, the use of an energy-based approach with nonlinearly behaving materials and the concept of hysteretic energy are more rational and practical. Therefore, in this study, the effect of dampers and their location on the seismic response of cable-stayed bridges is investigated by an energy-based method. For this purpose, a case study is examined via a numerical study. The results indicate that, to reduce the bridge’s seismic response to earthquake forces, a damper installed between the deck and pylon is more effective than a damper installed on the cables.

Key words: Control of structure, Cable-stayed Bridge, Damper, Energy-based method.


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1. INTRODUCTION

Typically, long span bridges connect important points within transportation systems, and it is crucial for that connection to be maintained after an earthquake. Therefore, their seismic performance should be acceptable. Cable bridges are one of the most common structural systems for long span bridges. For spans over 200 m, cable bridges are very suitable and
should be considered as one of the main options when selecting the structural system of a long-span bridge. Cable bridges are divided into two types: cable-stayed bridges and suspension bridges [1].

Designers are always investigating safe bridge designs and the retrofitting of existing bridges against earthquakes. Thus, various methods have been utilized to achieve these objectives. The most efficient approach consists of using various structural control systems. Generally, structural control is performed by using tools or members in the structure to control its behavior against lateral forces. In recent decades, the seismic control of structures has drawn considerable attention from engineers.

During strong earthquakes, structures usually exceed their elastic limit. Therefore, the design methods that have been used in recent decades, including force-based methods, do not provide complete information on the nonlinear behavior of structures during an earthquake. These methods usually result in forces that need to be considered. Hence, these forces are not optimal; therefore, the design of a bridge with consideration to these forces is not optimized and does not ensure an acceptable structural performance during an earthquake.

Plastic deformation should be considered when estimating earthquake damage or when determining the required maximum response. Regarding the nature of the earthquake force, the structure, or any member of the structure, is subjected to several opposite forces and different time-dependent deformations. This can be investigated by considering the energy absorbed into the structure.

Thus, the use of energy-based methods by incorporating the nonlinear behavior of the structure and the hysterical energy concept is a good alternative to methods based on force and displacement. The objective of developing the design of a bridge by employing energy-based methods is to minimize the response and hysteretic energy absorbed by the structural members. Thereby, the seismic response of the structure is improved.

The energy-based method was first proposed by Housner in 1956 [2]. In his method, the energy absorption capacity of the structure against large earthquakes was introduced as a structural evaluation parameter, and the equivalent velocity was defined as an indicator of absorbed energy. Therefore, Housner used this velocity as an approximation to the structure’s nonlinear behavior demand. Uang and Bertero were among the pioneers of the energy method. In 1990, they conducted a parametric study on the energy method [3] and obtained the main parameters and their effectiveness on energy. Finally, they defined two types of equilibrium equations for energy: one for absolute energy and another for relative energy. As
they pointed out in their report, the absolute energy was more physically meaningful in comparison to relative energy. Additionally, for structures with low and high periods, the calculations of the input energy were different in absolute and relative terms. In 1994, Fajfar and Vidic studied the input energy, the residual energy spectra, and their ratio [4]. In 2001, Riddell and Garcia, conducted a study on the nonlinear response of single-degree-of-freedom systems (SDOFs) under earthquake excitation. They concluded that by using the amount of energy absorbed through nonlinear deformations, the deformation capacity required to carry out an appropriate design could be achieved [5]. In 2002, Leelataviwat et al. used the concept of energy balance to calculate the design forces for an SDOF structure. Later they expanded the application of the concept to a multi-degree-of-freedom structure [6]. In 2003, Chou and Uang suggested the use of energy as a substitute for force and displacement responses. Accordingly, they used energy as an indicator of time-dependent structural damage [7]. Since then, the energy-based method has been extensively applied to seismic design and analysis. For instance, in 2010, based on the energy-based method and by considering the flexural rigidity and sag-extensibility of the cable, Cheng et al., investigated the damping properties of a cable-damper system [8]. In 2013, Habibi et al. proposed a stepwise multi-mode energy-based design method for seismic retrofitting with passive energy dissipation systems as an alternative to strength and displacement-based methods [9].

In this study, the effect of the damper and its location on the reduction of the cable-stayed bridge’s responses to earthquakes was investigated using an energy-based method and considering the hysterical energy concept. Through a case study, the analysis, modeling, and structural responses against earthquake were evaluated with and without dampers. The obtained results and their interpretation are presented in this paper.

2. BRIEF REVIEW OF ENERGY-BASED METHOD EQUATIONS

In the same way as in other methods, the design carried out in this study was based on the following equation:

\[
\text{Demand} < \text{Capacity} \quad (1)
\]

By considering the dynamic equilibrium equation of an SDOF system, and by integrating it with the displacement occurring during an earthquake, the structural capacity and demand for SDOFs could be defined within an absolute and relative state.

2.1. Absolute State

In this case, the general energy relationship is expressed follows:

\[
E_t = E_k + E_\xi + E_S + E_H + E_f \quad (2)
\]

where \(E_k\) is the absolute kinetic energy, \(E_\xi\) is the damping energy, \(E_A\) is the absorbed energy, \(E_S\) is the elastic strain energy, \(E_H\) is the energy dissipated by the hysteretic or plastic deformation, and \(E_f\) is the absolute input energy. These parameters were obtained from the following equations:

\[
E_k = \frac{1}{2} m v_t^2 \quad (3)
\]

\[
E_\xi = \int c \dot{v} d_v = \int c \dot{v}^2 dt
\]
\[ E_A = \int f_s d\nu = E_s + E_H \]

\[ E_I = \int m \ddot{\nu}_g d\nu_g \]

where \( c \) is the damping coefficient, \( m \) is the mass of the system, \( \nu_t \) is the absolute displacement of the system, \( \nu \) is the relative displacement of the system, \( \nu_g \) is the ground displacement, and \( f_s \) is the force stored in the system. With respect to Equations (1) and (2), the left side of Equation (2) can be assumed equivalent to the structural demand, while its right side is equal to the structural capacity.

2.2. Relative State

In this case, the general energy relationship is as follows:

\[ E'_I = E'_k + E'_\xi + E'_S + \]

where \( E'_k \) is the relative kinetic energy and \( E'_I \) is the relative input energy, which can be obtained as follows:

\[ E'_k = \frac{1}{2} m \dot{\nu}^2 \]

\[ E'_I = -\int m \ddot{\nu}_g d\nu \]

In this case, with respect to Equations (1) and (4), the left side of Equation (4) can be assumed equivalent to the structural demand, while its right side is equal to the structural capacity.

3. MODELING

In this paper, a case study was carried out to investigate the effect of dampers on the reduction of structural responses to earthquakes. Thus, the Vasco da Gama cable-stayed bridge in Lisbon, Portugal, which has been presented by Pedro and Reis in 2010 [10], was modeled. In 2016, Maleki and Shabestari, modeled this bridge and investigated its response modification factor [11]. In the present work, their model was modified and considered as a case study. The bridge was modeled using the CSI Bridge 2017 software, which is developed by Computers and Structures, Inc.

![Image of Vasco da Gama cable-stayed bridge in Lisbon, Portugal](image-url)
In this study, the Vasco da Gama cable-stayed bridge was modeled with applied loading under three different conditions, as follows:

- Without damper
- With dampers on the bridge cables
- With dampers between the deck and pylons

### 3.1. Model A: Cable-Stayed Bridge without Damper

#### 3.1.1. Modeling specifications and assumptions

As can be seen in Fig. 4, the bridge has three spans. The length of the main span is 300 m, while the length of the two side spans is 120 m.
Effect of Damper on Seismic Response of Cable-Stayed Bridge Using Energy-Based Method

In this bridge, a semi-fan cable arrangement system is used, and there are 56 cables attached to each pylon in the two planes on the left and right sides of the deck, respectively. In the main span, the spacing of the cables is 10 m, while the side span spacing is 8 m. The cables were modeled by frame elements, and their section was defined such that it represented a 30-strand cable. By modeling the cables in this way, their flexural stiffness could be considered. The geometric nonlinear behavior of the cables was captured by large displacement analysis. The labels of the cables in the model are presented in Fig. 6.

Additionally, due to the fact of the deck being modeled by a single element, and of the cables being located on both sides of the deck, rigid elements were used to make cables with an exact angle. These elements were modeled by link elements and defined as rigid elements.

To capture the nonlinear behavior of the structure, many nonlinear concentrated hinges were defined as fiber hinges. These hinges consider the nonlinear behavior of elements in the combination of axial forces and flexural moments simultaneously. To define these hinges, the sections of the elements were divided into many smaller elements.
3.1.2. Loading Specifications

In this model, various loads were applied to the bridge. The most important applied load was the dead load due to the weight of the constituent elements. The live load was defined based on the AASHTO specifications and with an impact factor of 1.3. The earthquake loads were assigned to the bridge according to the AASHTO specifications and with consideration to the non-urban bridge guidelines [12].

After developing a complete model of the structure and its loading, nonlinear time history analysis was used with consideration to the P-delta and large displacement effects. For each time history analysis, three datasets were assigned in each direction. The Northridge, El Centro, and San Fernando records were used as the time history datasets matching the design spectrum. Respectively, these three earthquake records were considered as Fema 1-1, Fema 6-1, and Fema 21-1, as shown in Fig. 7.

![Figure 7 Time history of Fema 1-1, Fema 6-1, and Fema 21-1. a) 1 dir; b) 2 dir; 3) up dir](image)

3.1.3. Cable Forces

In cable-stayed bridges, the vertical displacement of the deck due to the dead load should be very small, and the bridge should have a predetermined longitudinal profile. Therefore, the
cable tension on these bridges was determined accordingly. Thus, the deck of the bridge was modeled as a beam, whose supports were rollers aligned with the cables. Then, the dead loads were applied to the beam and the forces of the cables were obtained by analyzing the beam. The basic specifications of the cables were considered based on these forces. Subsequently, by using a trial and error process, the cable forces changed until the deflection profile under the dead loads was close enough to the predetermined desired state. The results of this trial and error process are presented in Tables 1 and 2.

**Table 1** Force and deck displacement in cables b1 to b14

<table>
<thead>
<tr>
<th>cable name</th>
<th>cable force in deck level (ton)</th>
<th>displacement of deck level (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>b1</td>
<td>175.38</td>
<td>-2.17</td>
</tr>
<tr>
<td>b2</td>
<td>172.4</td>
<td>-3.57</td>
</tr>
<tr>
<td>b3</td>
<td>168.86</td>
<td>-4.26</td>
</tr>
<tr>
<td>b4</td>
<td>164.8</td>
<td>-4.34</td>
</tr>
<tr>
<td>b5</td>
<td>160.25</td>
<td>-3.97</td>
</tr>
<tr>
<td>b6</td>
<td>155.31</td>
<td>-3.3</td>
</tr>
<tr>
<td>b7</td>
<td>150.1</td>
<td>-2.55</td>
</tr>
<tr>
<td>b8</td>
<td>144.77</td>
<td>-1.94</td>
</tr>
<tr>
<td>b9</td>
<td>139.54</td>
<td>-1.74</td>
</tr>
<tr>
<td>b10</td>
<td>134.79</td>
<td>-2.2</td>
</tr>
<tr>
<td>b11</td>
<td>131.12</td>
<td>-3.57</td>
</tr>
<tr>
<td>b12</td>
<td>129.42</td>
<td>-5.92</td>
</tr>
<tr>
<td>b13</td>
<td>130.65</td>
<td>-8.95</td>
</tr>
<tr>
<td>b14</td>
<td>134.57</td>
<td>-11.68</td>
</tr>
</tbody>
</table>

**Table 2** Force and deck displacement in cables c1 to c14

<table>
<thead>
<tr>
<th>cable name</th>
<th>cable force in deck level (ton)</th>
<th>displacement of deck level (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>c1</td>
<td>180.89</td>
<td>-4.85</td>
</tr>
<tr>
<td>c2</td>
<td>180.88</td>
<td>1.24</td>
</tr>
<tr>
<td>c3</td>
<td>181.39</td>
<td>5.4</td>
</tr>
<tr>
<td>c4</td>
<td>183.12</td>
<td>5.68</td>
</tr>
<tr>
<td>c5</td>
<td>112.87</td>
<td>2.9</td>
</tr>
<tr>
<td>c6</td>
<td>115.5</td>
<td>0.98</td>
</tr>
<tr>
<td>c7</td>
<td>117.17</td>
<td>1.73</td>
</tr>
<tr>
<td>c8</td>
<td>117.26</td>
<td>5.12</td>
</tr>
<tr>
<td>c9</td>
<td>115.62</td>
<td>9.94</td>
</tr>
<tr>
<td>c10</td>
<td>112.58</td>
<td>14.35</td>
</tr>
<tr>
<td>c11</td>
<td>109.22</td>
<td>16.4</td>
</tr>
<tr>
<td>c12</td>
<td>107.64</td>
<td>14.47</td>
</tr>
<tr>
<td>c13</td>
<td>111.18</td>
<td>7.87</td>
</tr>
<tr>
<td>c14</td>
<td>122.83</td>
<td>-2.36</td>
</tr>
</tbody>
</table>

**3.2. Model B: Cable-Stayed Bridge with Damper Installed on Cables**

Under this condition, a nonlinear damper with a stiffness of 100,000 $\frac{KN}{m}$ and a damping factor of 15.13 $\frac{KN.m}{s}$ was attached to each cable. Each damper was connected to the deck on one side, and to the cable on the other side. The location of the damper on the cable was at a distance equal to 0.06 of the cable length before the damper was attached. Time history analysis was also conducted under this condition.
3.3. Model C: Cable-Stayed Bridge with Damper between Deck and Pylons

Under this condition, a nonlinear viscous damper, with a stiffness of 500,000 \( \frac{KN}{m} \), damping coefficient of 2,000 \( \frac{K.N.m}{s} \), and \( \alpha \) equal to 0.5, was placed at the transverse beam location between the deck and pylons. Time history analysis was also conducted under this condition.

4. RESULTS AND FIGURES

The diagrams showing the performance of each model are presented according to the results obtained by the nonlinear time history analysis of the three described structural models.

First, the damper’s energy was investigated as a parameter in the evaluation of its performance. The diagrams below show the energy of the utilized damper in the modeling carried out with regard to each of the three earthquake records. As can be seen in the diagrams, the energy of the dampers in model B is negligible in comparison to the energy in model C. This suggests that these dampers are not efficient and that their placement in the structure does not affect the structure’s response. Therefore, the different structural responses were compared under the conditions of A and C, as shown in Figures 9, 10, and 11.

![Figure 8 Damper energy in Models B and C. a) Fema 1-1; b) Fema 6-1; c) Fema 21-1](image-url)
Figure 9 Results of nonlinear time history Fema 1-1 analysis for three described structural models. a) Time history of input energy; b) time history of hysteretic energy; c) time history of kinetic energy; d) time history of base shear; e) time history of base moment; f) time history longitudinal displacement at mid-span; g) time history vertical displacement at mid-span; ( Cable-stayed bridge without damper, Cable-stayed bridge with cable damper, Cable-stayed bridge with damper between deck and pylon)
Figure 10 Results of nonlinear time history Fema 6-1 analysis for three described structural models. a) Time history of input energy; b) time history of hysteretic energy; c) time history of kinetic energy; d) time history of base shear; e) time history of base moment; f) time history longitudinal displacement at mid-span; g) time history of vertical displacement at mid-span; (Cable-stayed bridge without damper, Cable-stayed bridge with cable damper, Cable-stayed bridge with damper between deck and pylon)
**Effect of Damper on Seismic Response of Cable-Stayed Bridge Using Energy-Based Method**

**Figure 11** Results of nonlinear time history Fema 21-1 analysis for three described structural models.  
(a) Time history of input energy; b) time history of hysteretic energy; c) time history of kinetic energy; 
d) time history of base shear; e) time history of base moment; f) time history longitudinal displacement  
at mid-span; g) time history vertical displacement at mid-span; (■ Cable-stayed bridge without  
damper, ■ Cable-stayed bridge with cable damper, ■ Cable-stayed bridge with damper between deck  
and pylon)

Moreover, in Table 3, the maximum values for each structural response in Models A and C are presented for Fema 1-1.

**Table 3** Maximum structural response values in Models A and C for Fema 1-1

<table>
<thead>
<tr>
<th></th>
<th>Model A</th>
<th>Model C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input energy (J)</td>
<td>1.19E+07</td>
<td>1.21E+07</td>
</tr>
<tr>
<td>Link hysteretic energy (J)</td>
<td>1.49E+06</td>
<td>9.10E+05</td>
</tr>
<tr>
<td>Kinematic energy (J)</td>
<td>4.46E+06</td>
<td>4.55E+06</td>
</tr>
<tr>
<td>Base shear (N)</td>
<td>1.21E+07</td>
<td>8.73E+06</td>
</tr>
<tr>
<td>Base moment (N-m)</td>
<td>-5.35E+10</td>
<td>-5.37E+10</td>
</tr>
<tr>
<td>Longitudinal displacement (m)</td>
<td>8.21E-02</td>
<td>8.79E-02</td>
</tr>
<tr>
<td>Vertical displacement (m)</td>
<td>9.99E-02</td>
<td>5.62E-02</td>
</tr>
<tr>
<td>Damper energy (J)</td>
<td>0.00E+00</td>
<td>4.22E+06</td>
</tr>
</tbody>
</table>
Based on Equation 2, the input energy was equal to the sum of the strain, kinetic, damping, and hysteretic energy. Therefore, according to the Energy Conservation Law, as the energy dissipated by the dampers increased, the structural responses, such as the displacement, velocity, and acceleration, were reduced as a consequence of the decreasing strain and kinetic energy. Additionally, at a certain seismic level, the lost hysteresis energy owing to the nonlinear behavior of members, such as the formation of plastic hinges, was reduced. Therefore, the structural damage caused by the earthquake was also reduced.

In the diagrams presented above, it can be seen that by adding a damper between the pylon and deck, the maximum hysterical and kinetic energy values of the structure decreased by approximately 35% and 5%, respectively. The maximum base shear value at the end of the pylon was reduced by approximately 25%. Moreover, the time until the occurrence of the maximum values was decreased, and the maximum longitudinal and transverse displacement values at mid-span were reduced by approximately 7% and 40%, respectively.

5. CONCLUSIONS

In this study, the effects of dampers on the responses of a cable-stayed bridge to earthquake were investigated using an energy-based method. Specifically, a case study was carried out regarding the effect exerted by dampers placed on the cables and the effect exerted by dampers placed between the pylons and deck on the structural responses to the recorded Northridge, El Centro, and San Fernando earthquakes. The numerical modeling results indicated that the dampers on the cables did not have any effect on the bridge’s structural responses to earthquakes. However, the damper between the pylon and deck reduced the structural responses considerably, including the hysterical energy, base shear, and vertical displacement responses. It was also shown that a large portion of the seismic force acting on the bridge was related to the weight of the pylons. For further investigations on the performance improvement of cable-stayed bridges, it is suggested that consideration be given to the behavior of the pylons and control systems.

REFERENCES

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