Performance evaluation of dynamic pulley damper system for installation angle

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Abstract. Dynamic pulley damper system (DPDS) as an innovative methodology for a response control system using a block and tackle mechanism to provide enhanced vibration reduction in high-rise buildings, has been proposed. The key aspect of this system is to exponentially increase the damper movement using an amplification mechanism comprising movable pulleys and a wire. This paper presents the relation between the arrangement of the wire’s angles, particularly in the amplification part, and the vibration reduction effect. The DPDS is introduced into a center-core high-rise building. Two small building varieties with different core heights are designed; then, a wire is stretched between the frame and the core structures. The capability of the DPDS to control vibration is confirmed via the shaking table test. Additionally, the accuracy of the simulation models is verified by a comparison with the experimental results. Moreover, this paper includes a parametric analysis test to determine the wire-stretching angle for establishing the most efficient design. The results of the above analysis show that the wire-stretching angle influences the seismic performance of the DPDS.

1. Introduction
To meet building integrity requirements and ensure the comfort of occupants, vibration control systems have been widely used in structural design, particularly in high-rise buildings. However, during the 2011 Great East Japan Earthquake, high-rise buildings in Tokyo and Osaka located 770 km from the epicenter of the quake swayed significantly for a protracted time, primarily due to the long period of ground motion, even where vibration control devices such as oil dampers and steel braces had been in place [1]. Damage to several non-structural elements such as ceiling panels was also reported, in addition to the damage to fire protection systems, and people became trapped in elevators. An analysis suggested that vibration control devices may only provide a maximum reduction response of 10%–20% in acceleration during an earthquake [2]. Furthermore, since the bending deformation in tall buildings during an earthquake are significantly increased by column stretching, the response reduction effect of damping devices, which operate based on story drift, cannot function to full effect. Accordingly, a new building vibration control system using a damper and block-tackle mechanism named as a dynamic pulley damper system (DPDS) was proposed [3, 4].
2. Constitutive formula for the dynamic pulley damper system

2.1. Fundamental mechanism of the dynamic pulley damper system

The basic configuration of the DPDS comprises an amplification system using movable pulleys with a wire and a damper installed on the wire movement path. The DPDS enables maximizing the damper energy absorption by increasing both the wire movement and damper deformation using the amplification mechanism. Figure 1(a) shows a construction diagram for the DPDS in a central-core high-rise building. A wire is stretched reciprocatively between the frame structure (a condominium tower) and the core structure (a parking tower) using pulleys. The fundamental principle of the amplification mechanism is explained in Figure 1(b). A wire is reciprocated \( n \) times between Pulley Group A (a frame structure) and Pulley Group B (a core structure). When the building is shaken during an earthquake, based on its displacement \( \mathcal{P} \), the degree of damper deformation is expected to be \( n \times \mathcal{P} \). Therefore, the enlarged damping force acts on the building in proportion to the number of wire reciprocation \( n \). Hence, the response direction of the wires and the building moving will be the same when the wire arrangement angle \( \theta \) is an acute angle, as shown in Figure 1(c).

A basic constitutive equation was developed for the DPDS, including elongation of the wires [5]. If the wires and their properties are symmetrically stretched, as depicted in Figure 2, the force–deformation relation of the model can be expressed as equation (1).

\[
F = \left(\frac{2n^2 \cos^2 \theta}{n/K_{pw} + 1/K_{hw} + 2/k_d}\right) \mathcal{P}
\]

(1)

Where, \( K_{pw} \) is the axial stiffness of the wire in the diagonal part, \( K_{hw} \) is the axial stiffness of the wire in the horizontal part, and \( k_d \) is the stiffness of the damper.

![Construction diagram](image1)

![Simplified mechanical model](image2)

![Response direction](image3)

**Figure 1.** Basic configuration of the DPDS.

**Figure 2.** Configuration model of the symmetrical arrangement of the DPDS.
2.2. Formulation of the dynamic pulley damper system equation

The wires were modeled as truss elements in this analysis to represent the force–deformation relation between wires, as shown in Figure 3. Figure 3(a) shows the system arrangement in a one-story structure, while Figure 3(b) shows the analysis model. Since a continuous wire was used in the DPDS, the total length of the wire had to be equal at all times, so that the deformation relation between damper and wire could be expressed as equation (2). The incremental form of force–deformation relation on the truss is established in equation (3).

\[
x = \Omega \left( -n \delta_1 + \delta_{pw} + \delta_{ph} \right) = \Omega \frac{n}{\alpha k_d + 2} \begin{bmatrix} 1 & -1 \end{bmatrix} \begin{bmatrix} \delta_1 \\ \delta_2 \end{bmatrix} \tag{2}
\]

\[
\begin{bmatrix} \Delta N_1 \\ \Delta N_2 \end{bmatrix} = \frac{n^2}{2(\alpha + 2/k_d)} \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \begin{bmatrix} \Delta \delta_1 \\ \Delta \delta_2 \end{bmatrix} \tag{3}
\]

In the above calculation, \(\delta_{pw}, \delta_{hw}\) are the wire deformation in the pulley distance \(L_{pw}, L_{hw}\), \(\alpha = n/K_{pw} + 1/K_{hw}, N_1, N_2\) are the axial forces, and \(\delta_1, \delta_2\) are the truss deformations. \(\Omega\) is the parameter of the wire arrangement (\(= 1 \ (0^\circ \leq \theta < 90^\circ), = -1 \ (90^\circ \leq \theta < 180^\circ)\)).

Accordingly, the frame analysis was implemented as follows.

Step1. Calculate the damper deformation \(x\) from the axial force \(N_1, N_2\) and the deformation \(\delta_1, \delta_2\) on the truss using equation (2).

Step2. Obtain damper force \(Q(x)\) using the formula for the force–deformation relationship.

Step3. Update the truss axial force using equation (3).

![Figure 3. Configuration model for the DPDS.](image)

3. Experimental investigation using a shaking table

Since the system comprises a complex configuration and mechanism, verification experiments were first required to demonstrate the DPDS’ damping capacity. This section describes the experimental arrangement, including the specimen body and measurement instruments. It also presents the results of the shaking table test, i.e., with and without DPDS using the fourth story steel frame specimen.

3.1. Specimen and the measurement system

Figure 4 illustrates the specimen scheme including the experiment instruments for conducting the shaking table tests. The specimen comprised a four-story frame structure and a core structure with two different heights, i.e., a high-height core model (Figure 4(a)) and a low-height core model (Figure 4(b)). The total height of the specimen body was 1,040 mm. The core part height was 580 mm for the high core model and 320 mm for the low core model. The specimen was designed with a time scale of \(1/\sqrt{10}\) times the real scale of a high-rise building with 2.8 seconds natural period. The fundamental period of the frame structure was 0.9 seconds. The specimen floor was constructed using a steel plate.
(SS400) with a length of 320 mm and a width of 160 mm, and the pillar was constructed from ultraduralumin (A7075) with a thickness of 2 mm, which remained elastic up to large story drifts. A steel damper of approximately 14 N yield strength was placed at the third story, and the end of the wire extending from the core structure was connected to the steel damper. Additionally, 30 N of the initial tension force was introduced and adjusted by turning up the turnbuckles. Here the wire loops were 3.5 times for the high core model, and 2.5 times for the low core model.

The layout for the measuring instruments is explained in Figure 4. Details regarding the instruments are as follows. Seven laser displacement sensors (IL-S100 and IL-300; Keyence Co., Ltd., Osaka, Japan) and six accelerometers (AFR-10A and AFR-20A; Tokyo Measuring Instruments Laboratory Co., Ltd., Tokyo, Japan) were installed for the lateral displacement and acceleration of each story, and four tension measuring plates with strain gauges were used to obtain the wire tension and the damping force of the steel damper.

![Diagram of measuring instruments](image)

(a) Fourth story frame structure with the second story core structure (high core model)  
(b) Fourth story frame structure with the first story core structure (low core model)

**Figure 4.** Shaking table test scheme and the experiment instruments.

### 3.2. Test cases and input waves

Table 1 summarizes the experimental test cases. Case F is the test model without DPDS and Case FWD is the test model with DPDS for different core heights. Figure 5 also shows the input acceleration waves used in the shaking table test, i.e., the 1940 El Centro NS wave and the 1968 Hachinohe NS wave, scaled to have a maximum velocity of 50 cm/s (50 kine). The time-step of each wave was multiplied by $1/\sqrt{10}$ to correspond to the specimen scale.

**Table 1.** Shaking table test models.

| Test Case | Case F | Case FWD_high | Case FWD_low |
|-----------|--------|---------------|--------------|
| Core Height | —      | 580 mm        | 320 mm       |
| Wire Loop | —      | 3.5 times     | 2.5 times    |
| Damper    | —      | Steel damper at 3rd story | Steel damper at 3rd story |
3.3. Experimental results

The maximum story displacement of the specimen is shown in Figure 6. In the figure, the black and red lines indicate Case F and Case FWD, respectively. The third story of the specimen, where the damper was installed, indicated the smallest displacement. Figure 7 compares the time histories of story displacement under the El Centro wave with a 30% magnification of the input acceleration wave (maximum acceleration 100 gal). Case FWD reduced more than 80% of the maximum story displacement in both the high and low core height models.

![Graphs showing story displacements under El Centro wave](image)

**Figure 6.** Story displacements under the El Centro wave (30%) (max. acc. 100 gal).

![Graphs showing displacement responses at fourth story](image)

**Figure 7.** Displacement responses at the fourth story under the El Centro wave (30%) (max. acc. 100 gal).

4. Analytical investigation

The simulation analyses for the experimental model were carried out using a software STERA_3D (Version 10.5) to examine its validity of analytical results via a comparison with the experimental results. Simulation model for high core model is shown in Figure 8. In the analytical model, the young’s modulus 60 GPA of the steel frame were determined to have the same fundamental period as Case F in experiment. Besides, the both of frictional force and damping force of steel damper were considered as bilinear behavior as shown in Table 3 and Figure 9. The frictional force generated at the contact between pulleys and wire was estimated by using equation (4) which was obtained from the static friction test and the dynamic friction test [6]. It is then, two damping forces were combined as one bilinear damper, and the yield force $F_y$ was given by equation (5).
Figure 8. Simulation Model.

Figure 9. Hysteretic performance.

Table 2. Damper parameters.

| Case          | Damper               | Initial Stiffness $K_0$ | Stiffness Ratio $K_1/K_0$ | Yield Point $F_{yf}, F_{yd}$ |
|---------------|----------------------|-------------------------|----------------------------|-----------------------------|
| High Core     | Friction Steel Damper| 1.09 kN/mm              | 0.144                      | 2.9 N, 9.3 N                |
| Low Core      | Friction Steel Damper| 1.09 kN/mm              | 0.144                      | 2.1 N, 9.3 N                |

\[
F_{yf} = 0.007 \times T_{initial} \times P_i \tag{4}
\]

\[
F_Y = F_{yf} + F_{yd} \tag{5}
\]

where, $F_{yf}, F_{yd}$; yield point force of friction and steel damper, respectively, $T_{initial}$; initial tension introduced to the wire, $P_i$; the number of pulleys.

Figure 10 and Figure 11 compare the displacement time histories for each story under the El Centro and Hachinohe waves. While experimental and analytical results at maximum displacement were nearly a match for all stories, the analytical results were small for CaseFWD_high and large for CaseFWD_low compared with the experimental results, respectively.

Figure 10. Time histories for story displacement for CaseFWD_low under the El Centro wave.
5. Parametric study for the wire-stretching angle

This section presents a parametric study using the same analytical model presented in the previous dynamic analysis with 2.5 wire loops to determine the most efficient wire-stretching angle for the DPDS design.

5.1. Simulation models

As shown in Equation (1), the damping force decreases as the wire-stretching angle $\theta_{\text{wire}}$ increases. However, in case of the proposed system that connects the frame and the core structures, other factors such as the fundamental period of the building and the characteristics of seismic waves must be taken into consideration. For this purpose, the simulation models were prepared by changing $\theta_{\text{wire}}$ at 10° intervals between 5° to 75° as detailed in Table 3 and Figure 12. Then, the dynamic response analysis were performed using the El Centro NS 50 kine and Hachinohe NS 50 kine, as depicted in Figure 5. Also, the static push over analysis with a triangular-type horizontal load distribution along the height of the simulation model was carried out.

| Analytical Model | Case1 | Case2 | Case3 | Case4 | Case5 | Case6 | Case7 | Case8 |
|------------------|-------|-------|-------|-------|-------|-------|-------|-------|
| Wire-stretching Angle $\theta_{\text{wire}}$ [degree] | 5 | 15 | 25 | 35 | 45 | 55 | 65 | 75 |
| Core Height $H_{\text{core}}$ [mm] | 764 | 730 | 693 | 649 | 593 | 513 | 379 | 82 |
| Fundamental Period [sec] | 0.368 | 0.370 | 0.373 | 0.379 | 0.390 | 0.476 | 0.544 | 0.689 |
5.2. Pushover analysis results
The response control effect of the DPDS for each angle was evaluated using the coefficient of vibration control performance (CVCP), which is calculated from the following equation (6). Since the amplification of the wire movement is maximized when the wire is stretched horizontally, the damping effect is utmost as well. Thereby, the CVCP shows the effectiveness of the DPDS from the comparison with the story displacement at 5° angle. Accordingly, the closer the coefficient is to 0, the smaller the vibration reduction effect during an earthquake; the closer it is to 1, the higher the effect.

\[
CVCP = \frac{i \text{- story displacement at } 5^\circ}{i \text{- story displacement at } \theta_{wire}} \left\{ \begin{array}{l}
\text{i \cdots 1, 2, 3, 4} \\
\text{\theta_{wire} \cdots 5^\circ \sim 75^\circ}
\end{array} \right.
\] (6)

Figure 13 compares the CVCP obtained from equation (1) (CVCP_formula) and the results of the pushover analysis (CVCP_pushover). It is seen that all results of CVCP_pushover are higher than CVCP_formula. Furthermore, it is revealed that the wire-stretching angle \( \theta_{wire} \) significantly influence on the deformation at third story where the damper was installed. However, the other story did not have big change in CVCP until \( \theta_{wire} \) reaches approximately 40°.

![Figure 12. Analytical model for the parametric study.](image)

![Figure 13. CVCP and angle relationship for the formula and pushover analysis.](image)

5.3. Results of the time history analysis
Figure 14 shows the results of the time history analysis at 10° intervals for each story under El Centro wave and Hachinohe wave. Although the CVCP at the third story indicated lower value compared with the other stories, its reduction could not be observed until the wire-stretching angle reached approximately 40°. Moreover, as shown in Figure 15 regarding the maximum story displacement, story deformation was the smallest at 45° (blue line with markers). The CVCP values of static and dynamic analyses are compared in Figure 16. Both results are similar each other and higher than the CVCP_formula. Therefore, the DPDS was capable of providing an adequate vibration reduction effect in high-rise buildings that need angles for wire arrangement to couple the core and the frame structure.
6. Conclusion
An innovative and economical vibration mitigating system for high-rise buildings was developed using a movable pulley amplification mechanism. Following on, an experimental and analytical investigation of dynamic pulley damper system (DPDS) was conducted. Moreover, a parametric study was performed to evaluate the performance of this system based on changing the wire-stretching angle. The significant conclusions drawn from the present study are as follows:

- The effectiveness of the DPDS to reduce building vibration during seismic excitation was established using the shaking table test.
- The validity of the simulation model was confirmed by comparison with the experimental results.
- The vibration control effect of the DPDS by changing wire-stretching angle was investigated based on the parametric analytical study. The results demonstrated that the DPDS provides same effect as 5° until 45°. Therefore, the DPDS is efficient and capable in high-rise buildings that need angle for wire arrangement to couple the core and the frame structure.
- Future studies incorporating simulation analysis for full-scale models are needed to evaluate dynamic system behavior and to present a construction design for practical use.
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