ADAPTIVE TUNING OF HYSTERETIC MASS DAMPER FOR REDUCING SEISMIC RESPONSE IN NONLINEAR MULTI-STORY BUILDINGS

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Abstract

In recent years, large tuned mass dampers (TMD) have been developed to mitigate seismic damage to super high-rise buildings subjected to long-period ground motions including a great Nankai Trough earthquake, which possibly strike a wide area of Japan in the near future. Although these large TMDs have been mainly applied to existing buildings for seismic rehabilitation, TMDs are expected in newly-built structures. In some buildings, their seismic effectiveness was clarified using observation data generated by moderate ground motions. However, linear elastic TMDs might lose their control performance beyond the elastic response of buildings during strong ground motion, because the equivalent natural period of the buildings become longer.

To overcome the inevitable loss of control performance by a conventional controlled method, we propose a nonlinear TMD for reducing seismic damage to steel structures or buildings. In this study, both the TMD and controlled steel buildings are considered systems having normal bilinear hysteresis. The nonlinear TMD can be adaptively tuned according to the extent of the nonlinearity of a building in terms of equivalent frequency during the elastic-plastic response. In addition, sufficient damping is obtained by inherent hysteresis of the TMD without mechanical damping. A strategy of the initial optimization is based on a stochastic vibration theory with equivalent linearization technique. Closed expression for optimizing the TMD is employed, which was proposed by the author in previous research. For optimized TMDs regarding a wide variety of nonlinear characteristics of controlled structures, numerical examples are demonstrated using stick models subjected to simulated ground motions. Time history analysis shows that the proposed TMD with five percent mass ratio can reduce peak displacement in buildings by approximately twenty percent even beyond its elastic range. It is also confirmed that the TMDs ensure a certain supplemental damping ratio over a wide variety of ductility factors. For TMDs with different mass ratios ranging from 0.02 to 0.1, the peak responses are well predicted using a proposed method based on the response spectrum method. Both the proposed optimal tuning method and estimation method of the peak seismic response allow performance design for the proposed nonlinear TMD in practical engineering.

Keywords: Tuned mass damper, Optimization, Nonlinear response, Robustness, Optimal tuning ratio

1. Introduction

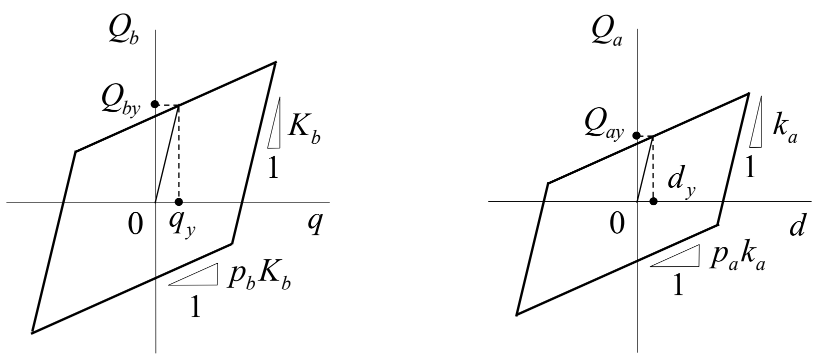
In recent years, large tuned mass dampers (TMD) have been developed to mitigate seismic damage to super high-rise buildings subjected to long-period ground motions including a great Nankai Trough earthquake, which possibly strike a wide area of Japan in the near future. Although these large TMDs have been mainly applied to existing buildings for seismic rehabilitation [1, 2], TMDs are also expected in newly-built structures. In some buildings, their seismic effectiveness has been clarified through structural monitoring that recorded moderate ground motions. However, linear elastic TMDs might lose their control performance beyond the elastic response of buildings during strong ground motion, because the equivalent natural period of the controlled buildings become much longer.

To overcome the inevitable shortcomings arising from a conventional controlled method, we propose a nonlinear TMD for reducing seismic damage to steel structures or buildings. In this study, both the TMD and controlled steel buildings are considered systems having normal bilinear hysteresis. The nonlinear TMD can be adaptively tuned according to the extent of the nonlinearity of a building in terms of equivalent frequency during the elastic-plastic response. In addition, sufficient damping is given to a building by inherent hysteresis of the TMD without mechanical damping.

This paper demonstrates the seismic effectiveness of nonlinear TMDs throuht time history analysis with shear-type lumped mass model. We present a practical estimation method based on stochastic vibration theory and response spectrum method for evaluating the peak displacement of a controlled building. This method allows us to obtain the required mass of a nonlinear TMD for a given seismic criterion.

2. Tuning of hysteretic TMD

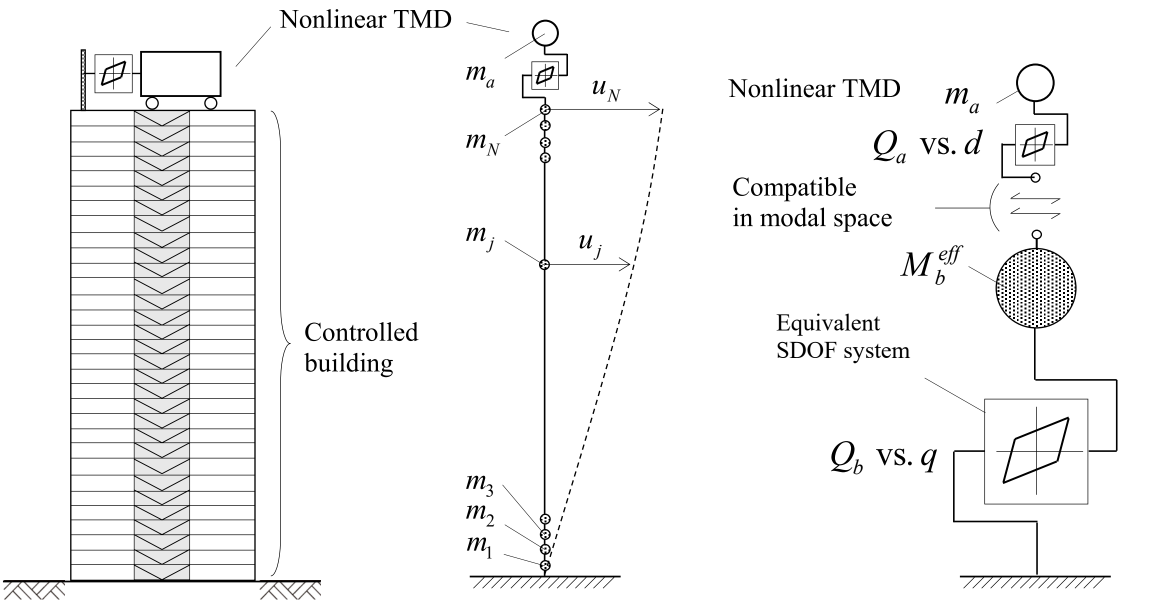
Consider a nonlinear TMD with the mass m_a mounted on a steel building subjected to ground acceleration $\ddot{u}_g$. Their envelope curves are assumed to be bilinear type, as illustrated in Fig. 1. Structural engineers have to find the initial and tangent stiffness $k_a, p_a k_a$, and the yielding strength Q_{ay} regarding optimal properties of the installed TMD, while a classical linear TMD is often determined by the closed formulae [3, 4]. Finding an optimal property might rely on time history analysis through try and error so that the nonlinear TMD is adaptively tuned together with varying equivalent natural period during strong ground motion.



(a) Controlled building (equivalent SDOF model) (b) TMD

Fig. 1 Envelope curves for specifying nonlinear characteristics

Instead of using this complicated calculation, we propose a design procedure for finding an optimal TMD based on stochastic vibration theory with an equivalent linearization method. This practical optimization was originally developed by using the equivalent two-degree of freedom model regarding a TMD-building system to reduce the first modal response. For this reason, the optimization procedure uses modal properties regarding the equivalent SDOF model for the controlled building. The procedure is summarized in the following steps.



(a) Controlled building (b) Shear-type lumped mass model (c) Two degree of freedom model

Fig. 2 Target building and corresponding vibration system

a) Determine the TMD mass ratios to the controlled building $\mu, \bar{\mu}$, which are respectively determined by

$$\mu = m_a / M_b^{eff}, \quad \bar{\mu} = \psi^2 \mu \tag{1a, b}$$

where ψ is the component of modal participation vector computed at the top floor, which may range from 1.3 to 1.5 in multi-story buildings. Notably, this value should be evaluated using the corresponding uncontrolled buildings having the effective mass M_b^{eff}, as shown in Fig. 2.

b) Compute the optimal TMD stiffness in case of undamped elastic buildings.

On the basis of H_2 norm optimization, the optimal stiffness k_a takes the form of

$$\gamma_0^* \approx \frac{\sqrt{1-\bar{\mu}/6}}{1+\bar{\mu}} \tag{2a}$$

$$k_a = m_a \omega_a^2, \quad \omega_a = \gamma_0^* \omega_b \tag{2b, c}$$

where ω_b are the natural circular frequencies of the building without TMD. This closed form of the tuning ratio γ_0^* is slightly different from popular one [4], because this formula considers an almost undamped TMD.

c) Select the target ductility factor D_{target} in which the TMD works most effectively.

Selected D_{target} should be more than 1.0 because the assumed TMD has no damping before its yielding. This step requires engineering judgment.

d) Compute the optimal TMD strength.

The optimal TMD strength Q_{ay} is related to optimal damping, and strongly affected by the building strength Q_{by}. Consequently, the optimal Q_{ay} can be written with non-dimensional parameters.

$$Q_{ay} = \alpha_{ay}^{*} m_a g\,, \quad \alpha_{ay}^{*} = \bar{\alpha}_y^{*} \alpha_{by} \tag{3a, b}$$

where g is gravity acceleration. Their normalized factors are summarized as follows.

$$\bar{\alpha}_y^{*} = \kappa \left\{ 1 + p_b \left(D_{\text{target}} - 1 \right) \right\} \bar{\alpha}_y' \tag{4}$$

For $\bar{\alpha}_y^{*} = \bar{\alpha}_y'$ (or $\kappa = 1$), both the TMD and building begin to yield simultaneously, that Abe discussed in the literature [5]. Because this case is not optimal, the value of κ is empirically chosen to be 0.25, where the TMD yields and produces damping during the building responses elastically. On the basis of stochastic theory, $\bar{\alpha}_y'$ is derived in the following form.

$$\bar{\alpha}_y' = \sqrt{2}\psi\, \gamma_0^{*2} \chi_0^{*} \tag{5}$$

χ_0^{*} is the deformation ratio (d / u_N) in a reference system that is identical to an elastic building with a classical Voigt-type TMD tuned. χ_0^{*} can be accurately estimated using

$$\chi_0^{*} \approx \frac{1}{\sqrt{2\mu}} + 0.83\sqrt{\mu} \tag{6}$$

These values are based on the strategy that the TMD having no dashpot yields ahead of the building to obtain sufficient damping.

e) Compute the optimal second stiffness of the TMD.

The parameter p_a is selected so that seismic electiveness is kept over a range of the extent of plastification as far as possible. We can estimate the reasonable p_a using the following empirical formula.

$$p_a = \frac{1}{1 + c\left(1 - p_b\right)\mu} p_b \tag{7}$$

where the regression parameter c is chosen to be 18.

3. Seismic effectiveness of TMD based on time history analysis

3.1 Target buildings and selected ground motions

As numerical examples, we assume that TMDs contribute to reducing the number of required braces embedded into a steel frame or realizing a higher damped TMD-building system.

Consider three steel buildings with 10, 14, and 20 stories. The last one is categorized in super-high rise buildings in Japan, and is potentially subjected to extremely strong ground motions with long-period components. The yielding strength of the first story is assumed to fit that of mean values investigated by The Building Center of Japan for 29 damped buildings. From the research, the yield ratio α_{by} becomes 0.14 seconds for buildings with 20 stories. The assumed distribution of the story strength in the building is based on the Ai distribution specified in Japanese seismic code. Elastic story stiffness is ideally promotional to the story strength distribution. Assuming $h_b = 0.02$, $p_b = 0.5$, and $\mu = 0.05$ as a typical case, we chose D_{target} to be 3.0 which implies a ductility factor of the steel damper embedded in the frame.

As listed in Table 1, phase angle of four historical ground motions is selected to generate simulated ground motions fitted to an identical target response spectrum based on the Japanese Standard, together with random phase angle. All the ground motions have 120 seconds of significant duration. Amplitude of each input motions are gradually scaled to assume small, moderate, large, and extremely large earthquakes. In the latter results, a peak ductility factor D is used to measure the intensity of prescribed motions used instead. In other words, larger D imply greater earthquakes. This interpretation enables us to apply the results to other buildings with a wide variety of the strength.

Table 1 Selected earthquakes for generating simulated ground motions

No.	Earthquake	Station	PGA (cm/s^2)	
1	Imperial Valley 1940	El Centro	341.7	210.1
2	Kern Country 1952	Taft	152.7	175.9
3	Tokachi-oki 1968	Hachinohe	231.0	181.2
4	Miyagiken-oki 1978	Tohoku Univ.	258.2	202.6
5	Kushiro-oki 1993	JMA Kushiro	692.8	576.0

3.2 Peak response

Figure 3 shows the peak inter-story drift in the height direction for the employed shear-type model subjected to gradually magnified input motions. The markers in the graphs are the average of responses to the ten input motions. These results indicate seismic effectiveness beyond the elastic range.

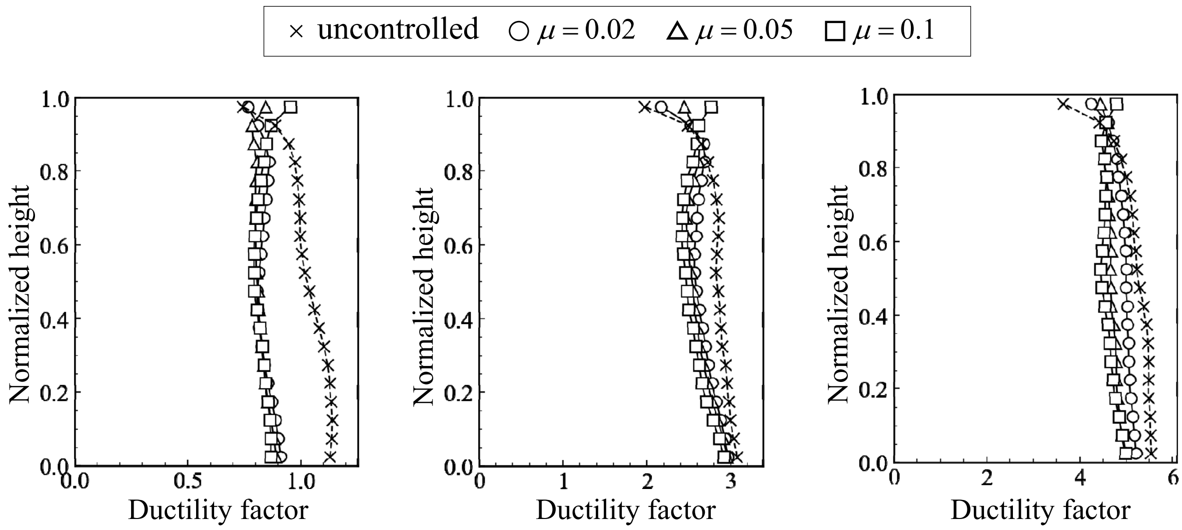
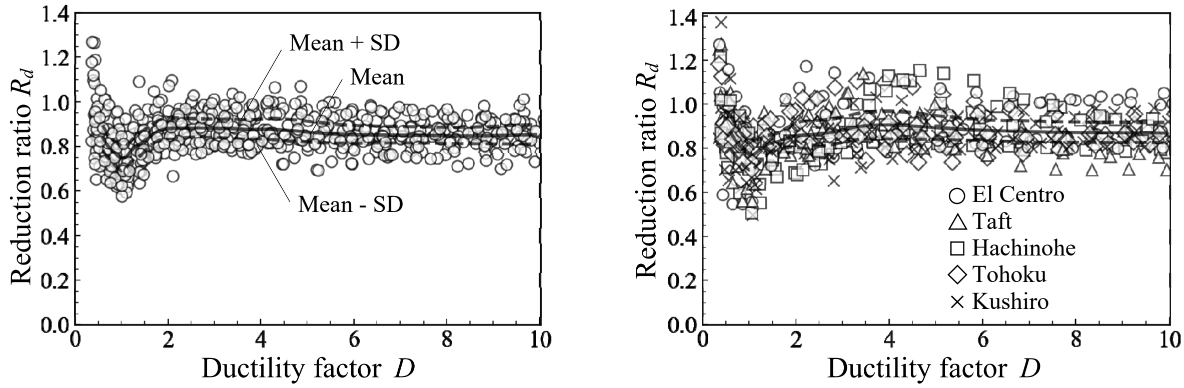
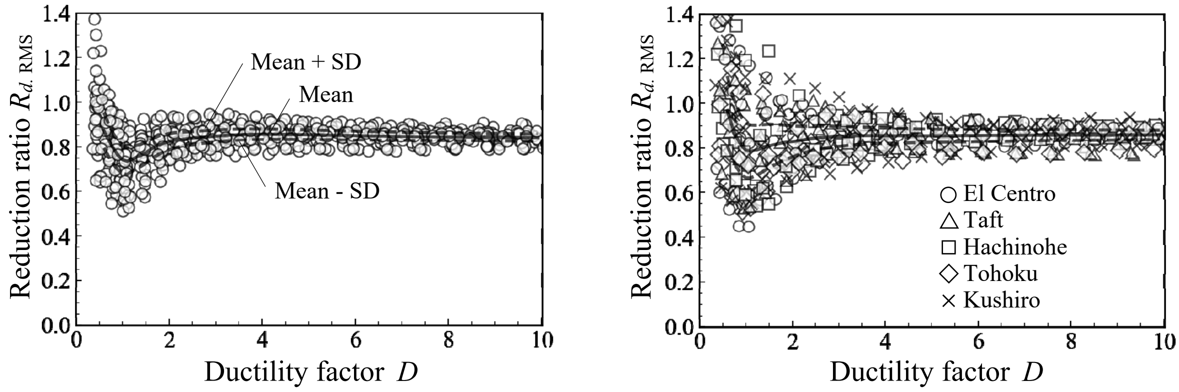


Fig. 3 Peak inter story drift angle (IDA) for random phase-based input motions



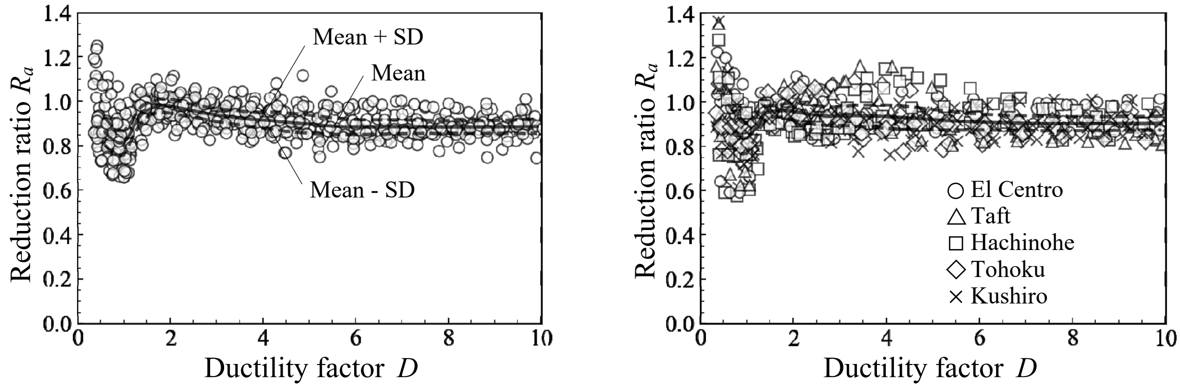
(a) Simulated ground motion with random phase angle (b) Observed earthquakes-based phase angle

Fig. 4 Seismic effectiveness in terms of peak displacement



(a) Simulated ground motion with random phase angle (b) Observed earthquakes-based phase angle

Fig. 5 Seismic effectiveness in terms of RMS displacement



(a) Simulated ground motion with random phase angle (b) Observed earthquakes-based phase angle

Fig. 6 Seismic effectiveness in terms of peak acceleration

3.3 Control performance indices

The previous results contain higher mode responses, which might not be significantly reduced by the TMD. Therefore, the controlled first mode response should be focused to assess the control performance accurately. For this purpose, we introduce indices in terms of peak and RMS displacement, and peak acceleration, defined by

$$R_d = \frac{\max_t |q(t)|}{\max_t |q_0(t)|}, \qquad R_{d,\mathrm{RMS}} = \sqrt{\frac{\sum_t q^2(t)}{\sum_t q_0^2(t)}}, \qquad R_a = \frac{\max_t |\ddot{q}(t)+\ddot{u}_g(t)|}{\max_t |\ddot{q}_0(t)+\ddot{u}_g(t)|} \tag{8a-c}$$

where q is the modal displacement controlled and $(\cdot)_0$ denotes values related to uncontrolled buildings. Quantity q is approximately extracted using the following formulae.

$$q(t) = \frac{1}{\beta_0}\frac{\boldsymbol{\phi}_0^T \mathbf{M}_0 \mathbf{u}(t)}{\boldsymbol{\phi}_0^T \mathbf{M}_0 \boldsymbol{\phi}_0}, \qquad q_0(t) = \frac{1}{\beta_0}\frac{\boldsymbol{\phi}_0^T \mathbf{M}_0 \mathbf{u}_0(t)}{\boldsymbol{\phi}_0^T \mathbf{M}_0 \boldsymbol{\phi}_0} \tag{9a, b}$$

where $\mathbf{M}_0$, $\boldsymbol{\phi}_0$, and β_0 are the mass matrix of a TMD-building system, modal vector, and the corresponding participation factor, respectively. $\mathbf{u}$ is the vector consisting of horizontal displacement at all the floors.

Figs. 4-6 shows the mean and standard deviation of calculated R_d, $R_{d,\mathrm{RMS}}$, and R_a with respect to the peak ductility factor D of the uncontrolled building. R_d approximately ranges from 0.8 to 0.9 in a wide range of D.

4. Prediction of peak response of controlled buildings using equivalent linearization and response spectrum analysis

4.1 Prediction procedure

We explain a method for predicting peak responses without time history analysis, which is useful to evaluate the statistical seismic performance. Predicted results are also useful to obtain the minimum required weight of TMD for given seismic demands. The framework comprises four necessary steps

a) Trial prediction of TMD's peak deformation

b) Equivalent damping ratios of the substructures (TMD and building)

c) Loss of supplemental damping to the TMD-building oscillation due to nonlinearities

d) Prediction of building response that is our interest

With iterative calculations, each value is evaluated in the following steps.

Let be i the number of iterations. Any computed values at step i are expressed by the notation $(\cdot)_{(i)}$. Firstly, given the ductility factor of an uncontrolled building D, we initially set $D_{b(i)} \leftarrow D$ and $D_{a(i)} \leftarrow 0$. These values allow us to compute the equivalent damping ratios h_b^p and h_a^p during the elastic-plastic response in the following [6].

$$h_{b(i)}^p = \frac{2}{\pi p_b D_{b(i)}} \ln \frac{1+p_b\left(D_{b(i)}-1\right)}{D_{b(i)}^{p_b}} + h_b \quad \text{for contolled building} \tag{10a}$$

$$h_{a(i)}^p = \frac{2}{\pi p_a D_{a(i)}} \ln \frac{1+p_a\left(D_{a(i)}-1\right)}{D_{a(i)}^{p_a}} \quad \text{for TMD} \tag{10b}$$

Secondly, the first choice of the supplemental damping ratio given to the TMD-building system takes the form of [7]

$$\Delta h^{eq*} = \frac{\sqrt{\bar{\mu}}}{4 + \bar{\mu}(12-\psi)/(2\psi)} \tag{11}$$

This damping ratio, however, is overestimated because the hysteretic TMD is not always perfectly tuned. The following formula can consider the difference between optimal damping ratio h_a^* and the actual one h_a^p in a reasonable way [7].

$$\Delta h_{0(i)}^{eq} = \frac{2\Lambda_{(i)}}{1+\Lambda_{(i)}^2}\Delta h^{eq*}, \quad \Lambda_{(i)} = \frac{h_{a(i)}^p}{h_a^*} \tag{12a, b}$$

This is analogous to a linear TMD with a dashpot, where more high damping is often added than optimum quantity to decrease TMD's deformation. For a linear TMD based on H_2 norm optimization, a value of h_a^* takes the form of [4]

$$h_a^* = \frac{1}{2}\sqrt{\frac{\bar{\mu}(1-\bar{\mu}/4)}{(1+\bar{\mu})(1-\bar{\mu}/2)}} \tag{13}$$

Furthermore, the supplemental damping expressed in Eq. (11a) should be decreased due to the development of nonlinearity h_b^p.

$$\Delta h_{(i)}^{eq} = \left\{1 - a(h_{b(i)}^p)^b\right\}\Delta h_{0(i)}^{eq} \tag{14}$$

where these coefficients are a of 2.9 and b of 0.86. Using equivalent linearization technique, we obtain the trial reduction factor R_d with the evaluated damping ratios $h_b^p, \Delta h^{eq}$ as follows [8].

$$R_{d(i)} = \sqrt{\frac{1+\alpha\left(h_b + h_b^p(D)\right)}{1+\alpha\left(h_b + h_{b(i)}^p(D_{b(i)}) + \Delta h_{(i)}^{eq}(D_{b(i)})\right)}} \tag{15}$$

where the constant α is assigned value of 25 for observed ground motions and 75 for simulated ground motions [8]. Notably, the computed R_d depends on the inaccurate (or trial) damping ratios $h_b^p, \Delta h^{eq}$ to be updated. For the controlled building, the update procedure is as follows.

$$D_{b(i+1)} \leftarrow R_{d(i)} D \tag{16}$$

Then, the deformation ratio χ becomes

$$\chi_{(i)} = \sqrt{\frac{2}{1+\Lambda_{(i)}^2}}\chi_0^* \tag{17}$$

We can also update the ductility factor of the TMD D_a using

$$D_{a(i+1)} \leftarrow \frac{\psi\gamma_0^{*2}\chi_{(i)}}{\bar{\alpha}_y^*} D_{b(i+1)} \tag{18}$$

If $D_{b(i)}$ meets the following convergence criterion, we obtain all the computed values as final results.

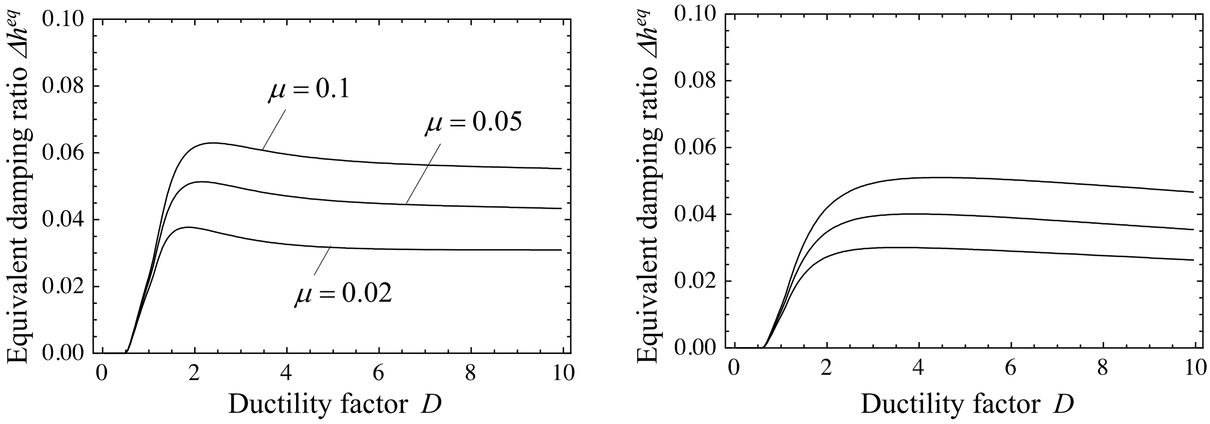
$$\frac{|D_{b(i+1)} - D_{b(i)}|}{D_{b(i)}} \le \text{tolerance} \qquad (19)$$

Otherwise, we back to Eq. (10), moving to the next step $i \leftarrow i+1$. The peak inter-story drift can be obtained with the converged D_b.

4.2 Comparison between predicted and time history analysis-based responses

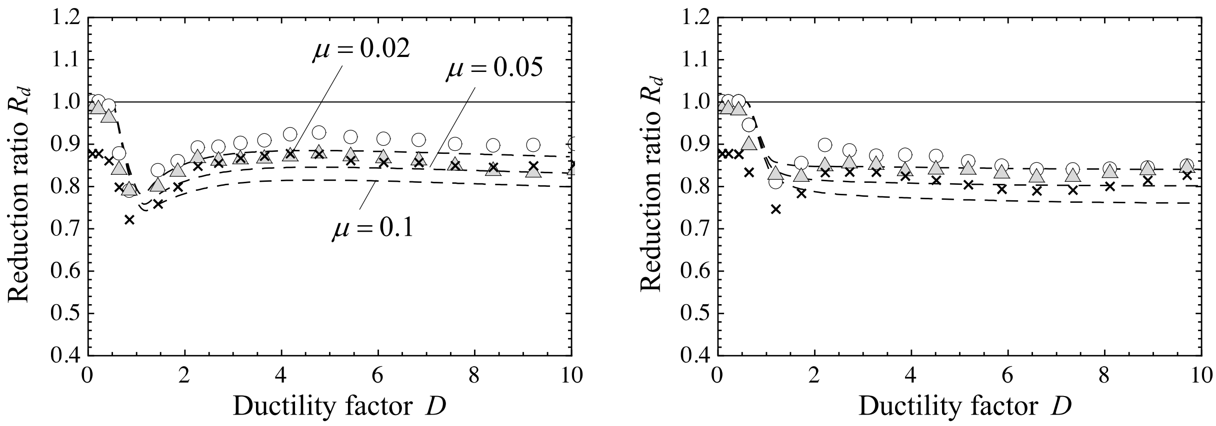
Numerical examples are demonstrated using the proposed prediction method. Without loss of generality, we assume that the ductility factors D in uncontrolled buildings are given to distinguish errors arising from response spectrum method. In other words, R_d is employed to measure predicted errors instead. Moreover, the following examples consider only simulated ground motions because the expected R_d is almost the same for any type of disturbances, as shown in Chapter 3.

As shown in Fig. 8, the equivalent damping ratio Δh^{eq} is almost the same for D larger than D_{target}, which was assumed to be 3.0 here. With the calculated Δh^{eq}, Fig. 9 explains that the proposed method predicts well for a wide variety of the mass ratio μ and stiffness ratio p_b.



(a) $p_b = 0.5$ (b) $p_b = 0.7$

Fig. 8 Variation of calculated supplemental damping ratio



(a) $p_b = 0.5$ (b) $p_b = 0.7$

Fig. 9 Prediction of time history-based response

In this study, we have focused on the design parameters regarding hysteretic TMDs. The proposed prediction method allows us to estimate the required mass of TMD for a given seismic criterion.

5. Conclusion

This study has revealed the feasibility of a hysteretic TMDs in terms of adaptive tuning for a nonlinear controlled building.

Although only bilinear hysteresis is assumed in this preliminary study, the basic concept can be easily applied to multi-linear type, including trilinear, which can consider gradual yielding. In addition, the author's future work is to develop an actual device to realize the passive control system proposed in this study.

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