SEISMIC DESIGN OF STRUCTURE WITH IMPROVED TOGGLE-BRACE-DAMPER SYSTEM

Li Bo[1] and Liang Xingwen[2]

[1]*Dept. of Civil Engineering, Chang'an University, Xi'an, China*
[2]*Dept. of Civil Engineering, Xi'an University of Architecture and Technology Xi'an, China*
Email: lblibolb@sina.com, liangxingwen2000@yahoo.com.cn

ABSTRACT:

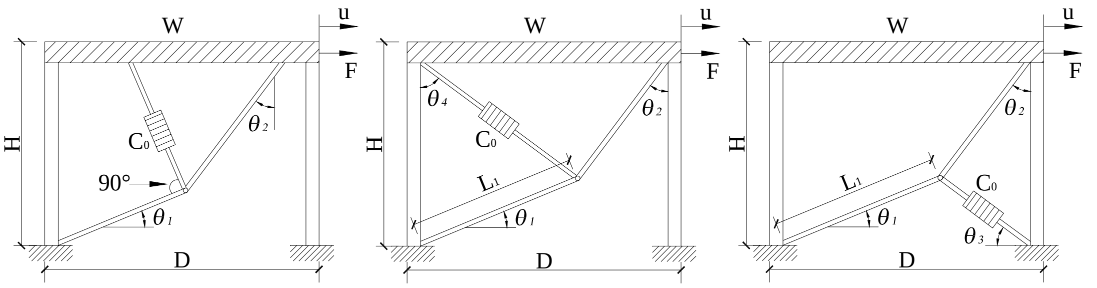
An enhanced toggle-brace-damper (TBD) installation configuration with the damper directly connected to the beam-column joint is introduced. Magnification factor for damper displacement and force are investigated. Expression for prediction of equivalent linear viscous damping ratio provided by the improved TBD systems installed in a building is then introduced. For practical application, a simplified procedure for design and analysis of building incorporating TBD systems is proposed based on Chinese design response spectrum, in which the damping coefficients of the dampers that are proportional to the relative modal displacements in the corresponding stories are distributed over the height of a structure to satisfy the given performance objective. The validity of this method is verified by nonlinear response history analysis (RHA). The study shows that this improved damper installation configuration not only decreases significantly floor displacement response but reduces story shear demands when compared with the conventional installation configurations of TBD and diagonal brace-damper.

KEYWORDS: Energy dissipation system, Equivalent damping ratio, Response spectrum analysis, Nonlinear response history analysis

1. INTRODUCTION

A structure with energy dissipation systems relies mostly on the energy dissipation devices to consume the input seismic energy so that the damage to the main structure is reduced or eliminated (Constantinou *et al.*, 1998). Currently, viscous liquid dampers are widely used one type of energy dissipation devices in the new and existing buildings (Soong *et al.*, 2002; Constantinou *et al.*, 1992; Syman *et al.*, 1998). Although installation configurations of dampers in diagonal brace and chevron brace provide convenience for construction, this may lead to the devices axial displacements less than or equal to the story drift, thus lowering their efficiency of energy dissipation (Constantinou *et al.*, 2001). The problem mentioned above could be addressed by taking the following measures. One is to select the large size damper and the other to amplify the displacement and velocity in damper. However the former will result in the increase in cost for rehabilitation of a building, so the latter may be an economic and efficient option for damper installation configuration.

The toggle-brace-damper system can result in device displacement that is larger than the structural drift, as shown in Figure 1 (a) (Constantinou *et al*., 1997). However, some questions have been raised on the conventional TBD system as follows. In the TBD system the damper force will be directly applied to the floor beam and may significantly affect the design of the floor beam. And the displacement amplification factor may be smaller than what is expected due to the flexibility of the beam. Therefore, the effective damping ratio contributed by the damper could be smaller than the expected design value (Hwang *et al*., 2005). To overcome the insufficiency inherent in this damper installation configuration, Taylor proposed two improved TBD systems, as shown in Figure 1 (b) and (c), in which the damper and brace elements are connected directly to the beam-column joints.



(a) conventional configuration (b) improved configuration for upper damper (c) improved configuration for lower damper

Fig. 1 Installation configurations of TBD systems

Experimental and analytical investigations on the improved TBD system have been carried out by many researchers and their advantage over the traditional one was also validated (Hwang *et al.*, 2005). However, the practical procedure for use in design of this improved TBD system remains largely unexplored. For practical application, this paper presents a simplified procedure for design and analysis of building incorporating the improved TBD systems based on Chinese design response spectrum. The validity of the method was verified by the nonlinear RHA of the structure with TBD systems designed according to the presented method. Furthermore, RHA of buildings installed with improved upper TBD, lower TBD, and the diagonal brace dampers were performed to compare the efficiency of vibration control for different configurations of energy dissipation devices.

2. EQUIVALENT DAMPING RATIO FOR STRUCTURE WITH TBD SYSTEMS

It has been shown that the following relationships exists for various configurations of the damper (Constantinou *et al*., 2001)

$$u_D = fu \qquad F = fF_D = f^2 C_0 \dot{u} \tag{2.1}$$

where u_D is the relative displacement along the axis of damper; u and $\dot{u}$ are respectively the interstory drift and interstory velocity; F_D is the damper force; C_0 is the damping coefficient of the damper; F is the horizontal component of the force exerted by the damper on the frame; and f is the magnification factor. For the configurations of lower and upper

TBD, the magnification factors are derived based on the assumption of small deformation and axially rigid brace (Hwang *et al*., 2005), as shown in Eq. (2.2) for lower TBD and Eq. (2.3) for upper TBD, where θ_1 to θ_4 are given in Figure 1.

$$f_L = \frac{\sin\theta_2 \sin(\theta_1+\theta_3)}{\cos(\theta_1+\theta_2)} \tag{2.2}$$

$$f_U = \frac{\sin\theta_2}{\cos(\theta_1+\theta_2)}\cos(\theta_4-\theta_1)+\sin\theta_4 \tag{2.3}$$

The equivalent damping ratio of a structure with TBD system can be derived following the same procedure as that for a structure with diagonal brace damper system (Constantinou *et al*., 2001).

$$\zeta_d = \frac{T\sum_j C_j \phi_{rj}^2 f_j^2}{4\pi \sum_i m_i \phi_i^2} \tag{2.4}$$

where ζ_d is the equivalent damping ratio contributed by the TBD systems; T is the first period of vibration in the direction under consideration; C_j is the damping coefficient of damper j; f_j is the magnification factor of damper; m_i is the seismic mass lumped at floor level i; ϕ_{rj} is the relative modal horizontal displacement of damper j of the first mode of vibration; and ϕ_i is the first modal displacement at floor i.

It may be more reasonable to distribute the damping ratio over the height of the structure such that the sum of damping coefficients of total dampers at floor i is proportional to corresponding first modal relative displacement along the axis of the damper, $f_i\phi_{ri}$, than uniformly distribute the damping ratio over the height of the structure. So this paper presents the following expression for distributing the required damping ratio throughout a structure.

$$C_j = \frac{4\pi\zeta_d \phi_{rj} f_j \sum_{i=1}^{n} m_i \phi_i^2}{T\sum_{j=1}^{n} \phi_{rj}^3 f_j^3} \tag{2.5}$$

After the parameter, *H/D*, is assigned by the architect the relationship between the geometric layout of TBD system and the magnification factor *f* can then be established employing the following steps (Hwang *et al.*, 2005). (1) determine the range of θ_1 that satisfies the constraint of $\theta_1 \le \tan^{-1}(H/D)$; (2) identify the range of L_1/D less than $1/\cos\theta_1$; (3) determine the feasible range of combination of θ_1 and L_1/D for a specified *u*/*H*; (4) establish the relationship between the magnification *f* and the inclined angle θ_1 for different values of L_1/D.

3. DESIGN PROCEDURE FOR STRUCTURE WITH TBD SYSTEMS

Design procedures for structure with TBD systems are not provided in modern seismic codes. Based on the Chinese design response spectrum (2002), a simplified procedure for seismic design and analysis of structure with the improved TBD systems is presented in this paper to determine the effective damping ratio that limit the roof displacement of the damped structure to the specified value. The detailed steps are as follows.

(1) Specify the limiting value of interstory drift, [Δ], for a structure with the improved TBD systems for the given the earthquake ground motion level.

(2) Determine the velocity exponent range for viscous damper from [0.35 1] (Asher *et al.*, 1996).

(3) Evaluate the total effective damping ratio, $\zeta_{eff}^{(i)}$, according to the assumed the supplemental equivalent viscous damping ratio, $\zeta_{d}^{(i)}$.

(4) Calculate the roof displacement based on China Code for Seismic Design of Buildings as follows, $D_r^{(i)} = \Gamma_1 S_d(T_1, \zeta_{eff}^{(i)})$, where Γ_1 is participation factor of the first mode shape normalized to unit value at roof level, and S_d is the spectra displacement depending on the fundamental natural period, T_1, and the total effective damping ratio, $\zeta_{eff}^{(i)}$, assumed in the step (3).

(5) Compute the maximum interstory drift expressed as, $\Delta_{\max}^{(i)} = D_r \phi_{ri,\max}$, where $\phi_{ri,\max}$ is the maximum modal interstory drift.

(6) If the estimated maximum interstory drift, $\Delta_{\max}^{(i)}$ obtained from step (5) is less than the limit one, [Δ], go to the next step. Otherwise, let $\zeta_{eff}^{(i+1)} = (\Delta_{\max}^{(i)} / [\Delta]) \cdot \zeta_{eff}^{(i)}$ and repeat step (3)-step (6) until the estimated value is less than or equal to the specified drift limit.

(7) Determine the damping coefficient of damper distributed over the height of the structure according to the Eq. (2.5).

(8) Check the actions for components of the building using static method of analysis at the stage of maximum displacement, maximum velocity and maximum acceleration, and the worst case should be used for design.

4. ILLUSTRUTIVE EXAMPLE

In this section, the illustrative example is presented to design and analysis of a building with improved TBD systems using the proposed method. The response quantities of the damped structure are determined using the simplified method of analysis recommended by FEMA-273 document (1997). In order to verify the validity of the proposed method, the response quantities of interest of the damped structure obtained by proposed method are compared with the mean results obtained by nonlinear RHA of the designed structure under the artificial ground motions the match with the design response spectrum. Finally, nonlinear RHA of buildings installed with improved upper TBD, lower TBD, and the diagonal brace dampers were performed to compare the efficiency of vibration control for different configurations of energy dissipation devices.

4.1. General Information on Structure

A three-story, three-span steel moment resisting frame is used to illustrate the design procedure. The structural geometry of the frame is given in Figure 2, in which the sections designed for beams and columns and the seismic weight are denoted. The yielding strength, f_y, of 345MPa is used for all steel members. After eigenvalue analysis, the first modal period, T_1, the first mode shape, $\{\phi\}_1^{\mathrm{T}}$, and the first modal participation factor of Γ_1, of the structure are respectively 0.756 sec, [1.000, 0.675, 0.250] and 1.399.

Linear FVD will be installed in the improved toggle brace configuration in interior span of the frame. The inherent damping ratio of the structure is assumed to be 2%, and the supplemental damping ratio resulting from all added dampers is expected to reach 18%. Some necessary parameters of the structure for designing damping system are listed in Table 1.

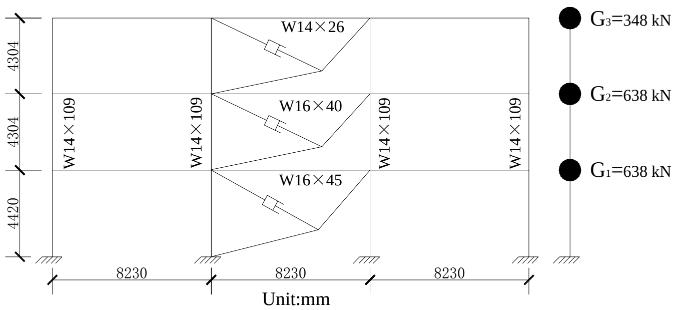


Fig. 2 Elevation of building with improved TBD systems

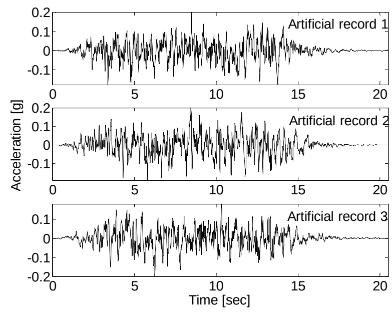


Fig. 3 Artificial ground motions compatible with design response spectrum

Table 1 Design parameters for improved TBD and diagonal-brace-damper systems

	Geometric parameters of TBD system						Magnification factor		
							Upper brace	Lower brace	Diagonal brace
Story level	H/D	L_1/D	θ_1 (rad)	θ_2 (rad)	θ_3 (rad)	θ_4 (rad)	f_U	f_L	f_D
2, 3	0.523	0.8	$\pi/10$	0.714	0.802	1.223	1.720	1.141	0.89
1	0.537	0.8	$\pi/10$	0.690	0.802	1.207	1.678	1.065	0.88

Velocity exponent of 1.0 is assumed for all dampers in the preliminary design. The characteristic period, T_g, of the site where the structure is located is 0.4 second. A performance acceptance criteria is specified for the structure with supplemental damping systems that the maximum interstory drift of damped frame subjected to a moderate earthquake with a peak acceleration, α_{max}, of 0.45 g, where g is gravity acceleration, is not greater than the elastic interstory drift limit of 1/300, prescribed in Chinese seismic design code [7], for steel moment frame under the action of a minor earthquake.

Since the effective damping ratio has been raised to 20%, the decay exponent is calculated as $\gamma = 0.9 + (0.05 - \zeta_{eff})/0.5 + 5\zeta_{eff} = 0.8$, and the damping modification factor is $\eta_2 = 1 + (0.05 - \zeta_{eff})/(0.06 + 1.7\zeta_{eff}) = 0.625$. The response of the damped frame to a moderate earthquake remains elastic and is assumed to be dominated by the fundamental mode of shape. So roof displacement amplitude, D_r, is

$$D_r = \Gamma_1 S_{d1} = \Gamma_1 \frac{S_{a1}}{\omega_1^2} = \Gamma_1 \frac{T_1^2}{4\pi^2}\left(\frac{T_g}{T_1}\right)^{\gamma} \eta_2 \alpha_{\max} g = 34 \text{ mm}$$

The maximum interstory drift, Δ_{max}, is $\Delta_{\max} = D_r \max\left\{\phi_{ri,1}\right\} = 34 \times 0.407 = 13.8$ mm , which is less than the specified value, $[\Delta] = h[\theta_e] = 4304/300 = 14.3$ mm .The assumed supplemental damping satisfies the given performance acceptance criteria, so no further iterations are required.

The damping coefficient of the damper at the third, second and the first story determined using Eq. (2.5) are respectively C_3 =0.213 kN·s/mm , C_2 =0.253 kN·s/mm , and C_1 =0.152 kN·s/mm .

4.2. Evaluation of Response of Damped Structure to Moderate Earthquake Excitation

Table 2 Summary of results obtained by simplified method of analysis

	Stage at maximum displacement			Stage at maximum velocity		
Story level	Lateral force (kN)	Story shear (kN)	Floor displacement (mm)	Damper axial force (kN)	Story shear (kN)	Lateral force (kN)
3	82.3	82.3	34	35.5	61.1	61.1
2	99.1	184.1	22.3	50.0	86.0	24.9
1	37.7	219.1	8.5	17.5	29.4	-56.6

The design lateral forces, F_{i1}, applied to the structure, at the stage of the maximum displacement is $\gamma_1\phi_{i1}(T_g/T)^{\gamma}\eta_2\alpha_{\max}G_i$. The peak damper force, F_{di1}, at the stage of the maximum velocity is $C_i(2\pi/T_1)D_r\phi_{ri}f_{ui}$. And the horizontal component, V_{di1}, of damper force exerted on the frame is $F_{di1}f_{ui}$, which could be balanced by the lateral force applied at each floor. The responses of the damped structure at the stage of maximum acceleration may be obtained by the load combination formula recommended in FEMA-273 (1997), i.e., $Q_{\max.\text{acc.}} = CF_1 \cdot Q_{\max.\text{disp.}} + CF_2 \cdot Q_{\max.\text{velo.}}$, where $Q_{\max.\text{acc.}}$, $Q_{\max.\text{disp.}}$ and $Q_{\max.\text{velo.}}$ are the response quantity of the damped structure at the stage of the maximum acceleration, the maximum displacement, and the maximum velocity, respectively, and $CF_1 = \cos[\tan^{-1}(2\zeta_{eff})]$, $CF_2 = \sin[\tan^{-1}(2\zeta_{eff})]$. The results obtained by the simplified method of analysis are shown in Table 2.

4.3. Comparison of Results from Simplified Method of Analysis and Time History Analysis

Three artificial acceleration time histories, as shown in Figure 3, compatible with the Chinese design response spectrum, as shown in Figure 4, were generated using the software SIMQKE (Vanmarcke and Gasparini, 1976), and were employed to analyze the designed structure with upper TBD systems to verify the validity of the simplified method. RHA results show that the means of the maximum floor displacements at the first to the third story are, respectively, smaller by 14%, 12% and 5% than those obtained by simplified method of analysis, as shown in Table 3. For the maximum interstory drift, the mean results of RHA at the first and the second story are, respectively, 14% and 9% smaller than those of simplified method of analysis, and on the contrary at the third story the mean maximum interstory drift obtained from RHA is greater than that from simplified method of analysis by 6%, as shown in Figure 5. Therefore, proposed method may well predict the floor displacements for structure with upper TBD systems, particularly at the roof level.

The damper force demands estimated by the presented method correspond to 6%, 4% and 18% smaller than those obtained from nonlinear RHA through the first to third story, respectively. Thus the proposed procedure underestimates the damper force demands at all stories, particularly significantly at the top story, which may be due to the fact that contribution of higher modes to the maximum velocity response at the top story may be significant and possibly underestimated by the proposed method.

Story shears estimated by the presented method are 3% greater than those obtained from nonlinear RHA both at first and second story. However, the story shear at the top story is 8% smaller for the former than for the latter. Similarly, this may be due to the fact that higher modes effects may not be taken into account at the top story.

4.4. Comparison of Seismic Response for Different Installation Schemes of Viscous Damper

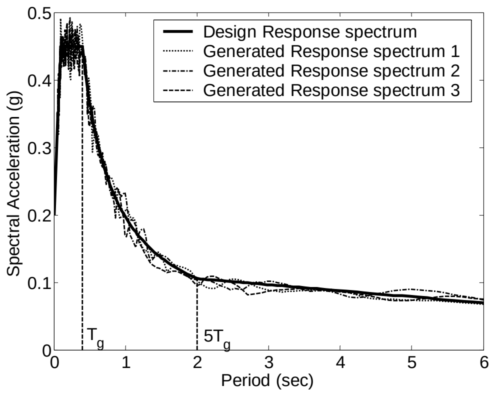


Fig. 4 Earthquake response spectra generated by ground motions compatible with design spectrum

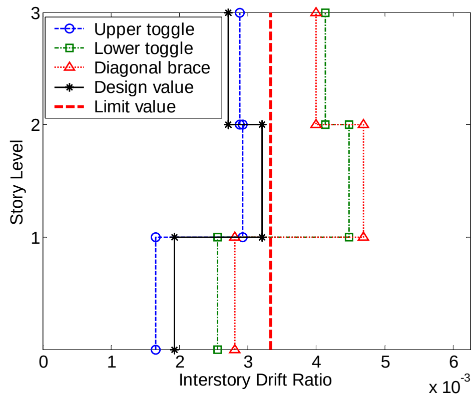


Fig. 5 Mean maximum interstory drift ratios of damped structure with different installation schemes of damper

The mean floor displacements for the structure with diagonal-brace-damper systems are not significantly larger than those with lower TBD systems, and, however, corresponds 1.70, 1.65 and 1.54 times those through first to the third story level for structures with upper TBD systems, as shown in Table 3. This may be due to the fact that the values of magnification factors shown in Table 1 for the structure with diagonal-brace-damper systems are close to those for the structure with lower-toggle-brace damper systems and much lower than those for structure with upper TBD systems.

Table 3 Comparison of response quantities obtained by simplified method of analysis and nonlinear RHA

Seismic excitation	Installation scheme of viscous damper	Floor displacement （mm）			Damper force（kN）			Story shear（kN)		
		First level	Second level	Top level	First level	Second level	Top level	First level	Second level	Top level
mean maximum value of THA	Upper toggle	7.3	19.9	32.3	33.2	52.1	20.6	208.1	193.8	107.1
	Lower toggle	11.3	30.6	48.4	27.7	45.9	20.0	273.3	252.5	146.0
	Diagonal brace	12.4	32.6	49.8	26.5	38.8	19.2	321.0	269.5	163.1
Simplified analysis	Upper toggle	8.5	22.3	34	35.5	50.0	17.5	214.2	200.2	99.0

For damper force demand there is little difference among three installation schemes for viscous damper. As shown in Table 3, although mean interstory drift at the top story for structure with upper TBD systems is respectively 33% and 35% smaller than that for structure with lower TBD systems and structure with diagonal-brace-damper systems, the damper force demand for the former is increased only by 3% and 7%, respectively, when compared with that for the other two installation schemes of damper.

Upper TBD installation scheme may substantially reduce the story shear demand, when compared against the other two installation schemes of damper. For instance, the mean maximum shear demand for structure with lower TBD systems corresponds to 1.31, 1.30 and 1.36 times that for structure with upper TBD systems through the first to the third story, as shown in Table 3.

5. CONCLUSIONS

The simplified design and analysis procedure is presented in this paper based on the design response spectrum provided in Chinese seismic code. Nonlinear RHA were performed on structures with various installation schemes of viscous damper designed in accordance with the proposed method using artificial acceleration time histories compatible with the design response spectrum. Conclusions applicable to the low-to-medium rise building with TBD systems are drawn from analyses results as follows;

(1) For the structure with improved upper TBD systems the proposed method may well predict roof displacement, and slightly underestimate the damper force and story shear demand, particularly at the top story level.

(2) The installation configuration for improved upper TBD is found to more effectively reduce the floor displacements and story shears demands than that for improved lower TBD and diagonal brace damper.

(3) The proposed method, which is based on the philosophy of displacement-based seismic design, is simple yet effective for use in rapid design and analysis of structures with TBD systems, particularly at the stage of preliminary design.

REFERENCES

Asher, J.W., Young, R.P. and Ewing, R.D. (1996). Seismic isolation design of the San Bernardino county medical center replacement project. *Journal of Structural Design of Tall Buildings*, **5:4**, 265-279.

Code for Seismic Design of Buildings. (2002). China Architecture and Buildings Press, Bei Jing.

Constantinou M.C., Soong T.T. and Dargush G.F. (1998). Passive energy dissipation systems for structural design and retrofit. Multidisciplinary Center for Earthquake Engineering Research, U.S.A.

Constantinou M.C. and Symans M.D. (1992). Experimental and analytical investigation of seismic response of structures with supplemental fluid viscous dampers. NCEER-92-0032, Buffalo: State University of New York.

Constantinou M.C., Tsopelas P. and Hammel W. (1997). Testing and Modeling of an Improved Damper Configuration for Stiff Structural Systems. Technical Report Submitted to the Center for Industrial Effectiveness, Buffalo, New York.

Constantinou, M.C., Tsopelas, P., Hammel, W., and Sigaher, A.N. (2001). Toggle-brace-damper seismic energy dissipation systems. *Journal of Structural Engineering*, **127:2**, 105–112.

FEMA 273, NEHRP guidelines for the seismic rehabilitation of buildings. (1997). Federal Emergency Management Agency, Washington, DC.

Hwang, J.S., Huang, Y.N. and Hung, Y.H. (2005). Analytical and Experimental Study of TBD Systems. *Journal of Structural Engineering*, **127:2**, 105–112.

Soong T.T. and Spencer B.F. (2002). Supplemental energy dissipation: state-of-the-art and state-of-the practice. *Journal of Engineering Structures*, **24:3**, 243–59.

Syman M.D. and Constantinou M.C. (1998). Passive fluid viscous damping systems for seismic energy dissipation. *Journal of Earthquake Technology*, **35:4**, 185-206.

Vanmarcke, E.H. and Gasparini, D.A. (1976). A Program for Artificial Motion Generation, User's Manual and Documentation. Department of Civil Engineering, Massachusetts Institute of Technology.