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Experimental study of RC building structures with supplemental viscous dampers and lightly reinforced walls

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Abstract

The paper presents the results of an experimental evaluation on the effectiveness of applying viscous dampers to reinforced concrete moment-resisting building structures. The unique feature of these moment-resisting concrete building structures, as is common practice in Taiwan, is that lightly reinforced concrete exterior walls and interior partition walls are provided in construction but not considered for their contribution of stiffness and strength in the design process. With these additional walls it is suspected that, with the small relative story displacement and velocity, the effectiveness of supplemental dampers will be very limited. However, the test results show that an efficient installation mechanism, the toggle-brace-damper system, is effective even with a small relative story drift in the seismic response control of the structure. In addition, on contrast to the usually assumed behavior, the slender wall system subjected to lateral seismic force reveals a double-curvature behavior in each story rather than a cantilever behavior as a whole. Furthermore, for energy consideration, the "momentary input energy method" is found to be more rational than the "absolute input energy method" to evaluate the damage potential to structures and to demonstrate the effectiveness of supplemental viscous dampers to structures.

Keywords: Viscous dampers; Reinforced concrete; Toggle brace; Seismic response; Shaking table test; Passive control

1. Introduction

Reinforced concrete moment resisting structures according to their definition are normally without reinforced concrete walls. However, it is the common practice in Taiwan that a great amount of reinforced concrete moment-resisting buildings are constructed with 15 cm thick lightly reinforced concrete exterior walls together with 12 cm thick lightly reinforced interior partition walls. These reinforced concrete walls are normally provided with the minimum reinforcement required by the American Concrete Institute (ACI) building code [1], and are not considered for their contribution of stiffness and strength to the structure during the design stage. As a consequence, these reinforced concrete moment-resisting structures are neither a pure moment resisting frame nor a dual system. Therefore, no clear design guidelines are readily available for the seismic design of this type of structure in the current seismic design code [2–4]. However, it is not the function of this study to investigate the inelastic seismic behavior of these structures or to possibly develop rational response modification factors for these structures. Instead, it is the goal to investigate the effectiveness of incorporating supplemental dampers into these structures. For incorporating structural dampers, it is recognized that with the additional exterior and interior lightly reinforced concrete walls the effectiveness of adding yielding type dampers will be very limited. This is because the story drift is greatly reduced by the additional walls such that the yielding type dampers can hardly become engaged into the energy dissipation under such a small story drift. Therefore, velocity type dampers, such as viscous

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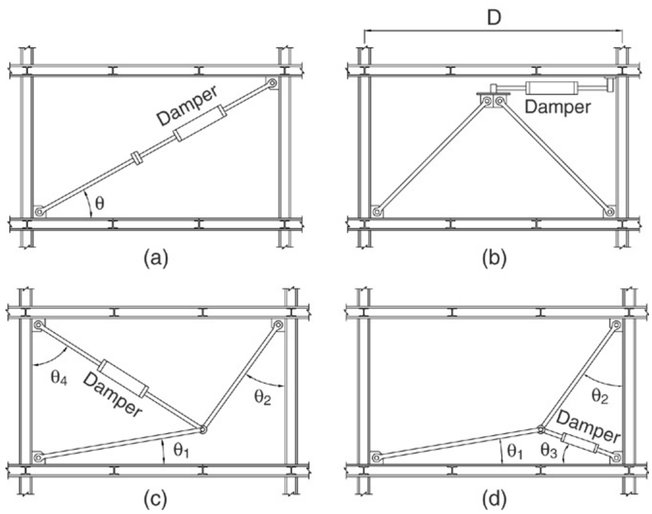

Fig. 1. Commonly used configurations for installation of viscous dampers: (a) diagonal-brace damper; (b) K-brace-damper; (c) upper toggle-brace-damper; (d) lower toggle-brace-damper.

dampers, rather than yielding type dampers have been applied to this type of structure in Taiwan. However, with the small story drift, it is still not sufficiently clear how effectively the supplemental viscous dampers will benefit this type of structure for seismic protection. It is therefore one of the purposes of this study to experimentally investigate the effectiveness of a viscous damper installation scheme in reducing the seismic responses of this type of structure.

For the installation of viscous dampers, there exist four commonly used configurations, the diagonal-brace damper, K-brace-damper, upper toggle-brace-damper and lower toggle-brace-damper, as shown in Fig. 1 [5]. Among these installation configurations, the upper toggle-brace-damper system [6,7] possesses the largest amplification factor for the axial damper displacement corresponding to a lateral story drift of the structure. Since it can be expected that the structure with “additional” lightly reinforced concrete walls will yield a relatively smaller lateral story drift, it is therefore decided in this study to adopt the upper toggle-brace-damper system as the energy dissipation system for a scaled-down three story reinforced concrete model. Shaking table tests are conducted to investigate the effectiveness of the toggle-brace-damper system by comparing the seismic responses and damage patterns of two test structures respectively with and without upper toggle-brace-dampers. In addition, the input energies are determined based on the absolute input energy [8] and the momentary input energy [9,10] respectively. The results are used to justify the rationality of the two energy methods in evaluating the damage potential of earthquake ground motions to structures.

2. Design formulas for structures with nonlinear toggle-brace-dampers

The force–velocity relationship of a viscous damper is described by

$$F_d = C_d|\dot{u}|^{\alpha}\,\mathrm{sgn}\,(\dot{u}) \tag{1}$$

where F_d is the damper force, C_d is the damping coefficient, $\dot{u}$ is the relative velocity between the two ends of the damper; α is the damping exponent; and $\mathrm{sgn}(\dot{u}) = 1$ when $\dot{u} \geq 0$ and $\mathrm{sgn}(\dot{u}) = -1$ when $\dot{u} < 0$. The damper with $\alpha = 1$ is called a linear viscous damper while the damper with α smaller than 1 is called a nonlinear viscous damper.

Following the concept of equivalent damping ratio provided by Federal Emergency Management Agency [11,12] and the formulations given in research reports [5,13], the composite damping ratio of a building structure with supplemental viscous dampers is determined based on the fundamental dynamic properties of the first vibration mode in the form of

$$\xi_{\text{eff}} = \xi_0 + \xi_d = \xi_0 + \frac{\sum_j C_j \lambda_j T^{2-\alpha_j} (f_j \phi_{rj})^{1+\alpha_j} A^{\alpha_j - 1}}{(2\pi)^{3-\alpha_j} \sum_i m_i \phi_i^2} \tag{2}$$

where ξ_{eff} = the composite damping ratio of the structure; ξ_0 = the inherent viscous damping ratio of the structure; ξ_d = the damping ratio contributed by the viscous dampers to the structure; C_j = the damping coefficient of damper j; α_j = the damping exponent of damper j; T = the natural period of the first vibration mode; ϕ_i = the normalized modal displacement at the ith story corresponding to the first vibration mode shape (the roof displacement is normalized to a unit value); A = the relative displacement of the roof to the ground; ϕ_{rj} = the relative modal displacement between the ends of damper j in the horizontal direction corresponding to the first vibration mode shape; f_j = the magnification factor of the axial deformation of damper j to the corresponding horizontal story drift; m_i = the mass of the ith story. In practical applications, the same damping exponent is usually designed for all dampers to be installed to a building structure. The parameter λ_j is then calculated by

$$\lambda_j = 2^{2+\alpha_j} \frac{\Gamma^2\left(1 + \frac{\alpha_j}{2}\right)}{\Gamma(2 + \alpha_j)} \tag{3}$$

where Γ is the gamma function. The magnification factors of Eq. (2) are defined by

$$u_d = fu \tag{4}$$

where u_d = axial deformation of the damper and u = the horizontal story drift; the magnification factors corresponding to the lower and upper toggle-brace-damper systems shown in Fig. 1 are respectively derived as [7]

$$f_L = \frac{\sin\theta_2 \sin(\theta_1 + \theta_3)}{\cos(\theta_1 + \theta_2)} \tag{5}$$

and

$$f_U = \frac{\sin\theta_2}{\cos(\theta_1 + \theta_2)} \cos(\theta_4 - \theta_1) + \sin\theta_4. \tag{6}$$

Based on Eqs. (5) and (6), it can be easily understood that if an appropriate geometric layout is selected the magnification factors of toggle-brace-damper systems can be much larger than the diagonal-brace-damper and K-brace-damper systems.

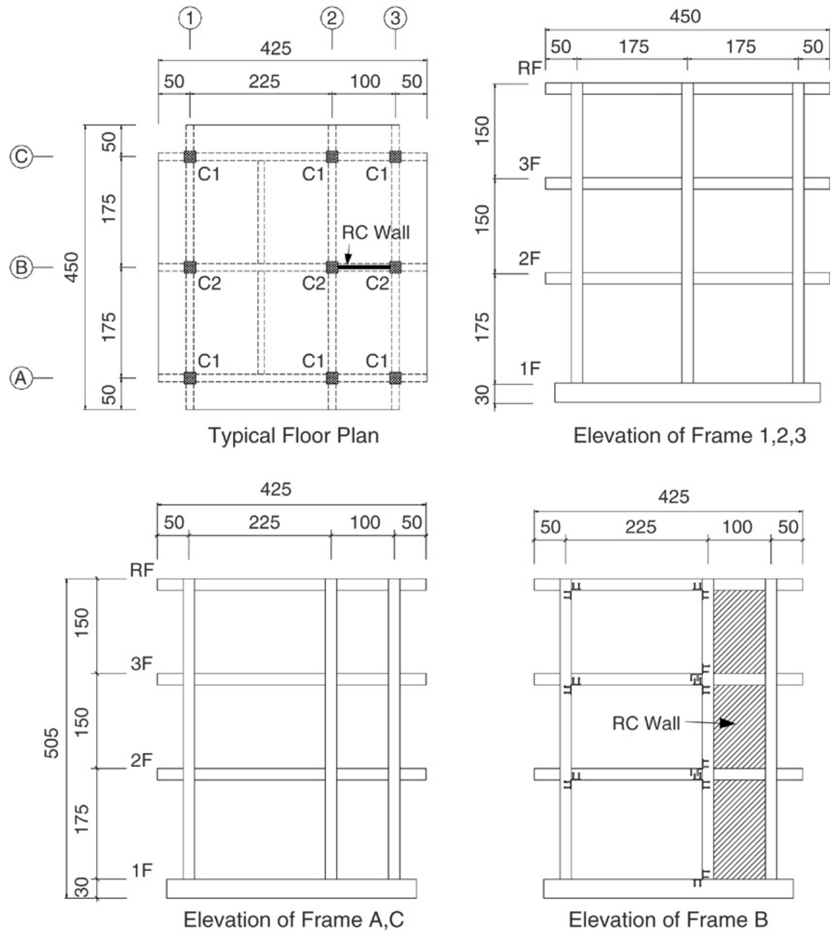

Fig. 2. Plan and elevation of test schedule with lightly reinforced concrete wall (unit: cm).

In addition, it has been demonstrated that the upper toggle-brace-damper system is more efficient than the lower toggle-brace-damper system in amplifying the axial deformation of the damper corresponding to a prescribed story drift [7]. Therefore, the upper toggle-brace-damper system is adopted in this study as the installation scheme of the dampers to the test structure such that the effectiveness of the dampers can be possibly enhanced considering the small story drift of the test structure with a lightly reinforced concrete wall.

3. Test structures

Two identical test structures were fabricated for shaking table tests. One was equipped with the upper toggle-brace-damper while the other was without the dampers. The test structure in the direction of shaking (X direction) is composed of three parallel three-story reinforced concrete frames, in which two exterior frames are moment resisting frames and one interior frame is a moment resisting frame with a lightly reinforced concrete wall in one of the two bays, as shown in Fig. 2. The elevation and plan dimensions of the test frame are also given in Fig. 2. The test structures are assumed to be a 1/2.5 scaled down model. The column cross section is 18 cm × 18 cm and the detailed reinforcements are shown in Fig. 3. The cross section dimensions of all girders are 12 cm × 18 cm and some typical reinforcement details are given in Fig. 3. The thickness of the concrete wall of the test model is 4 cm, and the reinforcement is 10 cm × 10 cm steel welded wire mesh with a wire diameter of 4 mm, as shown in Fig. 3. The horizontal reinforcement ratio is about 0.3% in agreement with the common construction practice in Taiwan with a reinforcement ratio of 0.25%∼0.4% which satisfies the minimum requirement of ACI [1] on the horizontal reinforcement ratio of a reinforced concrete wall. In order to enhance the workability for the casting of the 4 cm thick reinforced concrete wall, ready-mixed self compacting concrete is used. The height to width ratio (h_w/l_w) of the wall is equal to 4.0 such that it is categorized as a slender wall for which a flexural failure mode is assumed. The average concrete strengths for the first, second and third stories are respectively 2940 $\mathrm{N/cm^2}$, 4900 $\mathrm{N/cm^2}$ and 4212 $\mathrm{N/cm^2}$. The average tested yielding stresses for #2 (stirrups of beams and columns, and main reinforcements of floor slabs), #3 (main reinforcements of beams and columns), #4 (main reinforcements of footing), and 4 mm diameter wire (main reinforcements of walls) are respectively equal to 53.6 $\mathrm{kN/cm^2}$, 37.4 $\mathrm{kN/cm^2}$, 37.1 $\mathrm{kN/cm^2}$ and 78.0 $\mathrm{kN/cm^2}$. The seismic reactive weights in the second floor, third floor and roof are estimated to be respectively equal to 155 kN, 155 kN and 129 kN simulated by added lead blocks together with the reinforced concrete slabs and girders. The design base shear force is determined to be equal to 0.11 W

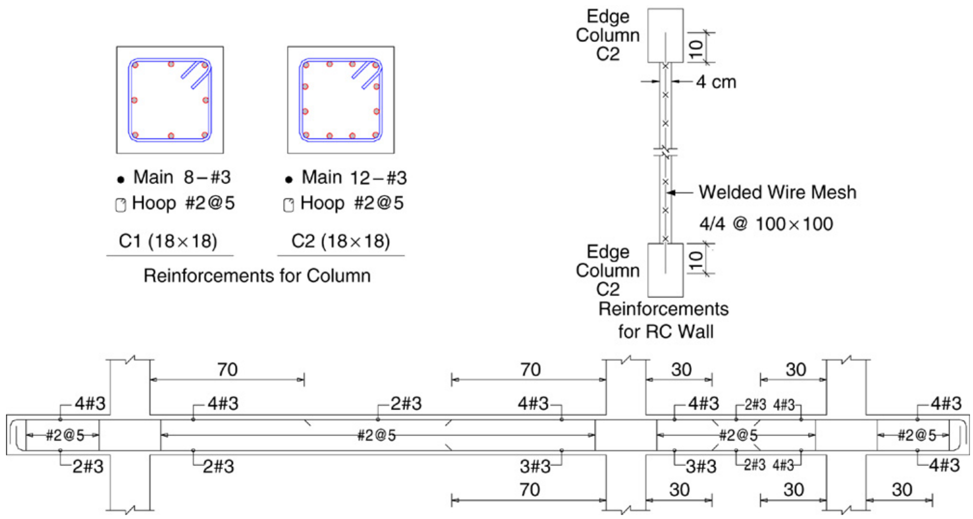

Fig. 3. Member dimensions and details of test structure.

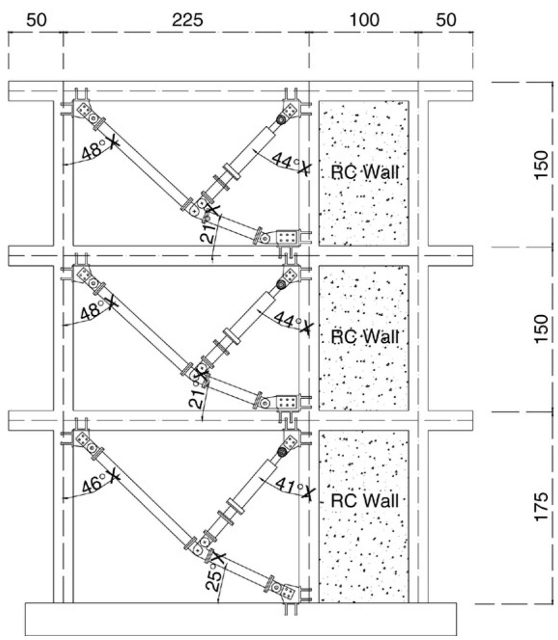

Fig. 4. Elevation of frame B of test structure with upper toggle-brace-dampers.

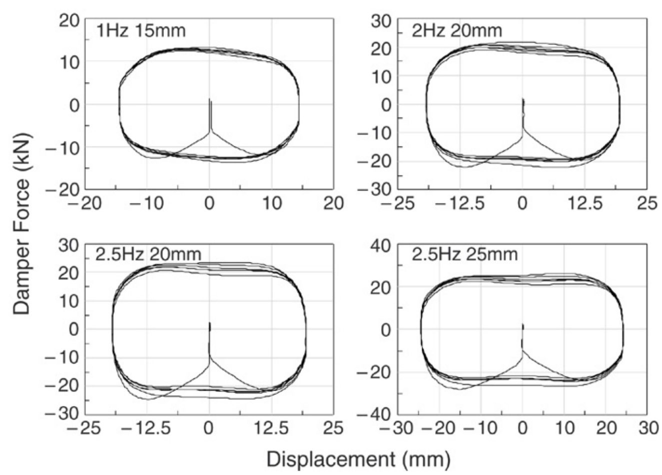

Fig. 5. Typical hysteretic curves from cyclic loading tests of viscous dampers.

according to the seismic design code of building structures of Taiwan [4], where W is the total seismic reactive weight of the test structure. It should be noted that the contribution by the lightly reinforced concrete wall to the stiffness and strength of the test structure is not considered during the design, according to current practice in Taiwan. In other words, the test structure is designed assuming it is a pure moment resisting structure rather than a dual system.

The viscous dampers are installed to the interior frame with an upper toggle brace mechanism, as shown in Fig. 4, to enhance the energy dissipation capability considering the presence of the additional lightly reinforced concrete wall which will limit the story drift during earthquake excitation. The dynamic cyclic loading tests of the dampers have been performed to determine the damping coefficient and damping exponent of the nonlinear viscous damper installed at each story. The damping exponents α_j of the viscous dampers at the first, second and third floors are respectively equal to 0.5, 0.4, and 0.4 while the damping coefficients, C_j, are correspondingly equal to 1.204 kN $(\text{s/mm})^{0.5}$, 2.117 kN $(\text{s/mm})^{0.4}$ and 2.027 kN $(\text{s/mm})^{0.4}$. The results of typical cyclic loading tests of the viscous dampers are shown in Fig. 5. A trial white noise shaking table test of the structure without dampers was conducted first to identify the fundamental properties of the test structure so that the damping contribution of the dampers can be estimated. The first mode natural period was 0.24 s and the corresponding modal displacements at first, second and third story were 1.0, 0.73 and 0.38, respectively. Substituting these preliminary test results, the damper properties and the parameters given in the installation layout of Fig. 4 into Eqs. (2) and (6), the theoretical added damping ratio of the toggle-brace-dampers is illustrated in Fig. 6. From the figure, it is obvious that the damping ratio contributed by the supplemental dampers is dependent on the roof displacement of the structure. However, it is worth noting that the fundamental dynamic properties such as natural period and mode shape may be slightly changed after the dampers are added to the structure [7].

Table 1
Test program

Test name	Ground motions	Nominal PGA (cm/s^2)
WN1	White noise	20
100% TCU078EW	EW component TCU078 station, 1999 Chi-Chi	439
200% TCU078EW	EW component TCU078 station, 1999 Chi-Chi	879
300% TCU078EW	EW component TCU078 station, 1999 Chi-Chi	1319
WN2	White noise	20

Table 2
Summary of maximum responses of test structure under various intensities of excitation

Excitation	Frame type	Max relative displacement at roof (mm)	Max absolute acceleration at roof (g)
100% TCU078EW	Without dampers	15.14	0.83
	With dampers	9.29	0.74
200% TCU078EW	Without dampers	32.24	1.37
	With dampers	17.15	1.09
300% TCU078EW	Without dampers	45.85	1.60
	With dampers	25.66	1.29

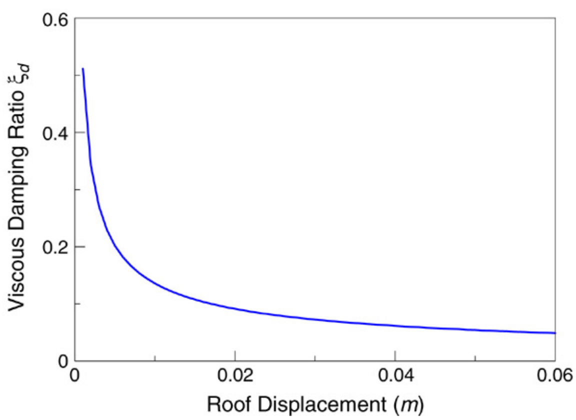

Fig. 6. Calculated added damping ratio contributed by viscous dampers.

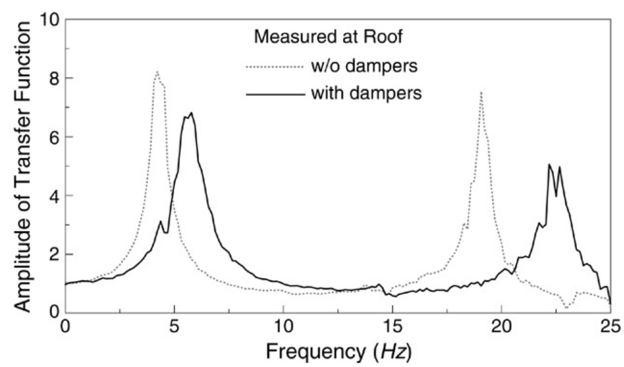

Fig. 7. Comparison of transfer functions of test structures with and without dampers subjected to 0.02g PGA white noise excitation.

Therefore, the added damping ratio may not be exactly the same as what Fig. 6 predicts. Nevertheless, Fig. 6 has provided a useful reference for the preliminary design of the dampers in an iteration process.

4. Test program and test results

The two test structures, one with dampers and the other without dampers are subjected to a series of white noise tests and earthquake tests, as shown in Table 1. All the ground motions are subjected to a time scale of $1/\sqrt{2.5}$ corresponding to the assumption of the test structure as a 0.4 scaled down model. The white noise test of the structure without added damper has shown a first mode damping ratio of about 7.6%. The white noise tests conducted between each two consecutive earthquake tests are to identify the changes on the natural frequencies and mode shapes of the test structures. A typical example at the very beginning of the tests is shown in Fig. 7 from which it is seen that the structure with toggle-brace-damper system has a slightly higher frequency than the structure without a damper. It is due to the fact that the gaps existing in the swivel joints of the dampers may slip during the structure shaking such that the dampers are visco-elastic-like rather than purely viscous [7]. Another reason may be because the connecting steel braces are insufficiently stiff comparing with the story stiffness such that the toggle-brace-damper systems reveal a slight visco-elastic behavior rather than a purely viscous behavior [14].

The maximum responses measured at the roof of the two test structures under various intensities of excitation are summarized in Table 2 from which it is seen that the dampers are effective in reducing the seismic responses of the test structure. However, the control on the displacement responses is more efficient than the control on the acceleration responses. Among those, for the test with the excitation of the 300% E–W component of the TCU078 station of the 1999 Taiwan Chi-Chi earthquake (denoted as TCU078EW), the damage patterns of the two test structures are summarized in Fig. 8 from which it can be realized that the toggle-brace-dampers can help the structure to minimize the structural damage even though the story drift of the test structure is very small due to the existence of the lightly reinforced concrete wall. In addition, from the damage pattern of the wall of the test structure without dampers, it is seen that the slender wall of the test structure does not deform in a cantilever shape as a whole. Instead, the wall

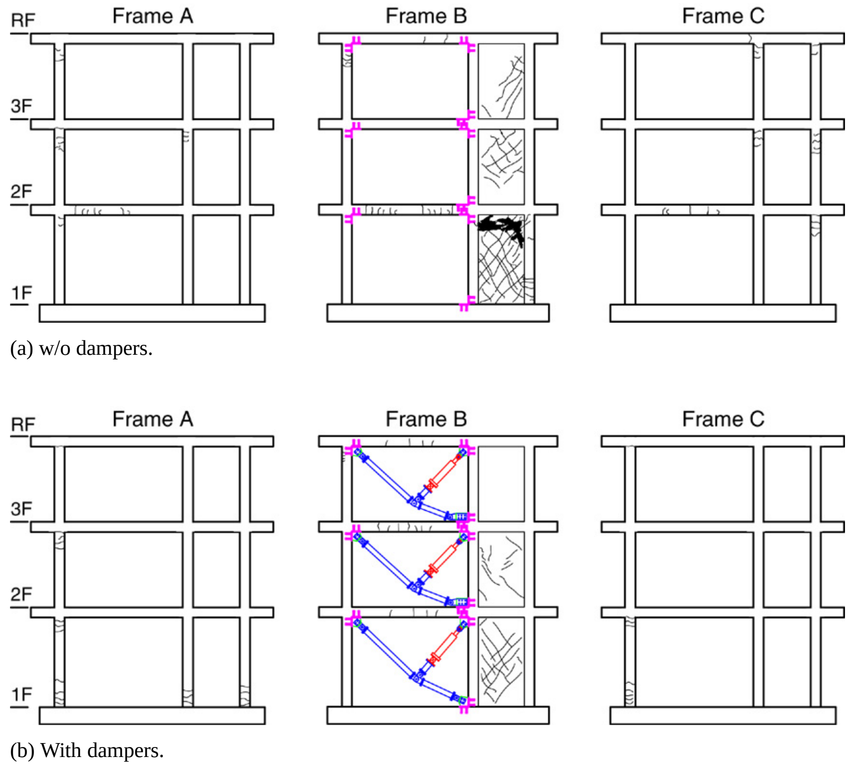

Fig. 8. Damage patterns of test structures subjected to 300% TCU078EW earthquake (a) without toggle-brace-damper system; (b) with toggle-brace-damper system.

elements in each story deform in a double-curvature manner which is different than what has been assumed or implied in most seismic design codes. The "clamping" effect of the concrete slab of each story on the wall has to be considered in the wall behavior in resisting the lateral seismic force. Also shown in Fig. 8, as other evidence for the double-curvature deformation of the wall element in each story, the damage pattern of the wall in the first story of the test structure without dampers under 300% TCU078EW earthquake shows that the upper part of the wall has suffered more severe damage than that of the lower part of the wall. This is because the wall is deformed in a double-curvature mode, and the concrete was poured from the top of the wall such that more aggregates were located in the lower part of the wall. Thus, the strength at the upper part of the wall in the first story is lower than the strength at the lower part of the wall.

Another evidence of the effectiveness of adding a toggle-brace-damper system to the RC structure with a lightly reinforced concrete wall is provided by the comparison of the response histories of the two test structures subjected to the same earthquake ground motions. As can be seen from Figs. 9 and 10, during the ground shaking of 300% TCU078EW earthquake record, both the acceleration and displacement response histories measured at each story of the test structure with toggle-brace-dampers are smaller than those of the test structure without dampers. In particular, the control of the displacement responses is significant. However, the control on the acceleration responses was not as significant as that of the control on displacement responses. This may be due to the following reasons: (1) the test structure is a very stiff structure for which the seismic response control is much more difficult compared with the control of a relatively flexible structure, in particular for the acceleration control; (2) during the ground shaking, the forces of the damper are added to each story and thus contribute to the acceleration response of each story even though there exist phase lags between the story shear forces of the RC frames and the damper forces at each story; and (3) the reduction of the spectral acceleration at the natural period or the effective period of the test structure is less sensitive to the damping increase, comparing with the reduction of the spectral displacement.

An interesting result shown in Fig. 11 indicates that the maximum story shear forces without the count of the contribution of the horizontal components of the damper forces are well reduced when the dampers are added to the test structure. Therefore, with the implementation of viscous dampers, the shear forces exerting on the RC frames are significantly reduced even though the reduction for the absolute acceleration response at each story is not as significant as that for the displacement responses.

5. Energy consideration

For energy consideration of a structure subjected to earthquake ground motions, the method proposed by Uang and Bertero [8] has long been used to determine various energy histories. Adopting the concept of Uang and Bertero, the absolute energy equation for structures with supplemental viscous dampers can be written as

$$\int m\ddot{v}_t \mathrm{d}v_t + \int c\dot{v}\mathrm{d}v + \int F_S \mathrm{d}v + \int F_D \mathrm{d}v = \int m\ddot{v}_t \mathrm{d}v_g \quad (7)$$

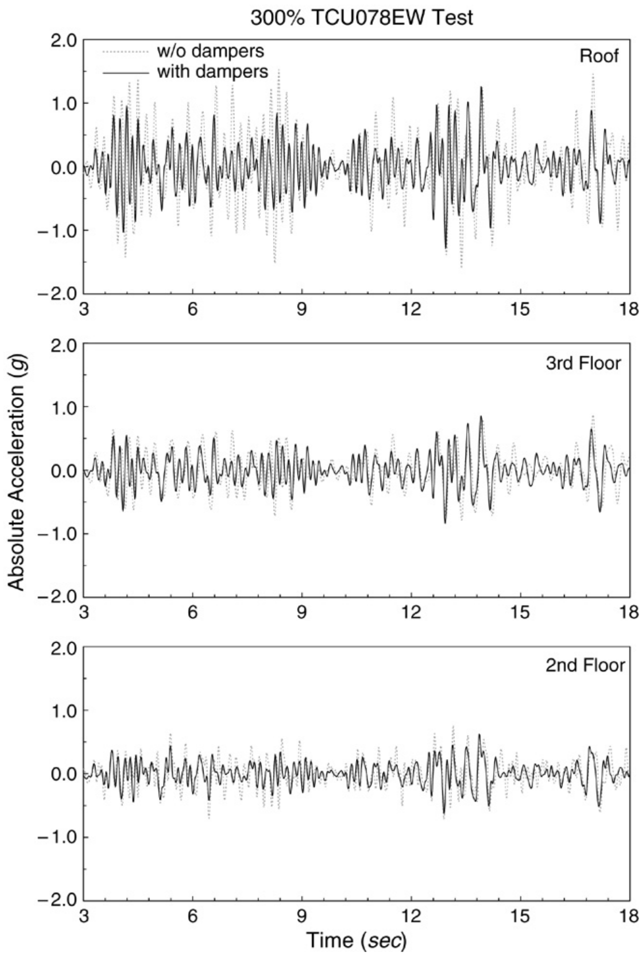

Fig. 9. Comparison of time histories of absolute acceleration of test structures with and without dampers subjected to 300% TCU078EW earthquake.

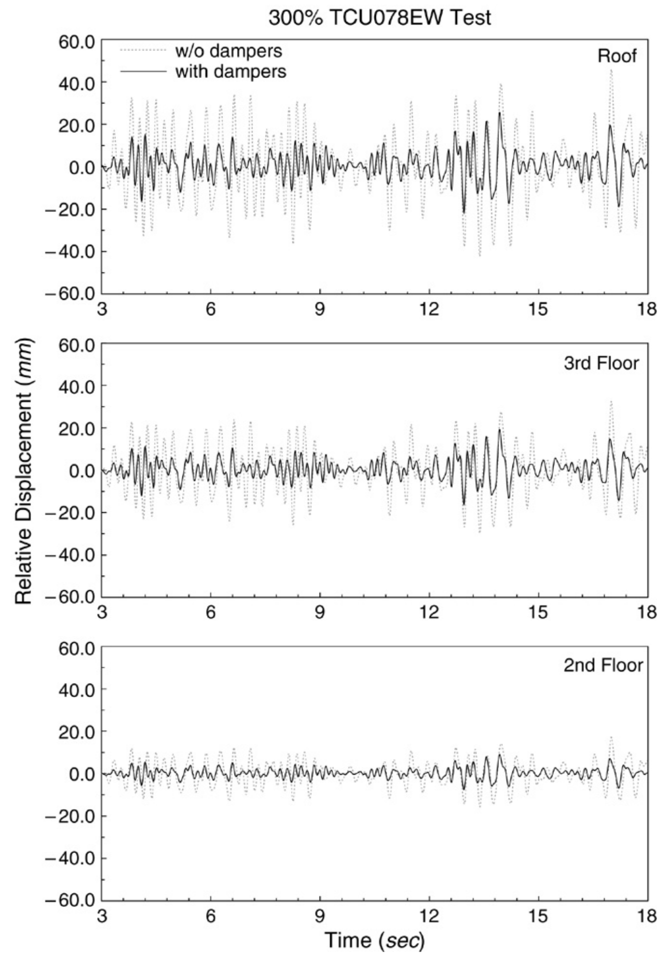

Fig. 10. Comparison of time histories of relative displacement of test structures with and without dampers subjected to 300% TCU078EW earthquake.

where the system parameters are denoted as m = mass; c = viscous damping coefficient; $\ddot{v}_t$ = absolute acceleration; v_t = absolute displacement; $\dot{v}$ = relative velocity; v = relative displacement; v_g = ground displacement; F_S = restoring force; and F_D = viscous damper force. The energy terms defined in Eq. (7) are respectively the kinetic energy, the inherent system damping energy, the absorbed energy composed of recoverable elastic strain energy and irrecoverable hysteretic energy, the viscous damping energy attributed to supplemental viscous dampers and the input energy by the earthquake ground motion.

The input energy histories of the 300% TCU078EW earthquake to the two test structures with and without added viscous dampers are calculated in Fig. 12. From the figure it is interesting to note that the total input energies determined at the end of ground shaking are approximately the same for both test structures. In addition, the input energy histories are somewhat similar for both test structures. These results are considered to be not reasonable since the test structure with supplemental viscous dampers has been subjected to less damage than the test structure without dampers, and thus the test structure with dampers should be subjected to less total energy input (or energy demand) than that to the test structure without viscous dampers. Some other irrational results can also be found in the study of Seleemah and Constantinou [13] in which the structure with added viscous dampers was subjected to the larger total input energy than the structure without viscous dampers. Based on these studies, it is considered that employing the absolute energy equation proposed by Uang and Bertero to determine the total energy input to the structure with supplemental viscous dampers may not be adequate. Therefore, in the following, the momentary input energy proposed by Hori et al. [9,10]

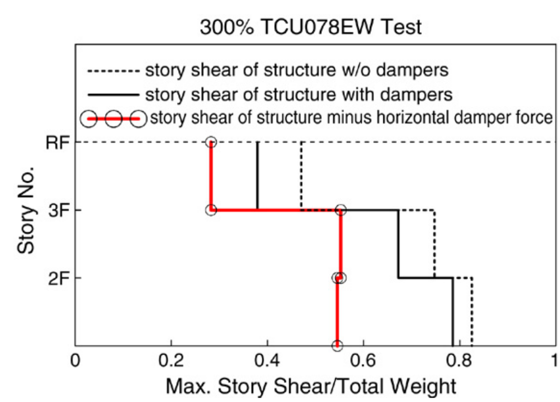

Fig. 11. Comparison of peak response of test structures with and without dampers subjected to 300% TCU078EW earthquake.

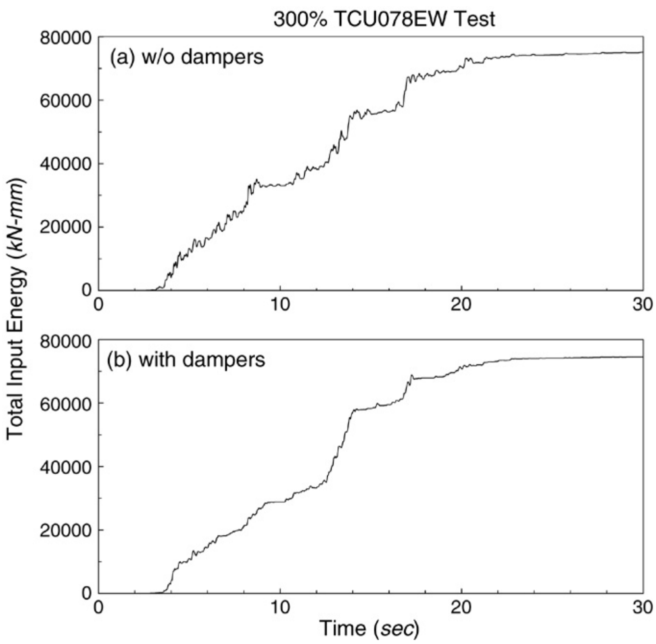

Fig. 12. Time histories of total input energy of test structures subjected to 300% TCU078EW earthquake (a) with toggle-brace-damper system; (b) without toggle-brace-damper system.

is employed to evaluate the energy demand on the two test structures.

Rewriting Eq. (7) as

$$\int m\ddot{v}_t\dot{v}_t\mathrm{d}t + \int c\dot{v}\dot{v}\mathrm{d}t + \int kv\dot{v}\mathrm{d}t + \int F_D v\dot{v}\mathrm{d}t = \int m\ddot{v}_t\dot{v}_g\mathrm{d}t \quad (8)$$

and taking the time derivative of Eq. (8), it is obtained

$$m\ddot{v}_t\dot{v}_t + c\dot{v}\dot{v} + kv\dot{v} + F_D\dot{v} = m\ddot{v}_t\dot{v}_g \quad (9)$$

which is the equilibrium equation of power at any instant of the ground shaking. The term $m\ddot{v}_t\dot{v}_g$ of Eq. (9) represents the input power to the structure by the earthquake. Instead of using the input power, Hori et al. has proposed the concept of momentary input energy as an index to represent the damage potential of an earthquake ground motion to a structure. For doing so, Eq. (9) is integrated with respect to the time within a time interval of $[t, t + \Delta t]$

$$\int_t^{t+\Delta t} m\ddot{v}_t\mathrm{d}v_t + \int_t^{t+\Delta t} c\dot{v}_t\mathrm{d}v + \int_t^{t+\Delta t} kv\mathrm{d}v + \int_t^{t+\Delta t} F_D\mathrm{d}v = \int_t^{t+\Delta t} m\ddot{v}_t\mathrm{d}v_g \quad (10)$$

where t and $t + \Delta t$ are two consecutive instants with a zero absolute velocity response of a structure during the ground shaking. According to the definition of the kinetic energy in Eq. (7), the kinetic energy will then be equal to zero at these two consecutive instants. The momentary input energy is then defined as

$$\Delta E = \int_t^{t+\Delta t} m\ddot{v}_t\mathrm{d}v_g \quad (11)$$

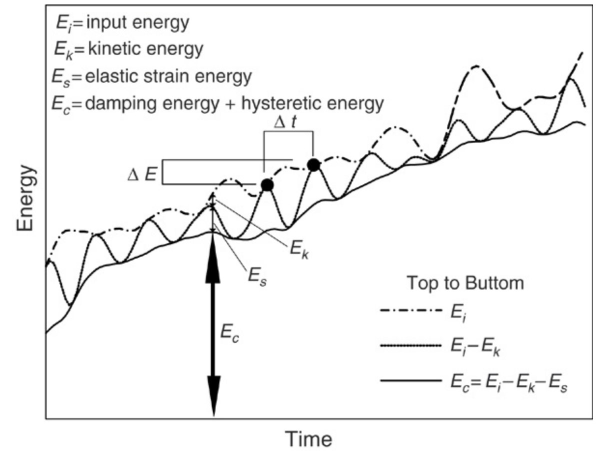

Fig. 13. Sketch of energy terms.

which is illustrated in Fig. 13. According to Eq. (11), the momentary input energy histories of the two test structures subjected to 300% TCU078EW are shown in Fig. 14. The area enclosed in each rectangle is the momentary input energy in the corresponding time interval $[t, t + \Delta t]$. From the figure it can be seen that the momentary input energy divided by Δt to the test structure with added dampers has been greatly reduced. Comparing the damage patterns of the two test structures, this result has demonstrated that the momentary energy method is more rational than the absolute input energy method in evaluating the damage potential of an earthquake to structures. Comparing Figs. 9, 10 and 14, it can also be seen that the larger structural responses always occur around the instants with larger input momentary energies. This result further shows the appropriateness of using the momentary input energy instead of the absolute input energy as an indication of the damage potential of earthquake ground motions to the structures. In addition, the momentary input energy history also indicates when the structure may possibly be subjected to damage during the process of ground shaking.

6. Conclusions

The construction of reinforced concrete moment-resisting building structures in Taiwan is commonly done with lightly reinforced exterior walls and interior partition walls whose contribution to the strength and stiffness of the structure are neglected in the design process. This study has disclosed that the toggle-brace-damper system is still effective in controlling the seismic responses of this type of structures even though their story drift (or relative story velocity) is relatively smaller than the pure moment-resisting buildings without the lightly reinforced concrete walls. Besides, the slender wall in a low rise shear type building such as the test structure of this study does not deform in a cantilever mode as a whole, rather it deforms in a double-curvature mode in each story. In addition, the momentary input energy method is proved to be superior to the absolute input energy method in demonstrating the effectiveness of adding viscous dampers to control the seismic responses of structures. The momentary input energy can also

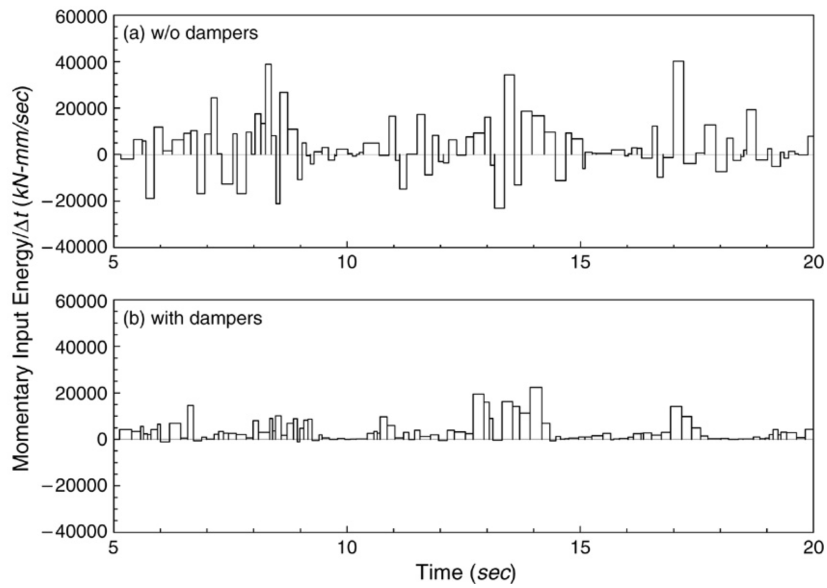

Fig. 14. Time histories of momentary input energy of test structures subjected to 300% TCU078EW earthquake (a) without toggle-brace-damper system; (b) with toggle-brace-damper system.

be used to identify the instants when the structures are subjected to larger energy demand during the ground shaking.

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