Seismic response of a tuned viscous mass damper (TVMD) coupled wall system

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Abstract: The authors propose a new tuned viscous mass damper (TVMD) coupled wall system for use in high-rise buildings. In this novel structural system, the TVMDs are arranged in a zig-zag configuration to couple wall piers in order to control both lateral inter-story drifts and floor accelerations. This novel structural system is used in this study to investigate the seismic performance of a 15-story prototype building. A nonlinear finite element model of the TVMD coupled wall (TCW system) is developed via the open-source finite element analysis software OpenSees. The TVMD is modeled by means of a newly compiled element named InertiaTruss, and the behavior of the coupled wall is simulated using a well-established multi-layer shell element approach. The seismic behavior of this novel TCW system is compared with walls coupled with conventional reinforced concrete beams (RCW system) and with viscous dampers (VCW system). The results indicate that, with a proper tuning design, seismic response parameters such as inter-story drift and floor acceleration are reduced by up to 16% and 28%, respectively, in the TCW system compared to the RCW system. The TCW system also shows better control of floor acceleration than the VCW system, though drifts are comparable. The force demands in the TVMD-to-wall joints

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are small due to the benefit of the TVMD zig-zag configuration. The analysis results also indicate that the effect of detuning in this new system is insignificant even when subjected to severe motions.

**Keywords:** tuned viscous mass damper (TVMD); coupled wall system; seismic control; nonlinear dynamic analysis; high-rise building.

1. Introduction

Recent earthquakes have highlighted that damage to structural components, which is primarily induced by large inter-story drifts [1], and damage to non-structural components, which is induced by both large inter-story drifts and floor accelerations [2], are the two greatest obstacles to achieving seismic resilience in high-rise buildings. As such, a structural solution that controls both inter-story drifts and floor accelerations is necessary to enhance the seismic resilience of these buildings. Reinforced concrete (RC) shear walls are often adopted in high-rise buildings to ensure adequate lateral stiffness and strength to resist wind loads and earthquake actions. Due to their high lateral stiffness, RC wall systems are able to adequately control inter-story drifts, but often amplify floor accelerations. While supplemental damping devices are often used to minimize floor accelerations, their use in high-rise buildings is inefficient in tall RC wall systems due to the relatively small inter-story drift and flexure-dominated lateral deformation mode. To overcome this challenge, a new structural system named tuned viscous mass damper (TVMD) coupled wall system (see Fig. 1) is proposed by the authors to enable control of both the inter-story drifts and floor accelerations of high-rise buildings [3].
In this system, TVMDs are installed between the coupled wall piers. The TVMDs (see Fig. 2) consist of a viscous mass damper (VMD) [4,5] and a spring in series [6]. The VMD is formed by connecting a rotational inertial mass in parallel with a rotational viscous damper with a ball screw system. The inertial mass described here has synergies with the inerter proposed by Smith et al [7], defined as a two-terminal inertial element that can generate a reaction force proportional to the relative acceleration between its two terminals.

Fig. 1. Building with TVMD coupled wall system.

Fig. 2. Tuned viscous mass damper.

\( m_r \) – inertial mass of inerter, \( k_b \) – stiffness of spring, \( c_d \) – damping coefficient of viscous damper

In the last two decades, a variety of control strategies that use inerter technologies have
been proposed. In these devices, the inertial force is commonly generated with the rotation of a flywheel. The rack and pinion system [8,9] and the ball screw system [4,5,10] are the primary methods used to convert the relative translational motion between two terminals into the rotation of the flywheel, and thus are recognized as two major types of inerter (i.e., rack and pinion inerter and ball screw inerter). By assembling a clutch into a conventional rack and pinion inerter, Makris and Kampas [11] further proposed the concept of a clutching inerter which ensures the inerter is only employed to resist the motion, while preventing the energy transfer from the flywheels back into the structure. The clutching inerter was later tested and modelled by Málaga-Chuquitaype et al. [12]. Additionally, the inertial force can be generated by the motion of fluids through two methods. One method to generate force with fluids is by driving a mechanical flywheel with a hydraulic motor [13], and another is by forcing the fluid to move rotationally in a helical pipe [14,15,16]. In addition, Gonzalez-Buelga et al. [17] developed another inerter-based vibration absorber by means of an electronic circuit. In this innovative vibration absorber, a capacitor is designed to be equivalent to an inerter based on the electrical and mechanical network analogy [7]. Through the interaction between the electronic circuit and the mechanical domain, this system can achieve similar vibration control performance as mechanical devices. Due to their outstanding vibration control performance, the inerter has been applied to a number of mechanical systems, such as car suspensions [18], motorcycle steering [19] and train suspensions [20], as well as to buildings and civil infrastructures [21].

Various types of inerter-based vibration absorbers have been proposed for response control of building structures, and they can be classified according to different configurations.
of inerter, spring and viscous damper (dashpot) elements. The well-studied inerter-based vibration absorbers include three types, i.e., the TVMD (Fig. 3 (a)), the tuned inerter dampers (TID, Fig. 3 (b)) [22] and the tuned mass damper inerters (TMDI, Fig. 3 (c)) [23].

Fig. 3. Inerter-based vibration absorbers.

\( m_r \) – inertial mass of inerter, \( k_b \) – stiffness of spring, \( c_d \) – damping coefficient of viscous damper, \( m_{TMD} \) – TMD mass in the TMDI absorber

A TVMD consists of a spring connected in series with an inerter and a dashpot connected in parallel (Fig. 3 (a)). The parameters of TVMDs are designed to have an optimal frequency and damping ratio, such that they can tune the vibration mode of the primary structure. At the tuning frequency, substantial energy dissipation can be achieved by amplifying the elongation of the dashpot (or motor [24,25,26]) within the TVMD, due to the opposite deformation of dashpot and spring. Ikago et al. have derived closed-form optimum design formulas based on fixed-point theory for the tuning design of single degree of freedom (SDOF) systems [6], and proposed a design method for shear type multi-degree of freedom (MDOF) structures [27]. These studies indicate that the use of the TVMD can simultaneously control the displacement and acceleration responses of a structure. In a TID, the inerter is connected in series with a spring and a viscous damper connected in parallel (Fig. 3 (b)). Lazar et al. [22] proposed the configuration of the TID absorber, and developed the design method of TIDs for use in shear type MDOF building structures. Afterwards, Lazar et al. [28]
and Sun et al. [29] further explored the use of TIDs in the vibration control of bridge cables. The studies of De Domenico et al. [30] showed that the addition of TIDs in base-isolated floors could improve the performance of the system by controlling both superstructure responses and based-isolated floor displacement. Wen et al.’s study [31] indicated TIDs and TVMDs have superior performance in mitigating the peak displacement responses of multi-story frame buildings, compared with the conventional viscous dampers. The TMDI comprises of a TMD connected with an inerter that acts as a mass amplifier for improving the efficiency of TMD (Fig. 3 (c)). The TMDI absorbers were proposed by Marian et al. [23], and they were proved to be efficient in control of vortex-induced vibration of long-span bridges [32], of earthquake-induced vibration of multi-story buildings [33] and base-isolated systems [34].

In the TVMD coupled wall system studied in this research, TVMDs are installed between the coupled wall piers in a zig-zag configuration. Previous studies have shown that this strategic arrangement of TVMDs makes efficient use of the relative vertical displacements between adjacent walls, which are induced by their flexural deformation, and maximize the response of the TVMDs [3]. Besides, this configuration enables the ease of design of the joints between TVMD and wall piers. At the joint (see Fig. 4), one TVMD is in tension and the other is in compression. As a result, the resultant vertical force at the joint provides the coupling action to the wall piers (vertical force components are in the same direction), while the resultant horizontal force applied to the joint is small (horizontal force components are in the opposite direction). The small horizontal tensile force demands at the joints ensure that the TVMD-to-wall joints are easy to design.
The author’s previous study [3] proposed the optimal tuning design methods for TCW system, and demonstrated the benefit of the system by linear response analysis. The objective of this study is to evaluate the seismic performance of the TVMD coupled wall system (TCW) under severe earthquakes, especially when the structural components undergo yielding and develop nonlinearity. This paper has two novel contributions. (1) The development of detailed nonlinear models to simulate the response of the TCW system. Due to lack of readily available elements to model TVMDs in commercial finite element (FE) programs, a new element named InertiaTruss is coded and compiled in the opensource software OpenSees to capture the behavior of the inerter and permit accurate simulations of the response of TVMDs. By integrating the TVMD model with a nonlinear wall model that adopts a multi-layer shell element, a rigorous nonlinear FE model of the TCW system is developed in this study. (2) The comparison of the novel TCW system with walls coupled with conventional reinforced concrete beams (RCW system) and walls coupled with viscous dampers (VCW system). Although past studies [21,31,35,36] compared the performance of TVMDs with other dampers, the comparison is made from analysis of simplified lumped-mass models or shear-type frames, and thus the findings of such studies do not extend to flexure-type
structures such as the coupled wall systems evaluated in this paper. Using the refined model from the first contribution, which can reasonably capture the nonlinear behavior of the systems, the comparison in this paper illustrates the advantages of the TCW under different earthquake intensities. The reasons behind the improved performance of the TCW vs. other systems are illustrated by means of energy analysis and modal decomposition analysis.

The paper is structured as follows. The second section presents a 15-story prototype building, and describes the design of TCW, RCW and VCW structural systems. The third section details the nonlinear model in OpenSees for the TCW system, particularly on the development and validation of the InertiaTruss element that is used for TVMD modelling. The fourth section presents nonlinear response history analysis results of the TCW system under various intensities of earthquake ground motions. The seismic behavior of this novel TCW system is compared with the RCW and VCW. The analysis results also highlight the robustness of the new TVMD coupled wall system under severe earthquake events and demonstrate its seismic control mechanisms.

2. Prototype structure and design of structural systems

2.1 Prototype structure

A 15-story prototype building is designed to study the seismic performance of the TCW, RCW and VCW systems. The building site is assumed to be located in Beijing with a peak ground acceleration (PGA) of 0.2 g for the design basis earthquake (DBE, with a probability of exceedance of 10% in 50 years) and a characteristic site period $T_g$ of 0.45 s. The prototype building adopts an RC frame-shear wall interacting structure. Each story has a height of 4.5 m, and each floor has plan dimensions of 40 m × 27 m (Fig. 5(a)). The total height of the
building is 67.5 m. The structure is designed in accordance with Chinese code for seismic
design of buildings (GB 50011-2010) [37] and Chinese technical specification for concrete
structures of tall buildings (JGJ3-2010) [38] using a modal response spectrum analysis
procedure, with a damping ratio of 5% assumed for all modes of vibration. The wall
thickness varies from 400 mm to 300 mm along the height of the structure. The Chinese code
[37] limits the inter-story drift ratio to 1/800 for RC frame-wall structures under the service
level earthquake (SLE, with a probability of exceedance of 63% in 50 years and a peak
ground acceleration of 0.07 g). In this study, a representative coupled wall is selected for
further analysis (see Fig. 5(a)). Three kinds of coupled wall systems, i.e., RCW, TCW and
VCW systems (see Fig. 5(b), 5(c) and 5(d)) are designed and analyzed for comparison. The
designs for the three systems are described in the following subsections.

![Prototype structure](image)

(a) Plan view of prototype structure (unit: mm)  
(b) RCW  
(c) TCW  
(d) VCW

**Fig. 5.** Prototype structure.

### 2.2 Design of RCW system

The proportion of moment resisted by the wall piers relative to the total moment resisted
by the coupled wall system when all coupling beams and wall piers yield is defined as the
coupling ratio (CR). El-Tawil and Kuenzli [39] recommend that for efficient designs, the CR ranges from 30% to 45%. In this study, the RCW system is designed to have a CR of 38.2%.

Fig. 6 shows the dimensions and reinforcement layouts of the wall piers. HRB400 steel reinforcement (nominal yield strength $f_y = 400$ MPa) and C45 concrete (nominal cubic compressive strength $f_{cu,n} = 45$ MPa and nominal axial compressive strength $f_{ck} = 29.6$ MPa) are adopted for the RCW. Note that the axial compressive strength, which is slightly lower than the characteristics cylinder strength value specified in codes in the U.S., Europe and Japan, is used in concrete design according to the Chinese code. The plastic hinge region represents the locations where the wall piers are likely to yield during severe earthquakes.

According to the Chinese code for seismic design of buildings (GB 50011-2010), the wall’s plastic hinge region should extend from the wall base to a height no less than 1/10 of the total structure height or the height of the first two stories. Therefore, in this study, the first two stories are assigned as the wall’s plastic hinge region. In addition, per the GB 50011-2010 requirements, the special boundary elements in the walls within the plastic hinge region are extended along one additional story (i.e., up to the third floor), to ensure the inelastic flexural deformation capacity of the walls. Therefore, in this structure, the wall piers in the first three stories are designed with special boundary elements, while those in other stories are designed with ordinary boundary elements. The boundary elements are designed to satisfy the strength demand under SLE and the detailing requirements of the GB 50011-2010 provisions [37].

Conventionally reinforced concrete coupling beams are adopted in the RCW as in typical Chinese designs for buildings of this height [38]. The widths of the RC coupling beams are identical with the wall pier thicknesses. The beams are designed to be governed by
flexure to ensure adequate ductility and satisfy the “strong shear-weak bending mechanism” in accordance with the recommendation of Chinese code GB 50011-2010 [37]. The floor seismic mass acting on the RCW varies from 50 ton to 63 ton for different floors (60.0 ton for floors 1-5, 54.4 ton for floors 6-10, 50.4 ton for floors 11-14, 63.2 ton for floor 15). The natural periods of the first three modes of the RCW model are 1.57 s, 0.38 s and 0.16 s.

![Fig. 6. Wall dimensions and reinforcement details of the RCW system (unit: mm).](image)

2.3 Design of TCW system

The selected coupled wall is also designed as a TCW system by installing TVMDs between two adjacent wall piers with a zig-zag configuration as shown in Fig. 5 (c). The geometric dimensions of the wall piers remain identical to those in the RCW system. As the installation of the TVMD will increase the damping ratio of the structure (see Fig. 8), the stiffness and strength demands for the coupling beams will reduce, resulting in lower coupling beam depths. The primary coupled wall system with reduced coupling beams is referred to as PCW for simplicity. Note that the coupling beams could potentially be removed.
in some structures if the TVMDs themselves can adequately couple the wall piers to enable
the system to satisfy the drift limits and strength requirement. However, due to the strict drift
limits of the Chinese codes, coupling beams are required to meet code limits for this structure.

The TVMDs are designed using the single-mode tuning design method [3], as follows:

I. Calculate the modal properties of the PCW and select the target tuning mode. If
selecting the $i^{th}$ mode as the target tuning mode, the modal displacement demand vector
of the TVMDs $\{\phi_d\}$ can be calculated from the $i^{th}$ mode shape vector of the PCW $\{\phi\}$,
as detailed in Reference [3]. For a given mode, the modal displacement demand of a
TVMD is defined as the relative displacement between the two nodes connected by the
TVMD in the corresponding mode shape vector.

II. Determine the apparent masses of the TVMDs. The apparent mass vector of TVMDs
$\{m_{r1}, m_{r2}, \ldots, m_{rn}\}$ can be determined by assuming it to be proportional to the modal
displacement demand vector of the TVMDs $\{\phi_d\}$, for a given mass ratio of $i\mu$. The mass
ratio $i\mu$ can be calculated as

$$i\mu = \frac{\{\phi_d\}^T[M_{p}]\{\phi_d\}}{\{\phi\}^T[M_{p}]\{\phi\}}$$  (1)

$$[M_{r}] = \begin{bmatrix} m_{r1} & 0 & \cdots & 0 \\ 0 & m_{r2} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & m_{rn} \end{bmatrix}$$  (2)

where $[M_{p}]$ denotes the mass matrix of the PCW, and the matrix $[M_{r}]$ is a diagonal
matrix, in which the $j^{th}$ diagonal element $m_{ij}$ denotes the apparent mass of the TVMD in
the $j^{th}$ story.

III. Determine the optimal frequency and damping ratio of TVMDs. For the target tuning
mode frequency $\omega$ of the PCW and a given mass ratio $\mu$, the optimal frequency $\omega_d$ and optimal damping ratio $\xi_d$ of TVMDs can be calculated by the fix point method as recommended by Ikago et al [6] as follows:

$$\omega_d = \frac{\omega}{\sqrt{1 - \mu}}$$  \hspace{1cm} (3)

$$\xi_d = \frac{1}{2} \frac{\sqrt{\frac{3}{2} \mu}}{(2 - \mu)}$$  \hspace{1cm} (4)

IV. Determine the stiffness and damping coefficient for each TVMD as follows:

$$k_{bj} = \omega_d^2 \cdot m_j \quad (j = 1, 2, ..., n)$$  \hspace{1cm} (5)

$$c_{dj} = 2 \cdot \omega_d \cdot \xi_d \cdot m_j \quad (j = 1, 2, ..., n)$$  \hspace{1cm} (6)

where $k_{bj}$ and $c_{dj}$ denote the spring stiffness and the damping coefficient of the dashpot within the TVMD in the $j^{th}$ story.

In this study, the TCW system is intentionally designed to suppress both inter-story drifts and floor accelerations simultaneously. For a coupled wall system, the inter-story drifts are dominated by the 1$^{st}$ mode response, while the floor accelerations are driven by a multi-modal response. A previous study by the authors (reference [3]) indicated that the addition of TVMDs not only suppresses the dynamic response of the tuning mode, but also increases the damping ratios of lower-order modes. Therefore, in this study, the 2$^{nd}$ mode is selected as the target tuning mode in the design of the TCW system (referred to as “TCW-2nd”). To validate previous findings [3], the system in which the TVMDs are tuned to the 1$^{st}$ mode (referred to as “TCW-1st”) is also designed and analyzed for comparison. As discussed in Subsection 4.2, tuning to the 2$^{nd}$ mode leads to superior seismic performance
than tuning to the 1st mode.

In this study, the mass ratio of the TCW-2nd system is set to be 0.1 based on engineering judgment. Very low mass ratios are ineffective in controlling response, whereas large mass ratios are difficult to manufacture. The force demand in structural elements and the inter-story drift of the TCW system are calculated from a modal response spectrum analysis under SLE in accordance with the Chinese design code. With the distributed TVMDs along the height, the system shows non-classical damping characteristics [40]. As such, the complex complete quadratic combination (CCQC) method [41] is used for modal response combination. Note that, the design is an iterative procedure by adjusting the section height of coupling beams. The response spectrum analysis and CCQC modal combination procedure are conducted using a linear numerical model developed in Matlab [3] to facilitate design iterations. For the TCW-1st system design, the dimensions and reinforcement details of the coupling beams remain identical to those of the TCW-2nd system. The mass ratio of TVMDs in the TCW-1st system is determined to be 0.02 by trial and error, such that its maximum inter-story drift under SLE is marginally less than 1/800, i.e., the drift limit specified by the GB 50011-2010 provisions. Note that both systems have nearly identical inter-story drifts under SLE. Although the TCW-1st system has a lower mass ratio than that of the TCW-2nd, the total TVMD apparent mass of the former (855 ton) is more than twice than that of the latter (369 ton), because the modal mass of the 1st mode of the primary structure is much greater than that of the 2nd mode.

The dimensions and reinforcement of the RC coupling beams of the TCW-2nd system and the optimal design parameters of the TVMDs are shown in Fig. 7. The TVMD design
parameters of the TCW-1st system are listed in Appendix I. In this study, the maximum apparent mass of one TVMD is 71 ton. Due to the large amplification effect achieved by the ball screw mechanism, such an apparent mass is not difficult to achieve by a device of typical dimensions. The reinforcement of the wall piers in the TCW system is identical to that in the RCW system, as shown in Fig. 6.

![Beam sections in the PCW, TCW and RCW system](image)

**Fig. 7.** Design of the TCW-2nd system and VCW system.

For the non-classically damped TCW system, complex modal analysis is conducted to obtain its complex modal properties. The participation mode vectors of the TCW system is calculated from the complex modes, as detailed in Ikago et al. [42]. Fig. 8 summarizes the natural periods, damping ratios and participation mode vectors of the TCW-2nd and PCW system. Note that the $i^{th}$ participation mode vector of the TCW-2nd system is calculated from

<table>
<thead>
<tr>
<th>$m_i$ (ton)</th>
<th>$k_b$ (kN/m)</th>
<th>$c_d$ (kNs/m)</th>
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<tbody>
<tr>
<td>46</td>
<td>13715</td>
<td>315</td>
</tr>
<tr>
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<th>$c_{vd}$ (kNs/m)</th>
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the $i^{th}$ conjugated pair of complex mode shape vectors, while that of the PCW is calculated from the $i^{th}$ real mode shape vector. Fig. 8 indicates that the 2$^{nd}$ mode (i.e., the target tuning mode) of the PCW system is split into 16 modes (i.e., the 2$^{nd}$ to 17$^{th}$ modes) of the TCW-2nd system due to the addition of 15 TVMDs. Among these 16 modes, the first and the last modes are dominated by global vibration of the coupled wall system, while the others are dominated by the local vibration mode of each TVMD, which has a period of vibration and damping ratio consistent with the adopted optimal tuning parameters. The modal periods, damping ratios and participation vectors of the 1$^{st}$, 18$^{th}$, and 19$^{th}$ mode of the TCW-2nd system are nearly identical to those of the 1$^{st}$, 3$^{rd}$ and 4$^{th}$ mode of the PCW system, respectively.

<table>
<thead>
<tr>
<th>Mode</th>
<th>PCW Period (s)</th>
<th>Damping ratio</th>
<th>Mode</th>
<th>TCW-2nd Period (s)</th>
<th>Damping ratio</th>
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</table>

$t_{opt} = 2\pi/d_{opt} = 0.363\text{s}; \ \zeta_{opt} = 0.199$

**Fig. 8.** Dynamic properties for PCW and TCW-2nd system.

2.4 Design of VCW system
The VCW system has identical coupling beams and wall piers as the TCW system, but
the TVMDs are replaced by viscous dampers (VDs). To permit a fair comparison, the sum of
damping coefficients of all VDs remains identical to that of TVMDs in the TCW-2nd system.
However, the damping coefficient distribution of VDs along the structural height is optimally
designed. Most optimal design methods for VD parameters developed in past studies are
applicable for shear-type structures (e.g., frames) [43,44,45]. A recent study by Huergo et al.
[46] investigated the VD parameter distribution for flexural-type tall buildings, and the
results indicated adequate performance of the story shear strain energy distribution (SSSED)
method. Therefore, the VD parameters of the coupled wall system are optimally designed
using the SSSED method based on the 1st mode shape. The obtained damping coefficients $c_{vd}$
of VDs are listed in Fig. 7. The natural periods of the first three modes of the VCW system
are 1.66 s, 0.38 s and 0.15s.

3. Finite element model

3.1 FE model of TVMD

In a previous study [3], a simplified numerical model of the TVMD coupled wall system
was developed in Matlab based on FE theory. In those models, the shear wall was simulated
by Timoshenko beam elements and the TVMDs were simulated by a 3 node-5 DOF element.
Although the model was sufficient to support the design and linear dynamic analysis of the
TCW system, it could not capture its nonlinear behavior. While several commercial FE
programs provide elements and material models to capture the nonlinear behavior of shear
walls, they yet lack elements for TVMD modeling. In a recent study, Málaga-Chuquitaype et
al. [12] proposed a method to simulate an inerter’s behavior through a combination of a
rotational inertia (defined as a mass in a rotational DOF) and an additional rigid link that is perpendicular to the motions of the inertia. However, this approach needs an extra dimension to set the additional rigid link element in the model, and thus following this approach it is not possible to model diagonal inerters or TVMDs in a three-dimensional structure. Therefore, in this study, a new inerter element is developed to model TVMDs based on the opensource FE analysis platform OpenSees. Note that, in the course of this study, Li et al. [47] proposed another method for modelling inerters in OpenSees, by using a link element with a newly compiled inerter material. Both Li’s model and the model proposed in this study permit the modeling of the inerters in three-dimensional structures.

As shown in Fig. 2, a TVMD model can be deconstructed into three elements – a viscous damper, an inerter, and a spring. Well established damper and spring elements are readily available and the behavior of the inerter can be simulated by a new truss element named InertiaTruss which is newly developed by the authors. As a TVMD or an inerter is only used to transmit axial force, it is a natural choice to develop this inerter element based on a three-dimensional truss element formulation. The command required to create the InertiaTruss object in the OpenSees domain is shown in the Appendix II. Special attention is paid in the development of this element to the construction of its inertial matrix. The inertial mass matrix of an inerter is related to relative acceleration, similar to the damping and stiffness matrices, which are related to velocity and displacement within a damper and spring element, respectively. Eq. (7) presents the equation of the force and acceleration of an inerter in the local coordinate system, and the symbols of the equation are illustrated in Fig. 9.
The authors extended the getMass() function in OpenSees, which can construct the inertial mass matrix \([M_r]\) of the InertiaTruss element from its input parameters. In addition, the authors extended the getResistingForceIncInertia() function in OpenSees, which can calculate the resisting forces of the InertiaTruss element using Eq. (7), based on the inertial mass matrix and the accelerations at its two end nodes. Note that, the linear transformation (from the local coordinate system to the global coordinate system) should be conducted for the inertial mass matrix \([M_r]\), as this matrix has off-diagonal non-zero terms.

To validate the accuracy of the proposed model, the InertiaTruss element is calibrated against past TVMD experimental data. The following presents one example of a TVMD shake table test conducted by Watanabe et al. [48]. The values of the translational apparent mass \(m_d\), damping coefficient \(c_d\), exponent \(a\), and supporting stiffness \(k_b\) of the tested specimen are shown in Fig. 10. During the test, the TVMD was subjected to a sinusoidal
displacement of the shake table with an input displacement amplitude of 15 mm and an exciting frequency of 0.35 Hz. More details of this test can be found in [48].

To simulate this test in OpenSees, the TVMD is modeled as illustrated in Fig. 2. This model contains two end-nodes to represent the two terminals of TVMD and a mid-node to represent the connection between the spring and the inerter/dashpot. TwoNodeLink elements with relevant elastic and viscous properties are used to simulate the spring and dashpot, respectively. In this validation, the dashpot is modeled with a power law constitutive behavior. The inerter is represented by the newly developed InertiaTruss element. The stiffnesses of the transverse and rotational DOFs in the local coordinate of the twoNodeLink elements are set to be infinitely rigid (by using a stiffness $10^6$ times larger than the axial stiffness of the spring) to ensure the mid node moves collinearly with two terminal nodes.

The Newmark-$\beta$ method is adopted to solve the integration of differential equations, and the Newton-Raphson algorithm is used to solve the nonlinear equations. Fig. 10 shows the analytical versus experimental time history results of the TVMD axial displacement ($u$), the displacement of the supporting spring ($u_s$) and the displacement of the inerter/dashpot ($u_d$). The axial force-displacement relation curves of the test and the numerical results are also compared in Fig. 10. The numerical model follows the test data well, which validates the development of the InertiaTruss element. The comparison with other experiments shows similar results and further validates the accuracy of the developed model.
Fig. 10. Comparison between TVMD experimental and analytical results.

(Superscript TEST denotes TVMD experiment and FEA denotes analysis simulation)

Furthermore, the accuracy of the proposed model is validated for the case when the TVMD is placed in a diagonal configuration. A mechanical model as shown in Fig. 11 was developed in OpenSees and verified against the numerical model of the 3 node-and-5 DOF TVMD element used in previous studies [3]. Note that this specific assembly is for validation of the developed OpenSees TVMD model, and it does not necessarily represent a real-life structure. This model has two DOFs, i.e., the displacement of two lumped masses in the $x$ direction. The two lumped masses are connected by a TVMD and spring in parallel, which are placed at a 45° angle relative to the horizontal direction. The parallel spring and the horizontal flexible spring at the support ensure the stability of the system under the loading. The modeling parameters are illustrated in Fig. 11. The structure is excited by dynamic forces at two lumped masses, where the forces are equal to the masses multiplying the acceleration input. The NS component of the El Centro record of the 1940 Imperial Valley earthquake is used as the input motion. Other assumptions regarding the choice of elements and solution algorithms are the same as described in the previous validation. The model with the 3 node-and-5 DOF TVMD elements is developed in Matlab and analyzed with the same input motion. Fig. 11 shows the comparison between the results of time history displacement data and axial force-displacement relation obtained via OpenSees and Matlab. The results are
almost identical, which suggests the proposed OpenSees model is capable of simulating the behavior of the inerter placed in a diagonal arrangement and is consistent with earlier models.

**Fig. 11.** Comparison between OpenSees analysis and Matlab calculations.

(Superscript CAL denotes Matlab calculation, FEA denotes OpenSees analysis)

### 3.2 FE models of RC wall piers and coupling beams

To develop the numerical models for the coupled walls in OpenSees, the RC wall piers and the RC coupling beams are discretized adopting multi-layer shell elements [49] and link elements, respectively. In the multi-layer shell element, concrete and distributed reinforcement are represented by concrete layers and smeared rebar layers, respectively. The longitudinal rebars in the boundary elements are modeled with truss elements connected to shell elements along their length. The concrete cover is represented by the Kent-Park model [50], where the strain at the peak stress is assumed to be 0.002 and the residual compressive strength after the ultimate strain is assumed to be 0.2 times the concrete peak stress.

Stirrup-confined concrete is represented by the Saatcioglu-Razvi model [51], which takes into account the increase in strength and ductility of concrete due to confinement. A bilinear model is used to represent the uniaxial tensile stress-strain relationship of concrete, where the ultimate tensile strain is assumed to be 0.001. The uniaxial stress-strain relationship of the steel reinforcement is represented by the Giuffrê-MenegottoPinto model [52], where a strain-hardening ratio is taken as 1% and the parameters $R_0$, $cR_1$ and $cR_2$, controlling the
curve shape of the transition from elastic to plastic branches, are taken as 18.5, 0.925 and 0.15. This model for wall piers has been validated with test results [53]. Note that although only the first two stories are assigned as the wall’s plastic hinge region, the wall piers are modeled as nonlinear throughout their height. The nonlinear history response analysis results, as described in section 4, indicate that the plastic deformation of wall piers is concentrated within the lower two stories, as intended by the code design.

The nonlinear elements used to simulate the RC coupling beams are defined with twoNodeLink elements with a hysteretic material model, where the skeleton curve of the force-displacement relationship is as defined per ASCE/SEI 41-17 provisions [54] (see Fig. 12). \( \theta \) denotes beam rotation and \( \frac{V}{V_n} \) denotes the normalized shear force of the coupling beam. The effective flexural stiffness is assumed to be 30% of the stiffness value based on the gross section properties as recommended by ASCE/SEI 41-17 [54]. The maximum strength (point C in Fig. 12) is taken as 1.15 times the yield flexural strength as recommended by Kwan and Zhao [55]. According to the shear stress level of RC coupling beams, the values \( a \), \( b \) and \( c \) are taken as 0.025 rad, 0.04 rad and 0.75, respectively. The model for RC coupling beams has been validated by comparison with experimental data [57].

![Fig. 12. The skeleton curve of RC coupling beam [54].](image)

### 3.3 Model assembly
The TVMD, coupling beam and wall pier models previously described are assembled into the TCW system as shown in Fig. 13(a). The rigid connection between coupling beams and wall piers is modeled with additional rigid beam elements (virtual beam shown in Fig. 13) which transfer the force developed at the coupling beam end to all wall shell elements at the coupling beam height. A floor rigid diaphragm is simulated by coupling the horizontal displacement DOF of the nodes on the same floor. The pin connections of the TVMDs to the wall pier are simulated by coupling the translational DOFs of coincident nodes. One of those nodes belongs to the wall pier, and the other two nodes belong to the TVMDs (as shown in the partial enlargement view in Fig. 13(a)). The Rayleigh damping model is adopted with the parameters determined based on the assumption that the damping ratio of the 1st and the 3rd mode are equal to 0.05. Note that, the Rayleigh damping model is only used for the primary structure of the coupled wall. The damping ratio of the TVMD elements is determined by their designed apparent inertial mass, spring stiffness and damping coefficient. The floor seismic mass acting on the coupled wall is represented by lumped masses assigned at each floor level. The FE model of the RCW system is assembled in a similar way as the TCW model, but without the TVMDs as shown in Fig. 13(b). The model of the VCW system is obtained by replacing the TVMDs with the twoNodeLink elements to represent viscous dampers as shown in Fig. 13(c).
4 Nonlinear response history analysis

4.1 Ground motion selection

Seven ground motions are selected from the PEER NGA West2 Ground Motion Database [56]. The target spectrum is the DBE response spectrum specified in GB 50011-2010 (with a damping ratio of 5%) and the linear scaling method is used to minimize the mean square error (MSE) of the ground motion response spectra with respect to the target spectrum over the period range of interest. The long period and the short period portion range of interest are determined with consideration of the 1st and 2nd mode periods of the TCW-2nd, VCW and RCW model [57]. The period range of interest is selected to span $[0.1 \text{s}, 1.5T_{2\text{TCW}}]$ and $[T_{1\text{RCW}} - 0.2, 2T_{1\text{TCW}}]$, where $T_{1\text{RCW}}$ and $T_{1\text{TCW}}$ are the 1st mode periods of the RCW and TCW-2nd system respectively, and $T_{2\text{TCW}}$ is the 2nd mode period of the TCW-2nd system.

The characteristic site period $T_g = 0.45$ s falls within the range of $[0.1 \text{s}, 1.5T_{2\text{TCW}}]$. When
selecting the ground motions, record characteristics with magnitudes over 6, average shear wave velocity of top 30 m of soil \((V_{30})\) between 150 m/s and 250 m/s are used as search criteria without the restrictions on the fault type and distance. Fig. 14 illustrates the acceleration time history records of the selected ground motions, the individual record spectra, the mean spectrum of the selected records and the target spectrum. The same ground motion records are scaled by 0.35, 1, 2 and 3.1 to obtain the SLE, DBE, maximum considered earthquake (MCE, with a probability of exceedance of 2% in 50 years), and very rare earthquake (VRE, with a probability of exceedance of 0.5% in 50 years) ground motion suites, respectively.

![Figure 14](image)

(a) Acceleration time histories of scaled motions

(b) Response spectra at DBE

**Fig. 14.** Records and spectra of the selected ground motions.

4.2 Nonlinear response history analysis results

4.2.1 Inter-story drift responses

Fig. 15 compares the peak transient inter-story drift distribution of the TCW, VCW and RCW systems. The maximum inter-story drift ratio appears at the top stories for all systems.
Their inter-story drift responses are almost identical at SLE as a result of the structural design requirements. The maximum drifts at SLE and MCE are smaller than 1/800 and 1/100, respectively, as prescribed by GB 50011-2010 [37]. The TCW-2nd and RCW systems have similar inter-story drifts at SLE and DBE, although the former has higher damping and the latter has higher stiffness. With the increase of earthquake intensity, the TCW-2nd shows better performance in terms of displacement control and the maximum inter-story drifts are 15.7% and 13.5% smaller than those of the RCW when subjected to MCE and VRE shaking, respectively. However, the TCW-2nd shows insignificant advantages over the VCW for drift control. The maximum inter-story drifts of the TCW-2nd are only 4.1% to 11.1% smaller than those of the VCW for the four earthquake intensities considered. More details of the displacement control mechanism are discussed in subsection 4.4.1. The maximum inter-story drift of the TCW-1st system is nearly identical to that of the TCW-2nd at SLE motions. However, the peak drift responses of the TCW-1st system are up to 26.7% and 15.8% larger than those of the TCW-2nd at MCE and VRE.

![Graphs showing inter-story drift responses](image)

**Fig. 15.** Average peak responses of inter-story drift.
4.2.2 Floor acceleration responses

Fig. 16 compares the peak floor accelerations at each floor of the TCW, VCW and RCW systems. The maximum accelerations of the TCW-2nd are 27.8%, 25.4%, 21.3% and 2.9% smaller than those of the RCW when subjected to SLE, DBE, MCE and VRE, respectively. The key driver for these reductions in acceleration is the tuning effect and supplemental damping. The TVMDs tune the vibration of the 2\textsuperscript{nd} mode of the primary structure and provide additional damping to the 1\textsuperscript{st} mode, which helps to suppress the acceleration response. Furthermore, the RC coupling beams in the TCW-2nd system are less stiff than those in the RCW system, which leads to a longer vibration period of the system and, as a result, lower accelerations.

As seen in Fig. 16, the amplification in acceleration from the base to the top floor decreases with increasing earthquake intensity. This is attributed to damage to the structural systems, which results in the softening of the structure and, in turn, period elongation. Because the RCW system is more severely damaged than the TCW-2nd and VCW systems, the period elongation and its influence on reducing the acceleration response is higher than for the other systems. This compensates the effect of the TVMDs, and thus the reduction of floor acceleration between the TCW-2nd and RCW at very high intensities of earthquake shaking, i.e. VRE. Nevertheless, the control of floor acceleration is significant at DBE and MCE shaking, enabling the protection of nonstructural components, while the consideration of nonstructural protection is less critical at VRE where the focus is on collapse prevention.

The differences in floor acceleration response between the TCW-2nd and VCW systems are more significant than their drift response. Compared with the VCW system, the TCW-2nd
system has better control of accelerations at middle and upper stories, which coincides with
the 2nd mode shape (used for tuning of the TVMDs). The floor accelerations of the TCW-2nd
system are up to about 20% smaller in middle and upper stories (e.g. 21.5% smaller in the 9th
floor and 18.9% smaller in the 14th floor). These results suggest that the inertial masses
(inerter) and supporting springs can help to enhance the acceleration control of the structure
when they are optimally tuned to the 2nd mode of the primary structure. More details of the
acceleration control mechanism will be presented in the subsection of 4.4.2.

The peak floor accelerations of the TCW-2nd system are up to 20% smaller than those
of the TCW-1st system at SLE, DBE and MCE. While both systems have nearly identical
accelerations at the top floor at VRE, the TCW-2nd system shows smaller accelerations in the
middle floors. Considering both floor accelerations and inter-story drifts as discussed in
subsection 4.2.1, the 2nd mode tuning design strategy achieves superior performance than that
of tuning to the 1st mode.

![Maximum floor accelerations of the 1st and 2nd mode tuned TVM systems at SLE, DBE, MCE, and VRE](image)

**Fig. 16.** Average peak responses of floor acceleration.

4.2.3 Responses of typical components and joints
Fig. 17 illustrates the time history data of the strain in the outermost longitudinal reinforcement bar, within the boundary element, of the wall pier at the first floor of the TCW-2nd model. The results show the variation in the degree of nonlinearity at different earthquake intensities (the analysis results with the input of 1980 Victoria Mexico earthquake ground motion is used as an example). As the shell element of the wall pier is coupled with the truss element by sharing nodes, this strain can also represent the average concrete strain in the vertical direction of the adjacent shell element. As illustrated in the figure, the rebar stays elastic at SLE. With the increase of the earthquake intensity, the rebar strain exceeds the yield strain $\varepsilon_y$ (up to $3.0\varepsilon_y$ at VRE) and the period of the TCW-2nd system is also elongated as a result of structural damage. Fig. 18 compares the shear force – rotation hysteretic curves of the RC coupling beam at the top floor and the moment – plastic hinge rotation hysteretic curves of the left wall pier (the plastic hinge length is taken as 0.5 times the wall depth per ASCE/SEI 41-17 [54] provision) in the TCW-2nd and RCW systems at VRE. Beam rotations in the RCW system are larger than the TCW-2nd, reaching maximum values of beam rotation of 4.4%, which is indicative of substantial damage to the RC coupling beam [58]. The hysteretic curves of the wall pier also show the RCW system is much more damaged at VRE, reaching rotations 29% greater than the TCW-2nd system.

![Fig. 17. Time history response of strain (1980 Victoria Mexico motion).](image)
Fig. 18. Hysteretic curves of typical components (1980 Victoria Mexico motion @ VRE).

Fig. 19(a) depicts the average peak force demands at the joints of the TCW-2nd model, when the structure is subjected to MCE motions. $F_H$ and $F_V$ denote the resultant horizontal force and vertical force at the joint induced by a pair of TVMDs, respectively. $V_J$ denotes the total vertical shear force demand at the joint that also includes the shear force induced by the coupling beam. The joint horizontal force demands by TVMDs are rather small (less than 23.8 kN), except for the joint at the top floor that connects only to one TVMD. This is because the horizontal force components of two zig-zag TVMDs connected at one joint can partially counteract each other. Fig. 19 (b) shows the time history of TVMD forces (denoted as $F_D$) and horizontal force demand of the joint at the 6th and 14th floors. The two TVMDs connected to one joint have the forces with a phase angle of approximately 180° during the earthquake shaking, i.e., one in tension and another in compression. This illustrates the advantage of the zig-zag configuration of TVMDs. The vertical shear force demands at the joint are not greater than 320 kN. Note that in a conventional coupled wall system, the beam-to-wall joints are easily designed to develop the expected shear capacity of the coupling beam (e.g., a magnitude of 1000 kN in a common high-rise building). Therefore, the
TVMD-to-wall joints can be easily designed with adequate strength to satisfy their force demands. In practical design, special attention should be paid to the top floor joint, which may sustain relatively large horizontal tensile force demand.

![Diagram of forces at the TVMD-to-wall joint under MCE](image)

(a) Average peak forces  (b) TVMD and joint forces (1980 Victoria Mexico motion @ MCE)

**Fig. 19.** Forces at the TVMD-to-wall joint under MCE.

In this study, the stiffness and strength requirement of the supporting braces in the TVMDs and VDs can be easily satisfied. A conventional steel brace with a cross-sectional area of 4000 mm$^2$ (e.g., a circular tube with the diameter $D = 200$ mm and thickness $t = 6.6$ mm) and length of 3500 mm is stiff enough to ensure that the brace stiffness exceeds 15 times the maximum value of spring stiffness in the TVMDs, and exceeds 200 times the maximum values of $c_d \omega_2$ in the VDs. Note that the $c_d \omega_2$ is regarded as the dynamic stiffness of the viscous damper for the 2$^{nd}$ mode vibration, where $c_d$ denotes the damping coefficient of VD and $\omega_2$ denotes the circular frequency of the 2$^{nd}$ mode of the system. At the maximum force of the TVMDs or the VDs, the axial stress in the proposed brace size would correspond to merely 50 MPa. Therefore, it is easy to design the supporting braces in both systems to ensure adequate performance of both TVMDs and VDs.
4.3 Detuning effect

The TVMDs are optimally tuned to a specific mode of the primary structure. As a result, changes to the frequency ratio (the ratio of the TVMD frequency to structural frequency) will have a detuning effect, i.e. it will affect the efficiency of vibration control. When the structure is damaged and the frequency ratio $\beta$ of the TVMD deviates from the design value, the structural response may be amplified. Past studies have demonstrated that, for the TMD system, the detuning effect has an adverse influence on the structural dynamic response and leads to a significant decrease in the efficiency of the TMD when subjected to strong ground motions [59].

In this study, the detuning effect is studied by means of the nonlinear dynamic analysis results of the TCW-2nd and PCW systems. To compare the nonlinear and linear responses, elastic models of these two systems are developed by artificially increasing the member strengths by 100 times the original values. Fig. 20 compares the peak inter-story drifts and floor accelerations of the nonlinear models and linear elastic models when subjected to VRE.

For the linear models, the installation of the TVMDs can reduce the maximum responses of inter-story drift and floor acceleration by 7.7% and 14.2%, respectively. For the nonlinear models, the maximum inter-story drift and floor acceleration responses of the TCW-2nd system are reduced by 13.5% and 2.9%, compared than the PCW system. Although the vibration control efficiency of the TVMDs may be affected by the structural nonlinearity, the responses of the TCW-2nd system remain smaller than those of the PCW when the structure yields and suffers damage. This is different from the detuning effect of TMDs, which would amplify the structural responses to be larger than the uncontrolled responses of the primary
Unlike a conventional TMD, the TVMD is actuated by the relative motion at its two ends. A TVMD not only suppresses the dynamic response of the tuning mode, but can also provide additional damping to vibrations with lower frequencies than the tuning mode [3]. When the structure undergoes nonlinearity and the frequency of the target tuning mode decreases, the TVMDs can still suppress the vibration of that mode by providing additional damping.

4.4 Vibration control mechanism

4.4.1 Displacement control mechanism

Fig. 21 illustrates the energy time history responses obtained with the nonlinear analysis data of the 1980 Victoria Mexico earthquake (at MCE). $E_I$ denotes the input energy by the earthquake, $E_K$ denotes the kinetic energy of the coupled wall. $E_D$ denotes the dissipated energy of TVMDs or VDs. $E_B$ and $E_W$ denote the dissipated hysteretic energy of coupling beams and wall piers. $E_R$ is the energy dissipated by the Rayleigh damping of the system. The total input energy of the TCW-2nd, VCW, and RCW systems are $2.10 \times 10^6$ J, $2.32 \times 10^6$ J and $0.5 \times 10^6$ J.
2.43×10^6 J by the end of the record. The values of $E_D$ of the TCW-2nd and VCW are 4.55×10^5 J and 4.16×10^5 J, which coincides with the observation in subsection 4.2.1 (the TVMDs show modest advantages over the VDs for the displacement control when they are tuned to the 2\textsuperscript{nd} mode). The TVMDs dissipate nearly 20% of the input energy. Correspondingly, the energy dissipated by coupling beams and wall piers, $E_B$ and $E_W$, is reduced by 39% and 56% in the TCW-2nd system.

An earlier study [6] showed that when a TVMD is tuned to the 1\textsuperscript{st} mode of the primary structure, the inerter and the spring tend to deform in opposite directions, which amplifies the deformation of the ball screw device and enlarges the energy dissipation. However, in this study, the target tuning mode of the TCW-2nd system is the 2\textsuperscript{nd} mode to control both of the inter-story drifts and floor accelerations. Given the consideration that the displacement response is controlled by the 1\textsuperscript{st} mode, the displacement control by the TVMD does not achieve those same benefits. The energy dissipated by the TVMDs in the TCW-2nd system is close to that dissipated by the VDs in the VCW system. Therefore, the TCW-2nd and VCW system have similar displacement responses.
Fig. 21. Energy dissipation during the nonlinear dynamic analysis (1980 Victoria Mexico earthquake @ VRE).

4.4.2 Acceleration control mechanism

Fig. 22 illustrates the decomposition of the time history responses of floor acceleration of the TCW-2nd and VCW, and the forces of TVMD and VD in that story. The analysis results of the 4th story are selected, where the damping coefficients of the TVMD and VD are similar ($c_{d4} = 186$ kN·s/m, $c_{vd4} = 200$ kN·s/m). The floor accelerations or the damper forces of the 1st, 2nd and higher-order modes are obtained using the filter with the frequency ranges of $[0.8/T_1^{TCW}, 1.2/T_1^{TCW}]$, $[0.8/T_2^{TCW}, 1.2/T_2^{TCW}]$, and $[0.8/T_3^{TCW}, +\infty)$, respectively. A 3-order Butterworth filter is used for calculation. As shown in Fig. 22(a), both the 1st and 2nd mode contribute significantly to the floor accelerations. The control forces of TVMD and VD also concentrate on the first two modes and have a limited contribution from higher modes (see Fig. 22(b)). Both systems show similar acceleration responses in the 1st mode, and the forces...
provided by TVMD and VD in the 1st mode are nearly identical. Due to the tuning design to
the 2nd mode, the TVMDs provide significantly higher force than the VD in the 2nd mode, and
thus effectively suppress the acceleration response of the 2nd mode vibration. As the 2nd mode
usually has high contributions to the floor acceleration response in high-rise buildings, the
TCW system tuned to the 2nd mode results in an improved acceleration response than the
VCW system. Note that, although the TVMD force is significantly larger than the VD force,
the demands to the TVMD-to-wall joints is relatively small due to the advantageous zig-zag
configuration of TVMDs, as described in subsection 4.2.3. In addition, the strength and
stiffness requirements of supporting braces remain to be reasonable. Therefore, the relatively
larger force of TVMDs does not lead to difficulty in the design of the prototype structure.

![Modal decomposition of floor acceleration and damper force](image)

(a) Floor acceleration  
(b) Damper force

*Fig. 22.* Modal decomposition of floor acceleration and damper force (1980 Victoria
Mexico earthquake @ DBE).

5. Conclusions

This paper studies the seismic responses of an innovative tuned viscous mass damper
(TVMD) coupled wall (TCW) system used to enhance seismic performance of high-rise
buildings by means of finite element (FE) analysis. The nonlinear FE model of a
representative TCW structural system in a 15-story high rise building is developed in
OpenSees. The TVMD is modeled by means of a newly compiled element named InertiaTruss. The coupled wall system is modeled with a previously validated multi-layer shell elements for wall piers and link elements for coupling beams. A comparative study of the story drifts and floor acceleration responses, obtained from nonlinear dynamic analyses, is carried out for the TCW, a viscous damper coupled wall (VCW) and coupled wall with RC coupling beams (RCW). The response analysis is conducted at four ground motion shaking intensities, namely service level earthquake (SLE), design basis earthquake (DBE), maximum considered earthquake (MCE) and very rare earthquake (VRE). The following conclusions are drawn from this study.

1. The proposed element in OpenSees is capable of simulating the behavior of the TVMD in a three-dimensional structure, and the analytical results replicate well those obtained from experimental tests.

2. The TCW system tuned to the 2\textsuperscript{nd} mode (i.e., the TCW-2nd) achieves superior performance than the TCW system tuned to the 1\textsuperscript{st} mode (i.e., the TCW-1st). The peak inter-story drift of the TCW-2nd system is up to 27\% smaller than those of the TCW-1st system under severe earthquakes, and the peak floor accelerations of the former are up to 20\% smaller than those of the latter.

3. Although all systems have identical inter-story drift responses at SLE, as a result of stringent structural design requirements in the Chinese code, the peak inter-story drifts of the TCW-2nd are up to 11\% and 16\% smaller than those of the VCW and RCW systems under severe earthquakes, respectively. Furthermore, the peak floor accelerations of the TCW-2nd system are up to 22\% and 28\% smaller than those of the VCW and RCW systems,
respectively.

(4) A modal decomposition analysis indicates that although the TCW-2nd and the VCW systems have similar performance in terms of controlling floor acceleration responses of the 1st mode, the TVMDs in the TCW-2nd more effectively suppress the acceleration responses of the 2nd mode than the viscous dampers in VCW. As such, the TCW-2nd shows enhanced performance over the VCW, conventionally used to reduce accelerations.

(5) In terms of joint detailing, owing to the zig-zag configuration of TVMDs, the resultant horizontal force of a pair of TVMDs connected in a joint is rather small, which facilitates the design of the TVMD-to-wall joints. In addition, the analysis indicates that the stiffness and strength requirements of the supporting braces of TVMDs can be easily satisfied.

(6) The comparison between responses of the TCW system and the primary coupled wall (PCW) system indicates that nonlinearity of the structure has little effects on the TCW-2nd system when compared to an equivalent RC beam coupled wall system without TVMDs. The effect of detuning in this new system is insignificant.

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Appendix I

The TVMD parameters of the TCW-1st system:
<table>
<thead>
<tr>
<th>Story</th>
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Appendix II

element InertiaTruss $eleTag$iNode$jNode$mr

$eleTag$ unique element object tag

$iNode$jNode end nodes

$mr$ value of the inertial mass

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