Research Article

Numerical Investigation and Design of Reinforced Concrete Shear Wall Equipped with Tuned Liquid Multiple Columns Dampers

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The tuned liquid multiple column damper (TLMCD) is a variation of the tuned liquid column damper (TLCD) that includes multiple vertical columns. A new damping system that embeds TLMCDs within reinforced concrete shear wall systems, termed tuned liquid wall damper (TLWD), is proposed, augmenting the traditional structural component with energy dissipation capabilities. The objective of this study is to assess energy mitigation and strength trade-offs in designing TLWDs and demonstrating the promise of TLWD systems in tall buildings through vertically distributed applications. This is done by investigating the performance of the proposed TLWD through the finite element model (FEM) of a simplified representation of a 42-story building equipped with the multifunctional component. A strength model for the TLWD is developed to empower faster performance evaluation on more complex models. Results from the FEM are used to validate the strength model and show that the model could be used conservatively in assessing strength performance. Design considerations are discussed based on the simplified representation. In particular, to improve mitigation performance while maintaining strength, it is found that a single-layer arrangement of the vertical columns is preferred, while distributing the inertia among a higher number of smaller columns. The proposed TLWD is numerically evaluated on a more realistic system consisting of a multi-degrees-of-freedom representation of the 42-story building under stochastic wind excitation. Simulation results demonstrate that the TLWD, used in a vertically distributed configuration through the building, could be used to mitigate vibrations, outperforming a traditional TLCD system with geometric constraints under 20 design wind realization. Results from the numerical simulations also confirmed the design considerations established through the simplified representation.

1. Introduction

High-strength and lightweight materials combined with advanced construction techniques have empowered the design of taller and more flexible high-rise structures, yet making them vulnerable to excessive motion that may cause discomfort during daily operations and safety concerns against natural hazards [1]. A solution is the integration of supplemental damping strategies, including passive, semi-active, and active systems [2]. Amongst these strategies, passive systems are now widely accepted by the civil

engineering community due to their low cost and known long-term mechanical robustness and reliability.

Of interest to this paper is the use of tuned mass dampers (TMDs), a vibration mitigation system that dissipates energy through inertia [3]. The tuned liquid column damper (TLCD) is a variation of TMD proposed by Sakai et al. [4, 5] that consists of a partially water-filled U-shaped tank and an internal orifice to generate hydrodynamic head loss in addition to the internal surface friction while liquid sloshing occurs [6]. The technology is considered as an attractive alternative due to its ease of installation, low maintenance requirements,

cost-effectiveness, and high mechanical robustness [7, 8]. In the last decades, the concept of TLCD was well investigated in terms of design optimization [9, 10], also with applications to multihazard mitigation [11, 12]. However, TLCDs require significant space for implementation [12] and are only effective over a limited frequency range. Cao et al. studied a variation of the TLCD termed tuned liquid multiple columns damper (TLMCD), which consists of multiple vertical columns joined with a horizontal tube [13]. The TLMCD is capable of tuning at multiple frequencies and providing enhanced performance in a constrained geometry.

Recently, the authors proposed to implement a TLMCD system within a reinforced concrete (RC) shear wall, termed tuned liquid wall damper (TLWD), with the intent to improve vibration mitigation capabilities of shear wall components [13]. The advantage of the TLWD is in providing a geometrically restricted and distributed location for the integration of TLMCD systems while enabling multifrequency tuning. Others have proposed similar systems. Ye et al. studied cast-in situ hollow floor slabs equipped with an internal tuned liquid damper, augmenting the structural damping ratio by approximately 2% [14]. Matia and Gat investigated an elastic beam embedded with a fluid-filled parallel-channel network, and simulation results demonstrated that the fluid-solid interaction reduced structural motion [15]. To the best knowledge of the authors, none have studied the integration of TLCD systems within RC shear wall components.

Prior work from the authors in [13] was focused on the development and validation of an analytical model for TLMCD systems. In this paper, the investigation extends to the TLWD system, whereas perforated openings within RC shear walls are used to integrate a TLMCD. Increasing the height and length of perforated openings may increase structural damping by increasing the size of the internal TLMCD, and it may also significantly reduce the strength capacity and lateral motion performance of the RC shear wall, yet with less significant adverse effects on ultimate flexural capacity [16–18]. Here, both strength and mitigation functionalities are investigated within given geometric constraints to satisfy both strength and motion performance requirements, as well as the trade-off between strength and energy dissipation is studied.

The rest of the paper is organized as follows. Section 2 describes the analytical and numerical models used in conducting the study. This includes the proposed simplified strength model. Section 3 conducts parametric studies for a simplified structure equipped with a TLWD. Section 4 validates the proposed strength model and, further, the parametric evaluation by conducting additional simulation using the validated model. Results from both Sections 3 and 4 are used to establish design considerations for implementing a TLWD system. Section 5 verifies the findings by demonstrating the proposed TLWD on a 42-story building. Section 6 concludes the paper.

2. Models

This section presents the models used in conducting the numerical study. It starts by a presentation of the TLMCD analytical model developed in prior work [13], followed by a

simplified analytical model to evaluate strength of a TLWD. After that the finite element modeling methodology is described, followed by that of the numerical simulations in MATLAB.

2.1. TLMCD Model. A TLMCD consists of multiple vertical columns joined with a horizontal tube equipped with multiple orifices located in between adjacent vertical columns. Figure 1(a) is the schematic of a TLMCD and Figure 1(b) illustrates a TLWD system. In Figure 1(a), $\ddot{x}_g$ is the acceleration transmitted from the floor, x_i is the liquid surface displacement in the i th column, A is the cross-section area of vertical column, h is the initial vertical liquid surface height (equal for every column), l_i is the horizontal center-to-center distance between the i th and $i + 1$ th columns, v is cross-section area ratio of vertical column to the horizontal tube, and ψ_i is the i th orifice blocking ratio.

A nonlinear dynamic model for the TLMCD can be derived using Lagrange equations [13]. Starting with

$$\frac{d}{dt} \left(\frac{\partial T}{\partial \dot{x}_i} \right) - \frac{\partial T}{\partial x_i} + \frac{\partial V}{\partial x_i} = Q_{oi} + Q_{fi} + Q_{ei}, \quad i = 1, 2, \dots, N-1, \quad (1)$$

where $\dot{x}_i$ is the liquid velocity in the i th column, t is time, and Q_{oi} , Q_{fi} , and Q_{ei} are orifice damping force, friction force between the liquid and inner surface of columns, and liquid inertia force, respectively, the kinematic energy T and potential energy V are taken as

$$V = \frac{1}{2} \rho_l A g \sum_{i=1}^N x_i^2, \quad (2)$$

$$T = \frac{1}{2} \rho_l A \sum_{i=1}^N \dot{x}_i^2 (h + x_i) + \frac{1}{2} \frac{\rho_l A}{v} \sum_{i=1}^{N-1} v^2 l_i \left(\sum_{j=1}^i \dot{x}_j \right)^2,$$

where ρ_l is the liquid density. The analytical model is based on the following assumptions: (1) all vertical columns are identical in geometry, (2) liquid compression and liquid-air interface diffusion are negligible, resulting in a constant total liquid volume during vibrations, and (3) the characteristic dimensions of the columns' cross sections are considerably smaller than the total length of the tube, resulting in uniform motion of the liquid surface over the cross sections. An N -column TLMCD can be treated as $N-1$ degree of freedoms (DOFs) by allowing the displacement of the last column be dependent on the other columns' displacements using the assumed kinematic constraint $x_N = -\sum_{i=1}^{N-1} x_i$. It should be noted that the analytical model becomes that of a conventional TLCD for $N = 2$. The damping force Q_{oi} is assumed to be proportional to the square of the liquid velocity $\dot{x}_i^2$ [9]. The energy dissipated by the orifice damping force W_o can be written as

$$\delta W_o = - \sum_{k=1}^{N-1} \frac{1}{2} \rho_l A \left[v \eta_k \left| \sum_{j=1}^k \dot{x}_j \right| \left(\sum_{j=1}^k \dot{x}_j \right) \sum_{j=1}^k \delta x_j \right]. \quad (3)$$

FIGURE 1: Schematic of (a) TLMCD [13] and (b) TLWD system consisting of a shear wall with embedded TLMCDs.

Q_{oi} on the i th DOF and selected head loss coefficient model η are written as [19]

$$Q_{oi} = \frac{\partial W_o}{\partial x_i} = -\frac{1}{2} \sum_{k=i}^{N-1} \rho_l A \left[v \eta_k \left| \sum_{j=1}^k \dot{x}_j \right| \left(\sum_{j=1}^k \dot{x}_j \right) \right], \quad (4)$$

$$\eta = (\psi + 0.707\psi^{0.375})^2 (1 - \psi)^{-2}. \quad (5)$$

In addition to the head loss effect caused by the orifice blocks, friction resistance between the liquid and inner tube surface provokes a friction head loss effect. For mathematical tractability, the friction force is assumed to be proportional to the liquid velocity $\dot{x}_i^2$. An expression for the energy dissipated by the friction force W_f can be written as

$$\delta W_f = - \sum_{k=1}^N \rho_l A \left[\mu |\dot{x}_k| \dot{x}_k (h + x_k) \delta x_k \right] + \sum_{k=1}^{N-1} \rho_l A \left[\mu v l_k \left| \sum_{j=1}^k \dot{x}_j \right| \left(\sum_{j=1}^k \dot{x}_j \right) \sum_{j=1}^k \delta x_j \right], \quad (6)$$

where μ is the head loss coefficient due to the friction resistance. The mathematical expression for the friction force Q_{fi} over a small unit of time for the i th DOF is expressed as

$$Q_{fi} = \frac{\partial W_f}{\partial x_i} = -\rho_l A \mu \left\{ |\dot{x}_i| \dot{x}_i (h + x_i) + \left| \sum_{j=1}^{N-1} \dot{x}_j \right| \left(\sum_{j=1}^{N-1} \dot{x}_j \right) \left(h - \sum_{j=1}^{N-1} x_j \right) + \sum_{k=i}^{N-1} \left[v l_k \left| \sum_{j=1}^k \dot{x}_j \right| \left(\sum_{j=1}^k \dot{x}_j \right) \right] \right\}. \quad (7)$$

The inertial force Q_{ei} is induced by the lateral motion of the building due to an external excitation. Since the liquid mass remains constant during vibrations, the inertial force Q_{ei} on the i th DOF is solely given by the contribution from the liquid motion in the horizontal tube. The energy dissipated by inertial force W_e can be written as

$$\delta W_e = -\rho_l A \sum_{j=1}^{N-1} \left(\ddot{x}_g l_k \sum_{j=1}^k \delta x_j \right), \quad (8)$$

$$Q_{ei} = \frac{\partial W_e}{\partial x_i} = -\rho_l A \ddot{x}_g(t) \sum_{k=i}^{N-1} l_k.$$

2.2. TLWD Strength Model. An analytical strength model for the TLWD is developed enabling fast numerical simulations of MDOF representations. The shear wall strength equations from ACI 318-14 [20] are used to calculate the strength of TLWD with a given TLMCD embedment profile. The strength model is built upon considering only the void caused by the vertical columns. A special reinforcement scheme would be required to cope with the void caused by the horizontal tube, which is left to future work. Lateral stiffness k , nominal shear strength V_n , nominal flexural strength M_n , and nominal axial strength P_n are considered as strength objectives. The moment of inertia of TLWD along the in-plane direction can be written

$$I_{\text{TLWD}} = \frac{1}{12} b_w l_w^3 - \sum_{i=1}^n A_i \bar{y}_i^2 - \frac{1}{4} N \pi r^4, \quad (9)$$

where b_w is the width of the wall section, l_w is the wall length, A_i is the area of i th vertical column, $\bar{y}_i$ is the distance of the neutral axis to the center of i th circular opening, N is the number of vertical columns, and r is the radius of openings. Stiffness value k for the shear wall with respect to the section installed above is inversely proportional to its deflection when subjected to a unit point load applied at the top. The total deflection of the TLWD is determined from the sum of shear deflection Δ_v and moment deflection Δ_m , with k defined as [21]

$$k = \frac{1}{\Delta_m + \Delta_v}. \quad (10)$$

Quantities Δ_v and Δ_m for a TLWD subjected to a unit horizontal load F can be calculated as

$$\Delta_m = \begin{cases} \frac{F h_w^3}{12 E_c I_{\text{TLWD}}}, & \text{for a fixed wall} \\ \frac{F h_w^3}{3 E_c I_{\text{TLWD}}}, & \text{for a cantilever wall} \end{cases} \quad (11)$$

for a cantilever wall,

$$\begin{aligned} \Delta_v &= \frac{1.2 F h_w}{A_w G}, \\ E_c &= 4700 \sqrt{f'_c}, \\ G &= \frac{E_c}{2(1 + \nu)}, \end{aligned} \quad (12)$$

where h_w is height of wall, E_c is Young's modulus, G is shear modulus, ν is Poisson's ratio, f'_c is compressive strength, and $A_w = A_t - NA$ is the cross-section area of the TLWD taken by removing the cross-section area of the vertical columns NA from the cross-section area of the wall geometry A_t . The shear strength V_n of a TLWD consists of the summation of the shear strength from concrete V_c and that from transverse reinforcements V_s :

$$V_n = V_c + V_s. \quad (13)$$

Shear strength values are modified based on the simplified method from ACI-318 Section 11.5.4.5 using the reduction factor α :

$$\alpha = \frac{I_{\text{TLWD}}}{(b_w l_w^3 / 12)}, \quad (14)$$

with the expressions for V_c and V_s written as

$$\begin{aligned} V_c &= 0.166 \alpha \sqrt{f'_c} (b_w d - NA), \\ V_s &= \frac{\alpha A_v f_y d}{s}, \end{aligned} \quad (15)$$

where d is the effective depth of the cross section, taken as 0.8 l_w from the strain compatibility analysis [20]. A_v is the cross-sectional area of transverse reinforcement, f_y is the yield strength of the steel reinforcement, and s is the center-to-center spacing of transverse reinforcement. Value P_n is governed by two failure cases, namely, crushing of concrete and buckling of steel reinforcement. From ACI 318-14, P_n is taken as the sum of axial load capacity of concrete and steel reinforcement, with

$$P_n = 0.85 f'_c (A_t - NA - A_s) + f_y A_s, \quad (16)$$

where A_s is the cross-sectional area of longitudinal reinforcement. Both the effects of axial load and bending moment are considered in the axial-bending interaction M_n illustrated for an 8-column TLWD in Figure 2, assuming an equivalent rectangular compressive stress distribution for the concrete and an elasto-plastic constitutive law for steel reinforcement.

In Figure 2, ϵ_{cu} is the ultimate compressive strain of concrete, ϵ_s is the strain of steel reinforcement, ϵ_c is the strain of concrete, ϵ_y is the yield strain of steel reinforcement, β is the ratio of the depth of the rectangular stress block to the neutral axis, C_c is the compression force from concrete, C_s is the compression force from steel reinforcement, and T_s is the tension force from steel reinforcement. The value for M_n given an applied axial load N_u can be calculated using equations (17)–(21) based on the strain distribution under force equilibrium:

$$M_n = \sum_{i=1} A_s^i \epsilon_c^i E_s (c - d_c^i) + C_c (c/2) + \sum_{i=1} A_s^i \epsilon_t^i E_s (d_t^i - c), \quad (17)$$

$$C_s = \sum_{i=1} A_s^i \epsilon_c^i E_s, \quad (18)$$

$$T_s = \sum_{i=1} A_s^i \epsilon_t^i E_s, \quad (19)$$

$$C_c = 0.85 f'_c \beta A_{cb}, \quad (20)$$

$$C_s + C_c = T_s + N_u, \quad (21)$$

where ϵ_c^i and ϵ_t^i are, respectively, the compressive and tensile strain of the reinforcement bar at the i th depth with $\epsilon_s \leq \epsilon_y$,

FIGURE 2: Strain distribution and resultant forces for an 8-column TLWD under (a) pure bending, (b) tension-controlled, (c) compression-controlled, and (d) balanced compression and tension.

d_c^i and d_s^i are, respectively, the depth of the i th reinforcement bar in compression and tension, E_s is the Young modulus of steel, A_s^i is the cross-section area of reinforcement bar at the i th depth, and A_{cb} is the area of compression block.

This TLWD strength model will be validated in Section 4 using data produced by numerical simulations.

2.3. Finite Element Model. The finite element model of a TLWD is constructed to validate the TLWD strength model. The concrete damage plasticity model (CDPM) in ABAQUS is used to simulate the nominal strength of TLWDs. The CDPM considers both tensile cracking and compressive crushing failure mechanisms based on the models proposed by Lubliner et al. [22] and Lee and Fevens [23]. In the CDPM, the five following parameters characterize the stress-strain behavior and damage of inelastic and cracking strain. The dilation angle ψ defines the angle of the inclination surface towards the hydrostatic axis. The eccentricity ϵ defines the shape of the hyperbola and can be calculated as the ratio of tensile strength to compressive strength [24]. Parameter f_{b0}/f_{c0} is the ratio of compressive strength in biaxial direction f_{b0} to uniaxial direction f_{c0} , recommended as 1.16 in [25]. Parameter K_c defines the shape of the cross section at failure in the deviatoric plane, recommended as 0.67 in [25]. The viscosity parameter μ is used to reduce the time interval to prevent divergence. For nonviscoelastic materials such as concrete, the value should be as small as possible [24]. Parameters used in characterizing the CDPM are listed in Table 1.

The uniaxial compressive stress-strain relationship is characterized following Hsu and Hsu model [26]. The stress-strain relationship, plotted in Figure 3(a), solely relies on f'_c , with

TABLE 1: Concrete damage plasticity model parameters.

Parameters	Value	Unit
ψ	31	Degree
ϵ	0.1	—
f_{b0}/f_{c0}	1.16	—
K_c	0.667	—
μ	0.0001	—

$$\sigma_c = \epsilon_c E_c, \quad \text{for } \epsilon_c \leq \epsilon_l,$$

$$\epsilon_l = \frac{0.5 f'_c}{E_c},$$

$$\sigma_c = f'_c \left[\frac{\beta (\epsilon_c / \epsilon_0)}{\beta - 1 + (\epsilon_c / \epsilon_0)} \right], \quad \text{for } \epsilon_c > \epsilon_l, \quad (22)$$

$$\beta = \frac{1}{1 - (f'_c / \epsilon_0 E_c)}$$

$$\epsilon_0 = 1.29 \times 10^{-5} f'_c + 0.00211,$$

where σ_c is the compressive stress, ϵ_l is the compressive strain at the end of linear phase, and ϵ_0 is the compressive strain at peak stress.

The tensile stress-strain relationship of concrete is characterized using Nayal and Rasheed tension stiffening model [27] modified by Wahalathantri et al. [28]. The tensile behavior, plotted in Figure 3(b), is linear until reaching the average splitting strength f_{ct} , corresponding to the cracking strain ϵ_{cr} . After, the stress decreases at various rates after reaching $0.77 f_{ct}$, corresponding to $1.25 \epsilon_{cr}$, $0.45 f_{ct}$, corresponding to $4 \epsilon_{cr}$, and $0.1 f_{ct}$, corresponding to $8.7 \epsilon_{cr}$, the cracking strain ϵ_{cr} and average splitting strength f_{ct} are written as

FIGURE 3: Materials stress-strain relationship: (a) uniaxial compression of concrete, (b) uniaxial tension of concrete, and (c) bilinear curve of steel.

$$\begin{aligned} \epsilon_{cr} &= \frac{f_{ct}}{E_c}, \\ f_{ct} &= 0.56\sqrt{f'_c}. \end{aligned} \quad (23)$$

The constitutive model of steel reinforcement bars is assumed to be bilinear elastic-perfectly plastic with identical stress-strain behavior in tension and compression, as shown in Figure 3(c). f_y , ϵ_y , and E_s are the yield strength, yield strain, and Young's modulus of steel reinforcement accordingly. The bond between reinforcement steel and concrete is assumed to be perfect:

$$\begin{aligned} \sigma_t &= E_s \epsilon_t, & \text{for } \epsilon_t \leq \epsilon_y, \\ \sigma_t &= f_y, & \text{for } \epsilon_t > \epsilon_y. \end{aligned} \quad (24)$$

Concrete and steel reinforcement are modeled using linear 3D 8-node solid and linear 2D 2-node truss elements. The embedded region constraint method is employed, where concrete and steel reinforcement is modeled as host and embedded regions, respectively. The embedded TLMCDs

are modeled as vertical voids occupied by liquid. A mesh sensitivity study is conducted to achieve convergence. The element size of concrete is determined as $50 \times 80 \times 110 \text{ mm}^3$ (width $\times$ length $\times$ height) and the meshing length of steel as 150 mm, with a total of approximately 85,000 elements.

2.4. Numerical Simulation Model. After the TLWD validation using the finite element model, the TLWD will be numerically simulated in the MATLAB environment. The equation of motion for a structure equipped with a TLWD can be written as

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{C}\dot{\mathbf{x}} + \mathbf{K}\mathbf{x} = \mathbf{E}_d\mathbf{F}_d + \mathbf{E}_e\mathbf{F}_e, \quad (25)$$

where $\mathbf{M}$, $\mathbf{C}$, and $\mathbf{K}$ are the mass, damping, and stiffness system matrices, respectively, $\mathbf{x}$ is the displacement vector, the dot denotes a time derivative, $\mathbf{F}_d$ is the damping forces generated by TLWD, $\mathbf{F}_e$ is the excitation force vector, and $\mathbf{E}_d$ and $\mathbf{E}_e$ are the associated force location matrices. The state-space formulation of equation (25) is written as

$$\begin{aligned}
\dot{\mathbf{X}} &= \mathbf{A}\mathbf{X} + \mathbf{B}_d\mathbf{F}_d + \mathbf{B}_e\mathbf{F}_e, \\
\mathbf{X} &= \begin{bmatrix} \mathbf{x} \\ \dot{\mathbf{x}} \end{bmatrix}, \\
\mathbf{A} &= \begin{bmatrix} \mathbf{0} & \mathbf{I} \\ -\mathbf{M}^{-1}\mathbf{K} & -\mathbf{M}^{-1}\mathbf{C} \end{bmatrix}, \\
\mathbf{B}_d &= \begin{bmatrix} \mathbf{0} \\ \mathbf{M}^{-1}\mathbf{E}_d \end{bmatrix}, \\
\mathbf{B}_e &= \begin{bmatrix} \mathbf{0} \\ \mathbf{M}^{-1}\mathbf{E}_e \end{bmatrix},
\end{aligned} \tag{26}$$

where $\mathbf{X}$ is the state vector, $\mathbf{A}$ is the state matrix, and $\mathbf{B}_d$ and $\mathbf{B}_e$ are the input vector. From [13], an N -column symmetric TLWD can be simplified into a linear system having $N/2$ equal pairwise natural frequencies and can be tuned through adjusting column spacings. The system stiffness matrix $\mathbf{K} \in \mathbb{R}^{((N/2)+1) \times ((N/2)+1)}$, mass matrix $\mathbf{M} \in \mathbb{R}^{((N/2)+1) \times ((N/2)+1)}$, and damping matrix $\mathbf{C} \in \mathbb{R}^{((N/2)+1) \times ((N/2)+1)}$ of the coupled SDOF structure can be written as

$$\begin{aligned}
\mathbf{K} &= \begin{bmatrix} k_s & 0 & 0 & \dots & 0 \\ 0 & 2\rho Ag & 0 & \dots & 0 \\ 0 & 0 & 2\rho Ag & \dots & 0 \\ \vdots & \vdots & \vdots & \ddots & 0 \\ 0 & 0 & 0 & \dots & 2\rho Ag \end{bmatrix}, \\
\mathbf{M} &= \begin{bmatrix} m_s + m_d & \rho A \left(2 \sum_{k=1}^{(N/2)} l_k - l_{(N/2)} \right) & \rho A \left(2 \sum_{k=2}^{(N/2)} l_k - l_{(N/2)} \right) & \dots & \rho A l_{N-1} \\ \rho A \left(2 \sum_{k=1}^{(N/2)} l_k - l_{(N/2)} \right) & \rho A \left[v \left(2 \sum_{k=1}^{(N/2)} l_k - l_{(N/2)} \right) + 2h \right] & \rho A v \left(2 \sum_{k=2}^{(N/2)} l_k - l_{(N/2)} \right) & \dots & \rho A v l_{N-1} \\ \rho A \left(2 \sum_{k=2}^{(N/2)} l_k - l_{(N/2)} \right) & \rho A v \left(2 \sum_{k=2}^{(N/2)} l_k - l_{(N/2)} \right) & \rho A \left[v \left(2 \sum_{k=2}^{(N/2)} l_k - l_{(N/2)} \right) + 2h \right] & \dots & \rho A v l_{N-1} \\ \vdots & \vdots & \vdots & \ddots & \vdots \\ \rho A l_{N-1} & \rho A v l_{N-1} & \rho A v l_{N-1} & \dots & \rho A (v l_{N-1} + 2h) \end{bmatrix}, \\
m_d &= \rho l A \left(Nh + \sum_{k=1}^{N-1} \frac{l_k}{v} \right), \\
\mathbf{C} &= \begin{bmatrix} c_s & 0 & \dots & 0 \\ 0 & 0 & \dots & 0 \\ \vdots & \vdots & \ddots & 0 \\ 0 & 0 & \dots & 0 \end{bmatrix},
\end{aligned} \tag{27}$$

where m_d is the liquid mass of the TLMCD and c_s is the structural damping coefficients. It should be noted that the linear damping coefficients of TLMCDs are taken equal to 0

in the linear damping matrix $\mathbf{C}$ because the TLMCD damping force is proportional to the square of velocity (equations (4) and (7)). The state vector $\mathbf{X} = [\mathbf{x} \ \dot{\mathbf{x}}]^T \in$

$\mathbb{R}^{(N+2)}$ is defined for the coupled SDOF system. An explicit discrete-time formulation is used to solve the state-space equation:

$$\mathbf{X}(t + \Delta_t) = e^{\mathbf{A}\Delta_t} \mathbf{X}(t) + \mathbf{A}^{-1} (e^{\mathbf{A}\Delta_t} - \mathbf{I}) [\mathbf{B}_e \mathbf{F}_e(t) + \mathbf{B}_d \mathbf{F}_d(t)], \quad (28)$$

where e is the exponential function and $\mathbf{I}$ the identity matrix.

3. Performance Investigation: Single Wall Section

In this section, the motion and strength performance of the TLWD are preliminarily investigated on a simplified SDOF system and a single RC wall geometry, respectively. The investigation starts by a parametric study on the TLMCD configuration for motion engineering. It is followed by an investigation of embedment strategies to optimize performance versus both motion and strength by comparing the numerical and FEM results. After that, results from the finite element analysis are used to validate the simplified analytical model for strength. Then, the validated model is used to further the study on trade-offs between motion and strength performance.

The SDOF system is based on a representation of a 42-story residential building (Building 1A) [29] equipped with a TLWD. Note that the behavior of the SDOF system does not fully represent that of the multi-degree-of-freedom system (MDOF). The structural mass m_s and fundamental frequency ω_s are obtained from the literature [29]. The structural damping coefficients are obtained assuming 5% natural damping ratio. The SDOF system is schematized in Figure 4 and model parameters are summarized in Table 2. The vertical liquid columns in the attached TLWD are equally spaced, and the first vibration mode is tuned to the fundamental frequency of the structure. The TLWD represents the assembly of numerous TLMCDs occupying an equivalent cross-section A_{eq} .

The strength evaluation assumes the TLMCDs are embedded in an interstory wall at the ground level from Building 1A [29]. The wall, illustrated in Figure 5, has a height $h_w = 4,167$ mm, length $l_w = 5,334$ mm, and width $b_w = 610$ mm. It is reinforced using 1.39% longitudinal and 0.58% transverse reinforcement ratio. The concrete compressive strength f'_c and steel reinforcement yield strength f_y are taken as 55.16 MPa and 413.69 MPa, respectively.

Transfer functions $H_1(\omega)$ and $H_2(\omega)$ are selected as the main motion performance objectives. They correspond to the amplification of displacement and acceleration responses in the frequency domain, respectively. They are evaluated numerically from the steady-state response using

$$\begin{aligned} H_1(\omega) &= \frac{|x_s(\omega)|_{\max}}{(p_0/k_s)}, \\ H_2(\omega) &= \frac{|\ddot{x}_s(\omega)|_{\max}}{(p_0/m_s)}. \end{aligned} \quad (29)$$

FIGURE 4: Schematic of the SDOF system equipped with a TLWD.

TABLE 2: Properties of the SDOF system.

Parameter	Variable	Value	Unit
Structural mass	m_s	38,540	Tons
Structural frequency	ω_s	1.57	rad/s
Structural stiffness	k_s	95,094	kN/m
Structural damping coefficients	c_s	6,054	kN-s/m
TLMCD length	l	5,050	mm
TLMCD liquid height	h	600	mm
Excitation magnitude	p_0	378	kN

FIGURE 5: Cross-section view of the RC wall and embedded location of a single-layered 6-column TLMCD (columns in dashed line).

Nominal shear V_n , flexural M_n , and axial strength P_n are selected as the main strength objectives, simulated through the displacement-controlled pushover analysis at the ultimate stage. For the same wall cross section, slender wall and short wall geometries are used to reach the flexural and shear failure modes separately. Among these, M_n is determined based on balanced axial-moment interaction from strain compatibility analysis with approximated axial load ratio $\rho_{axial} = 0.36$.

3.1. Motion Performance. The performance of the TLMCD is first investigated on single-layered configurations (i.e., all columns organized along the same horizontal line in Figure 5), with the overall geometry constrained to the shear wall's geometry. The parametric studies on the number of columns N and their diameter D are conducted with D expressed as a function of wall thickness b_w . A harmonic excitation of frequency ranging from $0.8\omega_s$ to $1.2\omega_s$ is used. Tuning frequency ratio f and head loss coefficient η are adjusted to obtain the optimal transfer functions $H_1(\omega)|_{opt}$ and $H_2(\omega)|_{opt}$. Both $H_1(\omega)|_{opt}$ and $H_2(\omega)|_{opt}$ over various values for D and N are plotted in Figures 6 and 7, respectively. Tables 3 and 4 list the corresponding mass ratio $\bar{m}$ of the equivalent TLMCD to the structural mass, the optimal tuning frequency $f|_{opt}$, the optimal head loss coefficient $\eta|_{opt}$

FIGURE 6: Transfer functions plot of 6-vertical column TLMCDs with various column diameter: (a) transfer function of displacement and (b) transfer function of acceleration.

FIGURE 7: Transfer functions plot of $0.5b_w$ columns' diameter TLMCDs with various number of columns: (a) transfer function of displacement and (b) transfer function of acceleration.

TABLE 3: Optimal 6-column TLWD parameters under various column diameters D .

D	$\bar{m}(\%)$	$f _{\text{opt}}$	$\eta _{\text{opt}}$	$W_d(KJ)$	$x_i _{\text{max}}(\text{mm})$
$0.21b_w$	0.23	0.993	0.28	19.97	328
$0.26b_w$	0.36	0.992	0.49	23.38	263
$0.33b_w$	0.55	0.989	0.74	26.98	200
$0.45b_w$	1.04	0.984	1.41	31.34	139
$0.5b_w$	1.27	0.982	1.85	32.36	124

TABLE 4: Optimal $0.5b_w$ -column diameter TLWD parameters under various numbers of column N .

N	$\bar{m}$ (%)	$f _{\text{opt}}$	$\eta _{\text{opt}}$	W_d (kJ)	$x_l _{\text{max}}$ (mm)
2	0.71	0.987	10.2	11.91	162
4	0.93	0.985	3.14	21.49	145
6	1.27	0.982	1.85	32.26	124
8	1.62	0.979	1.23	44.02	105
10	1.97	0.976	0.93	56.35	93
12	2.33	0.973	0.74	68.29	82

taken constant throughout the TLMCD and the energy dissipated by cycle W_d , under each D and N , respectively.

Results show that increasing the columns' diameters or numbers has a positive effect on mitigation capabilities, as it would be expected due to the increase in liquid mass. The lower column diameter cases reduce the maximum total damping forces but result in a larger liquid displacement $x_l|_{\text{max}}$ in vertical columns, as shown in Table 3. Similarly, the amplification of energy dissipation is observed from results listed in Table 4 when the column number increases from 2 to 8. This is mainly because the increased mass in the fundamental mode contributes a higher total damping force.

Overall, results confirm that the mitigation performance of the TLWD can be ameliorated by adding liquid mass, but there is a physical limit in doing so because the increased liquid mass yields a decrease in the concrete cross-section area, which will yield a loss in strength. Section 3.2 studies TLMCD-embedded strategies to optimize performance versus both motion and strength.

3.2. Embedment Strategies. A strength performance investigation is conducted under different embedded TLMCD configurations designed to yield similar mitigation performance under various optimal TLMCD configurations. First, single-layer TLMCD configurations are studied. These configurations, designed to arbitrarily maintain $H_1|_{\text{max}}$ and $H_2|_{\text{max}}$ constant at 6.7, along with the shear strength V_n , axial-bending strength M_n , axial strength P_n , and stiffness k of the wall obtained from the finite element analysis, are listed in Table 5. Results reveal that achieving mitigation performance through a distributed column strategy increases the shear strength and the axial-bending strength, but decreases the axial strength and lateral stiffness, yet with a marginally decreasing change in variations. Overall, distributing the TLMCD over 12 columns instead of 6 provoked a decrease in the concrete area A_c of 1.5%, with V_n and M_n increasing approximately by 9.7% and 7.8%, P_n and k decreasing about 3.1% and 1%.

Next, the effect of distributing inertia through a different number of layers of tubes on the TLWD strength is investigated. This study is done on the layered configurations illustrated in Figure 8 (one-, two-, and three-layer configurations), with the TLMCD configurations designed to maintain a constant total cross-section area across each configuration and by optimizing their design to arbitrarily maintain $H_1|_{\text{max}}$ and $H_2|_{\text{max}}$ constant at 7.9.

TABLE 5: TLWD strength performance under single-layer configurations.

D	N	$\bar{m}$ (%)	$H_1 _{\text{max}}$	V_n (10^3 kN)	M_n (10^3 kN-m)	P_n (10^3 kN)	k (10^3 kN/m)
$0.5b_w$	6	1.26	6.7	6.47	109.8	177.4	3,748
$0.45b_w$	8	1.31	6.7	6.83	114.9	175.8	3,808
$0.42b_w$	10	1.37	6.7	6.98	118.3	172.7	3,709
$0.37b_w$	12	1.4	6.7	7.1	118.4	171.8	3,710

Table 6 lists the strength values V_n , M_n , P_n , and k under each configuration. Results reveal that while the axial strength and lateral stiffness remains unchanged due to the constant concrete cross-section area, distributing liquid mass amongst additional layers negatively affect both shear and axial-bending strengths, with the axial-bending strength significantly decreasing (27.46%) between the one- and three-layer configurations. This can be attributed to the larger area occupied by the tubes over the width of the wall, causing stress concentration at the thinnest concrete cross section around middle section of the wall. This is shown by the FEM investigation in Figure 9. In the numerical analysis, concrete cracks were first observed over the middle sections between layers and then propagated to the outer surface. The failure stage was reached right after the formation of vertical sliding cracks. A similar failure pattern was also observed in the one-layer configuration, but, at higher thresholds, it was consistent with Table 6.

Lastly, the effect of distributing individual TLMCDs over the in-plane direction on mitigation performance is examined. The investigated embedded geometries have a constant number of columns, but the arrangements are divided into n TLMCDs as illustrated in Figure 10, where the concrete strength is approximated as constant across all configurations (i.e., ignoring changes in the horizontal tube) and properties are optimized to minimize the maximum value of the transfer functions. Table 7 lists the results. The examination shows that the utilization of a single-TLMCD arrangement yields significantly higher mitigation capability, attributed to the longer horizontal distance between the opposite columns. Because the length of the TLMCD is constrained by the size of the shear wall, the tuning frequency is only dependent on the ratio of column area to horizontal tube area v . Disjointed TLMCDs have a larger $v|_{\text{opt}}$, resulting in a less combined liquid mass.

4. Design of TLWD

In this section, results from the performance investigation presented in the previous section are used to validate the analytical TLWD strength model. After, the analytical TLWD strength model is used to further investigate trade-offs between motion and strength performance when designing a TLWD. Lastly, possible design strategies based on findings are discussed.

4.1. TLWD Strength Model Validation. The analytical model developed in Section 2.2 is validated using FEM data from

FIGURE 8: TLWD cross-section view of multiple-layer embedded TLMCD: (a) one-layer 4-column TLWD, (b) two-layer 4-column TLWD, and (c) three-layer 4-column TLWD.

TABLE 6: TLWD strength performance under multiple-layer configurations.

No. of layers	D	N	$\bar{m}$ (%)	$H_1 _{\max}$	V_n (10^3 kN)	M_n (10^3 kN-m)	P_n (10^3 kN)	K (10^3 kN/m)
1	$0.34b_w$	4	0.43	7.84	8.61	134.4	198.6	4,277
2	$0.23b_w$	4	0.43	7.84	7.95	115.5	198.8	4,255
3	$0.19b_w$	4	0.43	7.84	7.64	97.5	198.8	4,242

FIGURE 9: Concrete tension damage distribution of TLWD with three-layered embedment profile (DMAGET indicates tension damage of the concrete).

FIGURE 10: One-layer vertical column arrangement organized into (a) one 16-column TLMCD, (b) two 8-column TLMCDs, and (c) four 4-column TLMCDs.

TABLE 7: Results from distributed TLMCD investigation.

D	n	N	$\bar{m}(\%)$	$f _{\text{opt}}$	$v _{\text{opt}}$	$\eta _{\text{opt}}$	$H_1 _{\text{max}}$	$H_2 _{\text{max}}$
$0.33b_w$	1	16	1.47	0.981	0.41	0.25	6.64	6.68
$0.33b_w$	2	8	0.8	0.989	1.42	0.28	8.04	8.07
$0.33b_w$	4	4	0.62	0.993	4.66	0.28	8.94	8.88

TABLE 8: Structural strengths obtained by the FEM and analytical model.

N	D	V_n (10^3 kN)			M_n (10^3 kN-m)			P_n (10^3 kN)			k (10^3 kN/m)		
		FEM	Analytical	Diff (%)	FEM	Analytical	Diff (%)	FEM	Analytical	Diff (%)	FEM	Analytical	Diff (%)
8	$0.5b_w$	6.28	6.15	+2.1	105	106	-1.0	169	144	+14.9	3,563	3,767	-5.7
8	$0.45b_w$	6.84	6.74	+1.5	114	108	+5.6	176	149	+15.3	3,808	3,754	+1.4
10	$0.45b_w$	6.56	6.15	+6.3	114	106	+7.4	169	143	+15.4	3,656	4,081	-11.6
12	$0.33b_w$	7.59	7.29	+4.0	126	109	+13.3	182	153	+15.6	3,833	4,057	-5.8
16	$0.26b_w$	7.12	6.41	+10	115	110	+4.2	186	156	+16.3	4,104	4,143	-1.0
2	$0.21b_w$	9.57	9.33	+2.5	138	114	+16.5	200	170	+15.1	4,545	4,551	-0.1

the one-layer embedment profile investigation. Results from the FEM and analytical model are summarized in Table 8 with configurations ranked by strength obtained from the FEM, where a positive sign in the difference represents a conservative estimation of strength by the analytical model. While sometimes overestimating the lateral stiffness, the analytical model generally yields a conservative estimate of the strength capacities. This can be attributed to the wall geometries in the FEM being fully fixed at the bottom of the footings, resulting in a not fully constrained boundary condition at the bottom of the wall sections. Furthermore, the model assumes that concrete does not have tensile resistance and that concrete crushes at $\varepsilon_c = 0.003$, along with the approximation of parabolic stress distribution. An approximately constant 15% difference in axial capacity can be observed, attributable to the maximum concrete stress taken as $0.85 f'_c$. Nevertheless, the analytical model generally maintains a similar rank in strength, which can be useful in the design stage. Overall, accuracy of the strength model is deemed acceptable to perform further investigations that would be time-consuming using FEM, in particular for the MDOF application presented in Section 4.2.

4.2. Motion-Strength Trade-offs. The performance of the TLWD versus both motion and strength is investigated under a wide range of configurations using the validated analytical strength model. Under each configuration, the embedded TLMCD is tuned to minimize the maximum transfer function (i.e., keeping both $H_{1,\text{max}}$ and $H_{2,\text{max}}$ approximately equal), here assembled under the performance metric $H_{1,2,\text{max}}$. From the findings in Section 4.1, the study is limited to single-layer and single-TLMCD arrangements. The axial-bending strength M_n is obtained at the balanced axial-bending interaction $\rho_{\text{axial}} = 0.36$. Figure 11 plots the results of the multiconfiguration investigation, where a reduction in $H_{1,2,\text{max}}$ in Figure 11(a) is a desired feature, while that of the strength in Figures 11(b)–11(e) is not. One can observe an obvious trade-off between

motion and strength performance. However, the gain in motion reduction is marginally decreasing while the loss in strength is marginally increasing with N and Db_w increasing, showing that motion performance can be substantially increased through design of the TLWD system while minimizing the adverse effects on strength. Shear strength V_n is the most sensitive to variations in configurations, with a total variation of 47% in strength amongst all configurations, while other strength components M_n , P_n , and k only vary maximally by 12%, 24%, and 26% respectively.

4.3. Design Considerations. Results from the parametric investigation presented in Section 4.2, along with the extended investigation conducted using the validated strength model, point to the following considerations in designing a TLWD system. First, mitigation performance is generally achieved by adding liquid mass in the TLWD and implementing TLMCD designs with longer horizontal distances between opposite columns yields better performance. Second, in order to satisfy strength requirements, the TLMCD system should be arranged in a single layer within the wall if possible because distributing liquid mass along the width of the wall has a significant adverse effect on shear and axial-bending strength. Third, the use of many smaller columns is advantageous over fewer, larger columns in the constrained TLWD environment with respect to strength, except for axial strength due to the provoked decrease in the concrete cross-section area over similar mitigation performance. Fourth, the proposed analytical strength model could be used to quickly investigate various design configurations, whereas the model will generally yield conservative estimates with respect to strength. Fifth, there is a trivial trade-off between motion and strength performance in designing a TLWD, but altering the geometry of the TLMCD does not affect both motion and strength performance proportionally, therefore pointing towards an equilibrium point depending on the design requirements. Sixth, when tuning the geometry of the TLMCD system, one must pay particular attention to the shear strength, as it is highly sensitive to the

FIGURE 11: Surface plot of percentage reduction in performance objectives: (a) maximum of displacement/acceleration transfer functions, (b) nominal shear strength, (c) nominal flexural strength under balanced axial interaction, (d) nominal axial strength, and (e) lateral stiffness.

selected configuration and constitute a key design property in the design of a shear wall.

5. High-Rise Building Example

In this section, the performance of the TLWD is numerically validated on a realistic high-rise building subject to stochastic wind excitations. Motion and strength performance is assessed, with strength analyzed using the proposed analytical model.

5.1. Numerical Model. A 42-story RC residential building (Building 1A) [29] is used for the simulations. It consists of six centrally located core walls along east-west direction surrounded on the perimeter by concrete columns. The walls have constant lengths, but their width varies along the height of the structure. Geometries and material properties and reinforcement details can be found in literature [29]. Here, the east-west direction of the building is simulated as a lumped-mass shear system. The fundamental damping ratio of the building is taken as 2% with stiffness proportional

TABLE 9: Dynamic properties of 42-story building.

Floor	Height (m)	Mass (10^3 kg)	Stiffness (10^3 kN/m)	Floor	Height (m)	Mass (10^3 kg)	Stiffness (10^3 kN/m)
1	4.16	1189	11,492	22	3.25	893	946
2	3.25	1099	7,861	23	3.25	893	883
3	3.25	1099	6,225	24	3.25	893	821
4	3.25	1099	4,532	25	3.25	843	762
5	3.25	1099	3,677	26	3.25	843	741
6	3.25	1041	3,155	27	3.25	843	721
7	3.25	1041	2,797	28	3.25	843	702
8	3.25	1041	2,536	29	3.25	810	685
9	3.25	1041	2,336	30	3.25	810	669
10	3.25	1041	2,177	31	3.25	810	654
11	3.25	1000	1,970	32	3.25	810	640
12	3.25	1000	1,784	33	3.25	810	624
13	3.25	1000	1,615	34	3.25	777	609
14	3.25	1000	1,460	35	3.25	777	594
15	3.25	1000	1,315	36	3.25	777	580
16	3.25	934	1,256	37	3.25	777	567
17	3.25	934	1,205	38	3.25	777	557
18	3.25	934	1,159	39	3.25	777	547
19	3.25	934	1,118	40	3.25	777	538
20	3.25	934	1,081	41	3.25	777	530
21	3.25	893	1,012	42	3.25	872	521

damping. The dynamic properties of the building used in the simulation are tabulated in Table 9.

The simulated wind speed consists of two components, the steady mean speed V_m and the fluctuating speed V_f , due to aerodynamic turbulence:

$$V(z, t) = V_m(z) + V_f(z, t). \quad (30)$$

The steady mean speed is determined based on the wind profile power law [30] as follows:

$$V_{\text{terrain}} = V_0 \frac{v_*}{v_0} \frac{\ln(10/z_*)}{\ln(10/z_0)}, \quad (31)$$

$$V_m(z) = V_{\text{terrain}} \frac{\ln(z/z_*)}{\ln(10/z_*)},$$

where V_0 is the 3-second gust speed obtained from hazard maps in ASCE 7-16 [31], V_{terrain} is the mean velocity for a given terrain, v_* and v_0 are the shear velocities of the building site and open terrain, and z_* and z_0 are the surface roughness for the building site and open terrain, respectively. A multivariate stochastic Gaussian process with cross-spectral density function is used for modeling turbulence [32] as follows:

$$S_{ij}(\Omega) = \begin{cases} S_k(z_i, \Omega), & i = j, \\ \sqrt{S_k(z_i, \Omega)S_k(z_j, \Omega)} C_{ij}(z_i, z_j, \Omega), & i \neq j, \end{cases} \quad (32)$$

$$S_k(z, \Omega) = \frac{50}{\pi} v_*^2 \frac{z}{V_m(z)} \frac{1}{(1 + 50(\Omega z / 2\pi V(z)))^{(5/3)}}, \quad (33)$$

where S_k is the Kaimal power spectral density function in along-wind direction, Ω is the excitation frequency, and z is

the height of floors. The coherence function between i th and j th locations in equation (32) is defined as

$$C_{ij}(\Omega) = \exp\left(-\frac{10\Omega\Delta z}{\pi(V_m(z_i) + V(z_j))}\right). \quad (34)$$

The cross-spectral density matrix, $\mathbf{S}(\Omega)$, containing cross-spectral density functions at different heights, is decomposed using Cholesky's decomposition:

$$\mathbf{S}(\Omega) = \mathbf{H}(\Omega)\mathbf{H}^{*T}(\Omega), \quad (35)$$

where $\mathbf{H}(\Omega)$ is a lower triangular matrix with generally complex off-diagonal elements. Once the matrix is decomposed, the stochastic fluctuating wind speed $V_f(z_i, t)$ can be obtained by the following series:

$$V_f(z_i, t) = 2 \sum_{k=1}^n \sum_{l=1}^{N_w} |H_{ik}(\Omega_{kl})| \cdot \sqrt{\Delta\Omega} \cos(\Omega_{ik}t - \theta_{ik}(\Omega_{kl}) + \Phi_{kl}), \quad (36)$$

where Φ_{kl} is the random phase ranged from 0 to 2π ; the phase $\theta_{ik}(\Omega_{kl})$ and the double-indexing frequency Ω_{kl} is given by

$$\theta_{ik}(\Omega_{kl}) = \tan^{-1}\left(\frac{\text{Im}(H_{ik}(\Omega_{kl}))}{\text{Re}(H_{ik}(\Omega_{kl}))}\right),$$

$$\Omega_{kl} = \left(l - \frac{N_\Omega - l}{N_\Omega}\right)\Delta\Omega, \quad (37)$$

$$\Delta\Omega = \frac{\Omega_c}{N_\Omega},$$

where Ω_c is an upper cutoff frequency and N_Ω is the total number of random frequency points.

FIGURE 12: Typical wind hazard realization at the top floor of the building: (a) wind speed and (b) wind load time series.

The wind load is converted from wind speed to concentrated forces applied on the building floors [30] using

$$F_w(z, t) = \frac{1}{2} \rho V^2(z, t) A_p C_D, \quad (38)$$

where ρ is the air density, A_p is the projected area exposed to the wind pressure, and C_D is the drag coefficient of the structure. Across-wind forces and vortex shedding are neglected for simplicity, and it is assumed that the along-wind response of the structure dominates motion. A total of 20 wind realizations are produced taking a suburban region based on occupancy category II (residential building) [31] with 7% probability of exceedance in 50 years. Figure 12 plots a typical wind realization speed V (Figure 12(a)) and associated load F_w (Figure 12(b)) time histories occurring at the top floor of the building using the input parameters listed in Table 10.

5.2. TLWD Design. In the following numerical investigations, three optimally tuned TLWD configurations are considered: TLWD1 has a 2-column TLMCD of column cross-section diameter $D = 0.5b_w$, TLWD2 has a 6-column TLMCD of column cross-section diameter $D = 0.5b_w$, and TLWD3 has a 12-column TLMCD of column cross-section diameter $D = 0.37b_w$. TLWD1 is selected to mimic a distributed TLCDs system, TLWD2 is selected to improve on TLWD1 through added inertia, and TLWD3 is selected from Table 5 and to perform similar to TLWD2 for motion mitigation, yielding minimal strength reduction. The simplified strength model is used to calculate the strength reduction in parameters V_n , M_n , P_n , and k caused by the integration of the TLMCDs. In this numerical example, TLWDs are vertically distributed within the top 20 floors (from the 23rd to the 42nd floor inclusively). The shear wall parameters including the wall width b_w , longitudinal reinforcing ratio ρ_w , and transverse reinforcing ratio ρ_t are listed in Table 11 along with the resulting reductions in strength.

A comparison of strength reductions shows that TLWD1 yields a significantly stronger shear wall, except for the axial-

TABLE 10: Parameters for the stochastic wind model.

Parameters	Value	Unit
V_0	49	m/s
z_0	0.07	—
z^*	0.3	—
Ω_c	20π	rad/s
N_Ω	2^{13}	—
ρ_{air}	1.225	kg/m ³
C_D	1.4	—

bending strength compared with TLWD3. However, the effect of embedding a TLMCD on the axial-bending strength is not substantial, with a maximum reported reduction of 5.1%. Configuration TLWD3 does yield an improvement in strength relative to TLWD2. Nevertheless, the differences in strength reductions between both configurations are all within 3.6%. Overall, adding more column cross-section areas within the shear wall (TLWD1 versus TLWD2) does have an adverse effect on strength, while distributing the cross-section areas with the wall (TLWD2 versus TLWD3) improves on strength, consistent with findings in Section 5.1.

5.3. Motion Performance. For the numerical investigation on motion performance, it is assumed that all TLWDs have identical geometries and dynamic properties, and they are modeled as a single equivalent TLWD at each floor, with a maximum of three TLMCDs per interstory wall. The placement of TLWD is first investigated using TLWD2 configuration and selected because it adopts the geometry of a TLCD (TLWD1) but with added inertia. Four vertical distributions are considered, with the TLWDs installed over the top 5, top 10, top 15, and top 20 floors. Figure 13 plots the fitted normal distributions of the maximum structural responses under the 20 wind realizations. Table 12 lists the average drift μ_d and acceleration responses μ_a , the standard deviation of the drift σ_d and acceleration responses σ_a , and the probability of exceeding the acceleration threshold P_a . Results show that, for this particular building, increasing the

TABLE 11: Strength reduction (%) of the TLWDs compared to the original wall design.

Floor	b_w (mm)	ρ_w (%)	ρ_t (%)	TLWD1				TLWD2				TLWD3			
				V_n	M_n	P_n	k	V_n	M_n	P_n	k	V_n	M_n	P_n	k
23–25	610	2.4	0.9	23.9	4.9	6.1	15.1	38.3	4.9	18.7	24.4	36.2	3.5	20.5	23.9
26–30	533	2.7	0.6	26.1	5.1	7.2	17.3	45.0	5.1	21.5	27.9	41.4	4.2	23.2	27.0
31–36	533	1.8	0.6	26.1	4.2	7.4	17.3	45.0	4.2	22.1	27.9	41.4	2.7	23.9	27.0
37–42	533	1.4	0.6	26.1	3.4	7.4	17.3	45.0	3.4	22.2	27.9	41.4	2.1	24.1	27.0

FIGURE 13: Distribution of maximum absolute responses under various vertical TLWD arrangements: (a) interstory drift ratio and (b) acceleration, with the dashed vertical line showing the performance threshold.

TABLE 12: Performance of TLWD2 under various vertical arrangements.

	μ_d (mm)	σ_d (mm)	μ_a (mg)	σ_a (mg)	P_a (%)
Uncontrolled	10.7	1.3	36.6	6.02	28.6
Top 5-floor	10.1	1.2	32.9	5.06	8.03
Top 10-floor	10.1	1.2	31	4.55	2.4
Top 15-floor	9.8	1.2	29.9	4.28	0.9
Top 20-floor	9.8	1.2	29.6	4.18	0.6

number of TLWDs over more floors yields marginally decreasing gains in acceleration mitigation and the utilization of TLWDs does not significantly affect interstory drift. In the case of acceleration control, the use of TLWDs over at least the last ten floors is necessary in ensuring that the structure does not significantly exceed the acceleration threshold of 40 mg, where 40 mg corresponds to an acceptable motion performance for essential structures under rare winds [33]. It can also be noted that performance under drift is always satisfied, even in the uncontrolled case. From these results, arrangement of TLWDs over the top 15 floors is selected to continue the numerical investigation.

Mitigation performance for TLWDs1-3 is now assessed against that of a conventional TLCD installed at the top of

the building. To enable fair comparison, the TLCD's capacity is equal to that of TLWD1 by constraining its geometry, where its horizontal length is equals to that of one embedded TLMCD, and the area of its vertical columns is equal to the sum of all of the single column areas of each TLMCD. Note that such TLCD geometry may not be practical and is simply used to benchmark performance. Figure 14 plots the fitted normal distributions of the maximum structural responses under the 20 wind realizations, and Figure 15 plots the response profiles using the averaged maximum structural responses. Table 13 lists the average drift μ_d and acceleration responses μ_a , the standard deviation of the drift σ_d and acceleration responses σ_a , and the probability of exceeding the acceleration threshold P_a .

Results show that configurations TLWD2 and TLWD3 successfully reduce the mean maximum acceleration of the uncontrolled case by 18% and slightly outperform the TLCD case by approximate 3%, yet not necessitating large space at top of the building. A cross-comparison amongst TLWDs reveals that TLWD2 and TLWD3 slightly outperform TLWD1 by approximate 4.2% in terms of acceleration mitigation. However, this improvement in mitigation keeps the structure's maximum acceleration motion under the performance threshold at least 99% of the time. An

FIGURE 14: Distribution of maximum absolute responses using TLWDs arranged over the last 15 floors: (a) interstory drift ratio and (b) acceleration.

FIGURE 15: Mean maximum floor response profiles for TLWDs installed over the last 15 floors: (a) zoom on maximum interstory drift and (b) maximum acceleration over top 20 floors.

TABLE 13: Motion performance for TLWDs installed over the last 15 floors.

	μ_d (mm)	σ_d (mm)	μ_a (mg)	σ_a (mg)	P_a (%)
Uncontrolled	10.7	1.3	36.6	6.02	28.6
TLWD1	10.1	1.2	31.2	4.35	2.2
TLWD2	9.8	1.2	29.9	4.28	0.9
TLWD3	9.8	1.2	30	4.18	0.8
TLCD	10.1	1.2	30.8	4.27	1.6

inspection of the floor drift and acceleration profiles also shows that TLWD2 and TLWD3 outperform TLWD1 and the TLCD. This enhancement in performance, in particular, with respect to TLWD1, is consistent with the findings in Section 5.2.

6. Conclusion

In this study, a multifunctional shear wall, termed tuned liquid wall damper (TLWD), capable of vibration mitigation was investigated. The TLWD consists of a tuned liquid multiple column damper (TLMCD) system integrated within a reinforced concrete shear wall component. The objective of the paper was to evaluate trade-offs between motion and strength performance in designing a TLWD. To empower fast evaluation of strength properties on multi-degrees-of-freedom representations, a simplified analytical strength model was proposed based on ACI 318–14 and validated against finite element model simulations conducted on the simplified representation of a 42-story building. The finite element analysis combined with numerical simulations of the simplified representation was used to establish design considerations. In particular, it was found that distributing liquid mass among a higher number of single columns over a single-layer configuration was preferable in minimizing the reduction in strength while providing enhanced mitigation.

After that, numerical simulations were conducted on a more realistic representation of the 42-story building subjected to 20 stochastic wind loads. First, TLWDs were sequentially added starting from the top floor of the structure to address the mitigation performance on the vertical distribution of TLWDs. The results showed a decreasing marginal gain in mitigation performance. Further simulations were conducted comparing three different TLMCD configurations installed on the last 15 floors. Results showed that adding liquid inertia through additional columns (TLWD1 versus TLWD2) decreased strength while increasing mitigation capabilities and distributing the liquid inertia through additional, smaller columns (TLWD2 versus TLWD3) increased strength while also increasing mitigation capabilities, thus confirming design considerations established through the simplified representation. The use of TLWDs showed important gain in mitigation relative to an uncontrolled case, in particular, for acceleration mitigation. TLWDs also outperformed, yet not significantly, a geometrically constrained TLCD installed at the top of the structure occupying a notably larger space. The use of a distributed liquid inertia (TLWD3) exhibited the best mitigation performance.

Overall, this study demonstrated the potential of the TLWD system as a passive energy mitigation system. While a distributed TLWD system utilizes a higher total liquid mass compared to a conventional TLCD installed at the top floor, its advantage resides in constraining the energy dissipation system with an existing structural component, therefore eliminating the need to utilize a dedicated area in the build as long as structural strength can be ensured. The proposed TLWD is practically feasible due to the low

maintenance requirements, cost-effectiveness, and high mechanical robustness characteristics of TLMCDs. The performance of the TLWD system could be potentially improved by optimizing designs at each floor or by integrating semiactive capabilities by actively controlling the orifice block ratios. These investigations are left to future work.

Data Availability

The data that support the findings of this study are available from the corresponding author upon request.

Disclosure

Any opinions, findings, and conclusions or recommendations expressed in this study are those of the authors and do not necessarily reflect the views of the National Science Foundation.

Conflicts of Interest

The authors declare no conflicts of interest.

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