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Vibration Control of Downtown Toronto High-Rise Development

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Abstract—In this industry-academia collaboration, a multi-platform hybrid numerical simulation was performed to assess the effectiveness of a tuned liquid damper (TLD) to be installed in a high-rise condominium located in Downtown Toronto. The structure was modeled using OpenSees, while the nonlinear TLD model was developed in MATLAB. A dynamic analysis was performed using statistically-generated dynamic wind loads. It was determined that the TLD improved the performance of the structure (storey acceleration, roof drift) for various levels of wind loading, improving the comfort of the residents of the building. Minimal benefit was observed for reducing base shear as well as storey shear demands on the structure.

Keywords-tuned liquid damper; vibration control; structural control; hybrid simulation; wind engineering

I. INTRODUCTION

High-rise structures are flexible structures, and are highly sensitive to dynamic excitation – even vibration caused by ordinary wind loading may be problematic from the viewpoint of serviceability and comfort of occupants. Tuned-liquid dampers (TLDs) are passive vibration absorbers which are used to control wind-induced vibrations of tall structures. The TLD's properties are tuned to the system's fundamental oscillation frequency such that the liquids sloshing action dampens the building's oscillations. In this collaboration, a multi-platform tool was developed to accurately and precisely model the components required to understand the structure-liquid interaction. The structure was modeled using OpenSees [1], while the TLD was modeled in MATLAB [2] using Yu's model. Lastly, the dynamic wind loading applied to the structure was developed using NatHaz Online Wind Simulator (NOWS) web-based tool [3]. The details of each portion of the multi-platform tool are given below.

II. DEVELOPMENT OF MULTI-PLATFORM TOOL

A. Structural Model

OpenSees, an object-oriented open source software framework, was used to model the tower. The structural model which was developed for the central tower of the 90 Harbour Street development from the 6th storey upwards – since the podium levels and underground parking garages are shared between the two towers, they were assumed to provide a rigid base for each of the individual towers. As such, the base nodes of the structure were modeled as fixed nodes in OpenSees.

The structure was modeled using fibre section elements, wherein the sections were varied based on cross-sectional area, level of horizontal and vertical reinforcement, and concrete strength. Due to the complexity of the structure, and the large number of individual element properties to consider, the tower was split into seven categories (by storey), varying along the height of the structure. The individual member properties were assumed to remain constant in each section of the building.

Lastly, the floor slabs were not modeled directly in OpenSees. Rather, they were assumed to provide a rigid support at each floor level. The EqualDOF command in OpenSees was used, wherein each node at a given floor level is forced to have the same displacements as the designated master node at that floor. While the structural properties of the floor slab were not modeled, the mass of the slabs was still accounted for, with the mass being distributed to the nodes by tributary areas.

B. TLD Model

TLDs dissipate energy through liquid boundary layer friction, free surface contamination, and wave breaking. With the horizontal component of the liquid velocity related to the wave motion, wave crests descend as amplitude of motion increases, and simple linear models are no longer adequately able to describe the liquid behaviour [4].

1) Theory of Yu's Model: Using shallow water wave theory, the TLD can be modeled as an equivalent nonlinear-stiffness-damping (NSD) system [5][6]. The behaviour of the NSD system is matched to that of the real TLD using an energy dissipation matching procedure, where the NSD parameters m_d,c_d,k_d are determined by introducing the interaction force created by the TLD liquid sloshing inside the tank.

The energy dissipation inside the tank can be found using

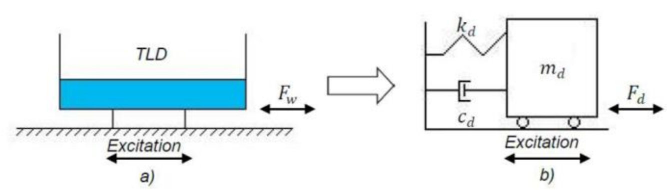

Figure 1: Schematic representation of (a) TLD and (b) equivalent NSD model.

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$$E_w = \int_{T_s} F_w dx \tag{1}$$

where F_w is the force generated by the liquid sloshing inside the tank. The non-dimensionalized energy can be determined:

$$\acute{E}_w = \frac{E_w}{0.5 m_w (A\omega)^2} \tag{2}$$

The non-dimensionalized energy dissipation parameter for the NSD model is determined using the behaviour when subjected to base excitation with frequency β:

$$\acute{E}_d = 2\pi |F_d| \sin\theta \tag{3}$$

where the non-dimensionalized magnitude $|F_d|$ and phase angle θ can be found using:

$$|F_d| = \frac{\sqrt{(1 + (4\zeta_d^2 - 1)\beta^2)^2 + 4\zeta_d^2\beta^6}}{1 + (4\zeta_d^2 - 2)\beta^2 + \beta^4} \tag{4}$$

$$\theta = \tan^{-1}\left(\frac{2\zeta_d\beta^3}{-1 + (1 - 4\zeta_d^2)\beta^2}\right) \tag{5}$$

where $\zeta_d = \frac{c_d}{c_{cr}}$ is the damping ratio of the NSD model. The non-dimensionalized energy dissipation parameters are matched together using a least-squares method, analyzing the results through two parameters: the frequency shift ratio $\xi = \frac{f_d}{f_w}$ and the stiffness hardening ratio $\kappa = \frac{k_d}{k_w}$. The frequency of the water in the tank is determined using:

$$f_w = \frac{1}{2\pi}\sqrt{\frac{\pi g}{L}\tanh\left(\frac{\pi h_w}{L}\right)} \tag{6}$$

The stiffness of the water in the tank can then be determined:

$$k_w = m_w(2\pi f_w)^2 \tag{7}$$

In order to calculate the restoring force that the TLD applies to the structure, the following equations were used:

$$F_{TLD} = k_d x_{TLD} + c_d \dot{x}_{TLD} \tag{8}$$

where x_{TLD} and $\dot{x}_{TLD}$ are the displacement and velocity of the TLD determined from the OpenSees model, and k_d and c_d are the NSD stiffness and damping parameters, determined from the energy matching procedure as follows:

$$k_d = 2.52\left(\frac{x_{TLD}}{L}\right)^{0.25} m_d(2\pi f_w)^2 \tag{9}$$

$$c_d = 0.52\left(\frac{x_{TLD}}{L}\right)^{0.35} 2(k_d m_d)^2 \tag{10}$$

2) Verification of Yu's Model: To justify the use of Yu's model in this simulation, results from the MATLAB model were compared with previously performed experiments on TLDs [4]. The TLD which was tested had measurements of 464 mm x 305 mm, and a water height of 40 mm. The corresponding tank frequency was 0.667 Hz, and the weight of the water in the tank was 5.64 kg. The TLD was subjected to a sinusoidal displacement history with amplitude of 20 mm, and the results are shown in Figure 2.

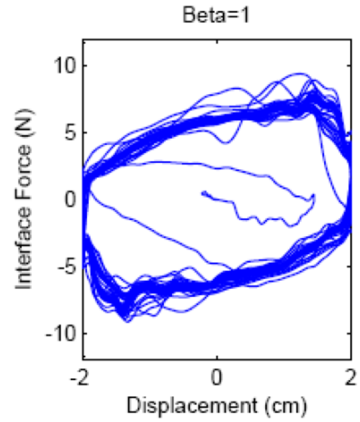

Figure 2: Experimental results for TLD force-displacement behaviour. TLD dimensions: 464 mm x 305 mm, water height 40 mm.

The same TLD was then modeled in Matlab using Yu's model. In this implementation, a simple SDOF structure (portal frame) was modeled with the TLD at the roof level. A sinusoidal force was applied to the portal frame which resulted in a TLD displacement of 20 mm, as in the experimental setup. This system was analyzed using the Newmark-beta method, and results are shown in Figure 3.

The results show that the MATLAB model matches the experimental data well – the shape of the force-displacement curve is similar, and the magnitude of the force is similar as well. Since the shallow water limit $\left(\frac{h_w}{L} < 0.1\right)$ is met for both experimental and numerical TLD models, we can conclude that Yu's model is able to accurately capture the experimental TLD behaviour for the shallow water scenario.

C. Dynamic Wind Loading Simulator

The dynamic wind loading patterns for the structure were obtained using the NatHaz Online Wind Simulator (NOWS) web-based interface. The tool provides users with an on-line simulation of stationary Gaussian multivariate wind fields.

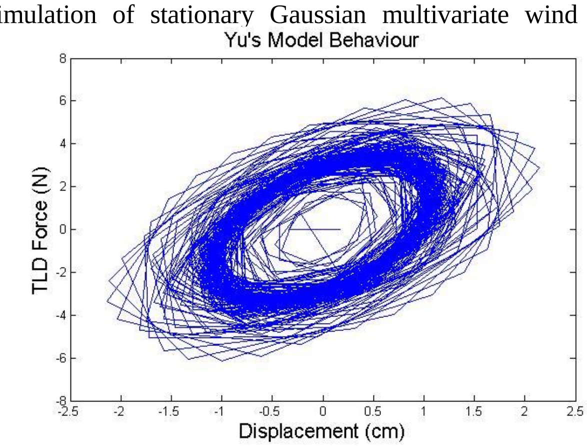

Figure 3: Matlab model results for TLD behaviour. TLD dimensions same as for experimental specimen in Figure 2.

Random wind fields of varying intensities, corresponding to various National Building Code of Canada 2010 (NBCC) limit states [7], were generated using a discrete frequency function with Cholesky decomposition and Fast Fourier Transform (FFT). The outputs of the tool were a time history of wind speeds at the various heights of the structure – a sample output from the tool is shown in Figure 4. These wind speeds were then converted to equivalent forces using the guidelines given in ISO-19902, which gives:

$$F_w = \frac{1}{2}\rho_a (U_w)^2 C_s A \tag{11}$$

where ρ_a is the mass density of air, U_w is the given wind speed in $\frac{m}{s}$, C_s is the shape coefficient of the structure, A is the tributary cross-sectional area of the wind-facing surface, and F_w is the equivalent lateral force.

D. *Multi-Platform Model Algorithm*

The methodology and algorithm that was used to define the interaction between all components of the tool is shown in Figure 5. This interaction loop was repeated for each time step that the wind loading was simulated. The outputs of interest were taken from the Matlab model and the OpenSees model, and included a time history of TLD force as well as structural displacements, acceleration, storey and base shears, as well as element forces.

E. *Solution Procedure*

The model was analyzed using a transient analysis method, and the Newton numerical method. While the details of this solution method are not given in this paper, further details can be found in literature [8].

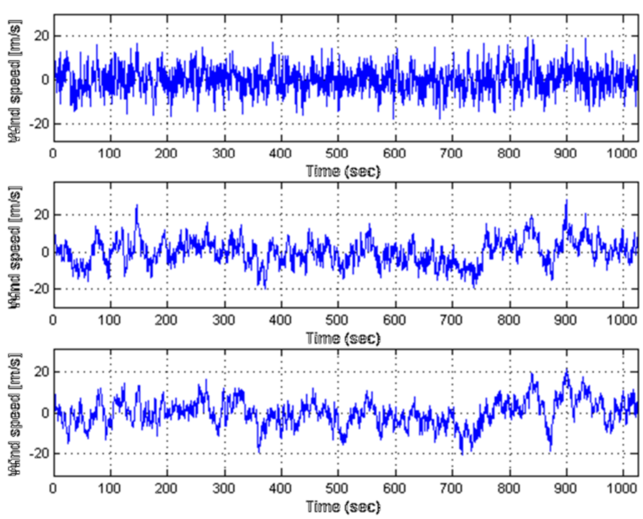

Figure 4: Fluctuating wind speed (about mean wind speed) at Floors 1, 34, and 67 for a sample wind loading scenario obtained from NOWS web tool.

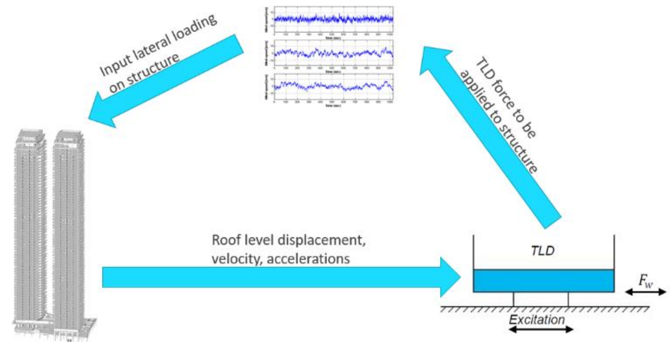

Figure 5: Schematic representation of interaction algorithm used in TLD-structure simulations.

Table 1: Comparison of periods of vibration (ETABS vs. OpenSees) for dominant modes of vibration.

	ETABS Model	**OpenSees Model**
Mode 1 Period	6.166 sec	6.204 sec
Mode 1 Direction	E-W	E-W
Mode 2 Period	4.970 sec	3.738 sec
Mode 2 Direction	N-S	N-S

The results show that the first mode period is quite well-matched between the two models. This is important as the structure was analyzed in the direction of the first mode (E-W).

F. *Description of Wind Loading Profiles*

Table 2 shows the three wind loading profiles which were chosen for analyzing the structure with and without the TLD. These wind loading profiles were selected in order to analyze the structure and be able to compare the results to NBCC 2010 human comfort and structural integrity checks. Scenario 1 corresponds to a 1/10 level of wind, which is recommended by the NBCC 2010 for checking serviceability comfort levels as well as structural performance. Scenario 2 corresponds to the recommended 1/50 wind level, and Scenario 3 represents a more severe wind storm.

Table 2: Summary of wind loading conditions.

	Scenario 1	**Scenario 2**	**Scenario 3**
6th floor wind speed	12.1 m/s	35.5 m/s	53.2 m/s
30th floor wind speed	20.4 m/s	45.4 m/s	68.2 m/s
Roof wind speed	23.1 m/s	51.4 m/s	77.2 m/s

The structure was analyzed under various performance indicators, including peak storey accelerations. These values were compared to the NBCC 2010 acceleration limit for human comfort, as well as the limit suggested by the Council for Tall Buildings and Urban Habitat (CTBUH).

In NBCC 2010, several levels of service for the performance of the building under wind loading are considered – specifically, Commentary I sentence 78 requires that the maximum acceleration be less than 1.5% of g, as any higher level of acceleration is perceptible to occupants (barring additional effects such as visual cues, body position, and orientation). This acceleration guideline is the same in CTBUH, with the added restriction that the Root-Mean-Square (RMS) acceleration should be less than 0.5% of g. Furthermore, NBCC 2010 Commentary I sentence 74 states that the lateral deflection of the building should be no more than 1/500 of the building height [10].

G. Discussion of Results

Table 3: Summary of key structural performance parameters with TLD (and % change compared to uncontrolled structure).

Parameter	Scenario 1	Scenario 2	Scenario 3
Peak roof drift (m)	16.8 (-12%)	172 (-20%)	395 (-21%)
Peak storey acceleration (% g)	0.69 (-12%)	9.22 (-2%)	24.8 (-0%)
Peak Base Shear (MN)	1.7 (-1.1%)	24.1 (-0.2%)	65.6 (-0.1%)

Using the multi-platform tool previously described, it was determined that for the 1/10 level of wind, the structure met the performance criteria set out in both NBCC 2010 as well as CTBUH – the storey accelerations were well below the NBCC 2010 1.5% of g guideline, and the RMS acceleration was 0.507% of g. It is important to note that in the CTBUH guidelines, a 6-year level of wind was recommended when checking RMS acceleration limits, whereas a 10-year level of wind was used in the simulations for consistency with NBCC 2010. Furthermore, the roof drift was well below the 400mm limit prescribed by NBCC 2010, with a maximum roof drift of 19mm without the TLD and 16mm with the TLD.

The roof drift limits are similarly met for Scenario 2, which represents a more severe wind level. While the peak storey acceleration limits are not met for Scenario 2, the wind level represents a more severe loading than suggested for serviceability checks (Scenario 1). The mean storey displacements for Scenario 2 are shown in Figure 6, and a marked reduction in storey displacements can be observed, with more significant improvements to be seen at the higher storeys of the structure.

Table 4: Summary of improvement in mena storey displacement/shear results (top & bottom 20 storeys) with TLD vs without TLD.

Parameter	Scenario 1	Scenario 2	Scenario 3
Top 20 Storeys			
% Change in mean storey displacement	-8.27%	-7.51%	-6.90%
% Change in mean storey shear	+5.71%	+2.06%	+0.62%
Bottom 20 Storeys			
% Change in mean storey displacement	-0.25%	+0.98%	+0.65%
% Change in mean storey shear	-1.10%	-0.36%	-0.15%

Figure 7 also shows the time history of roof displacement under the most severe wind storm, Scenario 3. The results show that even for such a high level of wind, the TLD can provide significant reductions in roof drift, keeping the structure within the NBCC 2010 limit for roof deflection.

Table 4 summarizes some key parameters for structural performance, namely storey displacements and storey shear, but for the upper 20 storeys of the structure only. This is to understand the benefits that the TLD provides to the most flexible parts of the structure. For all three wind loading scenarios, the TLD can reduce the storey displacement demands at the upper storeys of the structure – this is critical for the comfort of the occupants, as these storeys typically experience the largest displacements.

Across all three wind loading scenarios, a couple of trends can be seen for the structural performance with and without TLD. The first is that there is a negligible change in base shear for the structure with the addition of the TLD; slightly higher storey shears are observed at the upper levels of the structure, whereas slightly lower storey shears are observed at the lower levels of the structure. While the TLD can dissipate vibration energy through non-linear liquid sloshing, the addition of a significant mass (1.5% of the structural mass) at the roof level of

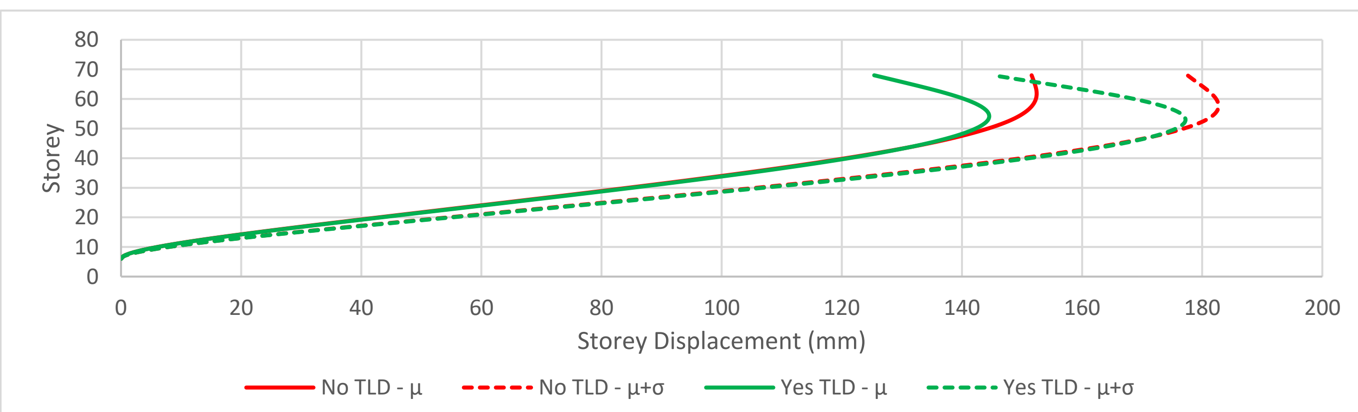

Figure 6: Mean storey displacement for wind loading scenario 2.

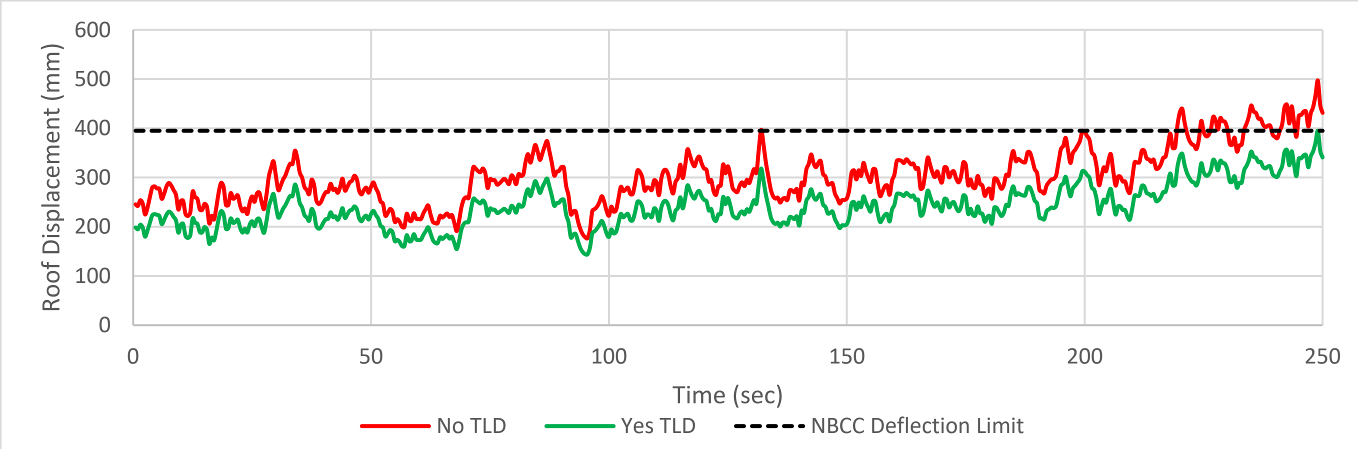

Figure 7: Roof displacement time history for wind loading scenario 3.

the structure imposes large forces on the upper storeys of the structure.

Secondly, a noticeable improvement in structural performance is observed with regards to roof displacement and global mean storey displacement, while a smaller improvement is seen with regards to storey accelerations. As was the case with the storey shears, the TLD provides more benefits at the upper storeys of the structure than at the lower storeys. This is still a good result, as these measures are critical for human comfort and serviceability checks, and the upper levels of the structure are where accelerations and displacements are the greatest – and hence the need for improved performance is also greatest.

III. Conclusion and Recommendations

The objective of this collaboration was to analyze structural performance of a real high-rise structure both with and without TLD under various levels of wind loading to better understand the effects and potential benefits the TLD could provide to the structure. Structural performance was then to be compared to the limits set in NBCC 2010 and CTBUH for human comfort and safety, including maximum lateral deflection of the structure and maximum storey accelerations.

The TLD model which was built was based on Yu's model, where the TLD is modeled as an equivalent non-linear-stiffness-damping oscillator. The TLD properties were tuned to the first mode of vibration of the structure, which was determined in OpenSees and cross-checked and verified with existing linear models in ETABS developed by the team at Stephenson Engineering, as well as third-party wind tunnel testing results.

The structure was analyzed under dynamic wind loading using a transient, multi-platform simulation algorithm. Three wind loading scenarios were considered, including NBCC 2010 service-level wind loads, and two more severe wind loading cases. The structure was found to remain in the elastic range for all three cases, and the structure met the NBCC 2010 and CTBUH human comfort limits. At higher levels of wind, the TLD effects were more pronounced, with benefits in terms of storey displacements and accelerations observed especially at the upper storeys of the structure. However, improvements to the structure with respect to storey and base shear were negligible with the TLD addition, due to the large additional mass imposed by the TLD at the roof level of the structure.

It is recommended that the TLD and structure are monitored together, with the TLD being modified such that its properties match that of the structure. TLD performance is highly correlated with the tuning of its frequency to the structure – as the structure ages and concrete cracks, it is likely that its vibration characteristics will change, and hence the TLD will likely need to be adjusted (e.g. changing water levels) to match the structure over time.

IV. Acknowledgements

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