Hindawi
Shock and Vibration
Volume 2019, Article ID 7091819, 13 pages
https://doi.org/10.1155/2019/7091819

Research Article

Analysis of a Benchmark Building Installed with Tuned Mass Dampers under Wind and Earthquake Loads

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Received 28 May 2019; Revised 27 September 2019; Accepted 5 October 2019; Published 19 November 2019

Academic Editor: Franck Poisson

This study presents analysis of a benchmark building installed with tuned mass dampers (TMDs) while subjected to wind and earthquake loads. Different TMD schemes are applied to reduce dynamic responses of the building under wind and earthquakes. The coupled equations of motion are formulated and solved using numerical methods. The uncontrolled building (NC) and the controlled building are subjected to a set of 100 earthquake ground motions and wind forces. The effectiveness of using different multiple TMD (MTMD) schemes as opposed to single TMD (STMD) is presented. Optimal TMD parameters and their location are investigated. For a tall structure like the one studied here, TMDs are found to be more effective in controlling acceleration response than displacement, when subjected to wind forces. It is observed that MTMDs with equal stiffness in each of the TMDs (usually considered for wind response control), when optimized for a given structure, are effective in controlling acceleration response under both wind and earthquake forces. However, if the device is designed with equal mass in every floor, it is less effective in controlling wind-induced floor acceleration. Therefore, when it comes to multihazard response control, distributed TMDs with equal stiffnesses should be preferred over those with equal masses.

1. Introduction

Wind response control of structures using passive tuned mass dampers (TMDs) has been extensively investigated in the last few decades. A detailed review of recent developments in vibration control of structures using passive TMDs is presented in Elias and Matsagar [1]. Optimally designed TMDs (tuning frequency optimized) have been found to be more effective than conventional ones, where the TMDs are tuned exactly to the frequency of the main structure. Multiple TMDs (MTMDs) are more effective when they are distributed along the height of the structure (d-MTMDs); their optimal placement depends on modal properties of the uncontrolled and controlled structures [2–9]. Advanced TMD schemes such as particle TMDs and TMDs with inerter are more effective than the optimal TMDs [10–17]. Energy-based design of TMDs or increasing the energy absorption of structures is an alternative procedure as explained in references [18–21]. Greco and Marano [18] reported that the energy criterion gave a great reduction of the response of the system equipped with TMD. Maximizing the ratio between the energy dissipated in the isolation system and the input energy globally transferred to the entire structure is another procedure for optimal design of TMDs [19]. Energy-based optimization was also applied on viscous dampers [20, 21]. The application of TMD in response mitigation of base-isolated buildings was also investigated by researchers [22–24].

TMD schemes are generally optimized for a specific type of loading (wind or earthquakes). Although simultaneous occurrence of strong wind and earthquake is a rare event, a structure may be subjected, during its lifetime, to both strong wind and earthquake ground motion. Consideration of multihazard performance is therefore important, especially for very tall and other structures which are sensitive to wind. A few studies have presented performance of buildings with/without vibration controller devices under both wind and earthquake excitations [25–34]. These studies show that while earthquake ground motions cause large floor acceleration demand, wind forces induce large interstory drift demands in tall buildings. Multihazard design consideration for tall buildings, therefore, needs to control both floor acceleration and interstory drift. Chapain and Aly [32] reported that viscous dampers showed their potential to enhance dynamic performance of buildings under multiple hazards. Earlier to this, Rezaee and Aly [33, 34] reported that semiactive controllers were quite effective in vibration mitigation of a wind turbine subjected to multiple hazards.

Effectiveness of passive TMDs in multihazard vibration control of tall buildings has not been sufficiently reported in the literature. This study aims to fill this gap by investigating performance of different TMD schemes in controlling vibration of a tall building subjected to wind and earthquake forces. The investigation aims to understand if a control scheme designed for wind is effective when the structure is subjected to earthquake and vice versa. Different types of control schemes making use of single and multiple tuned mass dampers are investigated. Single or multiple TMDs placed on the top of the building are compared to those distributed throughout the structure (d-MTMDs).

2. Benchmark Building with/without TMDs

The structure considered in this study is a 76-story benchmark building, which is very sensitive to wind [33–36]. Detailed information of the building can be found in reference [33, 35]. Different control schemes considered in this study are illustrated in Figure 1. A single tuned mass damper placed at the top of the building is denoted as STMD and is shown in Figure 1(a). Multiple TMDs placed on the top floor are denoted by MTMDs and are shown in Figure 1(b). Multiple TMDs placed on different floors of the building are denoted by d-MTMDs. Under this scheme, any floor can be installed with one TMD.

Optimum design of STMD for wind and earthquake response mitigation is different [35, 37]. Patil and Jangid [36] proposed a robust procedure to estimate optimal parameters of TMDs for wind-sensitive structures. Sadek et al. [38] provided optimal parameters of TMDs for earthquake-induced vibration control. The STMD and MTMDs designed for wind-sensitive structures [24, 36] are, respectively, denoted by STMD1 and MTMDs1, whereas STMD2 and MTMDs2 are the schemes designed for earthquake-induced vibration control [36, 38]. Multimodal response control of structures using TMDs is discussed in references [5–9]. Placing all TMDs at the top of a structure for multimodal response control under earthquakes is denoted by e-MTMDs [5–7]. Similarly, in the w-MTMD scheme, all TMDs are placed at the top and are designed for mitigation of wind response [8, 9]. TMDs distributed along the height of the building are denoted by ed-MTMDs and wd-MTMDs, respectively, when they are optimized for earthquake- and wind-induced vibration control according to [5–9]. Optimization of ed-MTMDs assumes that TMDs at all the floors have the same mass, while that of wd-MTMDs assumes that their stiffnesses are equal. This distribution of mass and stiffness is based on the assumption that the contribution of the first mode of vibration is higher in wind response than in seismic response, which can be relatively broadband. Thereby, in wd-MTMDs, the device tuned to the first mode has the highest mass which results in better control of the mode. However, these are only working hypotheses, and in the following, we test both of these schemes in wind and earthquake response control and find that equal stiffness scheme is preferable for both types of excitations. Since these optimizations are for either wind or earthquake forces, a detailed parametric study is conducted here to optimize the parameters of TMDs for both wind and earthquake forces. The schemes obtained by this optimization are denoted as ed-oMTMDs (equal TMD mass at every floor) and wd-oMTMDs (equal TMD stiffness at every floor).

3. Numerical Study

The benchmark building is classified as wind-sensitive structure because the ratio of height to length (306.1 m/42 m = 7.288) is more than 5 [35]. The first story is 10 m high; 2nd and 3rd, 38th to 40th, and 74th to 76th stories are 4.5 m high; and all other stories are 3.9 m high. Detailed description of the benchmark building and its numerical model is given in Yang et al. [35], wherein the rotational degrees of freedom have been removed by static condensation. The numerical model therefore considers one translational degree of freedom at each floor. A MATLAB code is prepared to conduct the numerical simulation of the uncontrolled and controlled buildings subjected to wind and earthquake excitations. It is to be noted that the cross-wind forces ($\{F_t\}$) are applied on the N degrees of freedom (DOFs) of the structure and with zero forces on n DOFs of the TMDs. The governing equations of motion for the building installed with the TMD schemes are already known [6–9].

Multihazard assessment of the benchmark building was presented by Venanzi et al. [31] using a linear elastic model. They tested the response of the building under wind and earthquake forces. They found that the contribution of the outer frames to the overall response is not as important as that of the inner core and that the inner core remained elastic even under strong ground motion and concluded that linear elastic modelling results in reasonable response parameters. Based on these conclusions, a linear elastic model is used and structural response is calculated by time integration using Newmark's method.

Ground shaking is represented by 100 real accelerograms described by Saha et al. [39]. The selected ground motions have peak ground acceleration in the range 0.025 g to 1.08 g, with g representing acceleration due to gravity.

Wind forces are represented by a time series of 900 sec duration and are generated according to [35, 40]. The mean wind velocity at the top of the building is varied from 14 m/sec to 66 m/sec. Along the height, wind velocity is varied according to a power law with an exponent of 0.365. Yang et al. [35] have described that forces applied on the benchmark structure in both directions (along-wind and across-wind) were derived from wind tunnel tests. They made a rigid prototype model of the benchmark structure (76-story building) and analyzed it in the open circuit type boundary layer wind tunnel facility available at the Department of Civil Engineering, the University of Sydney,

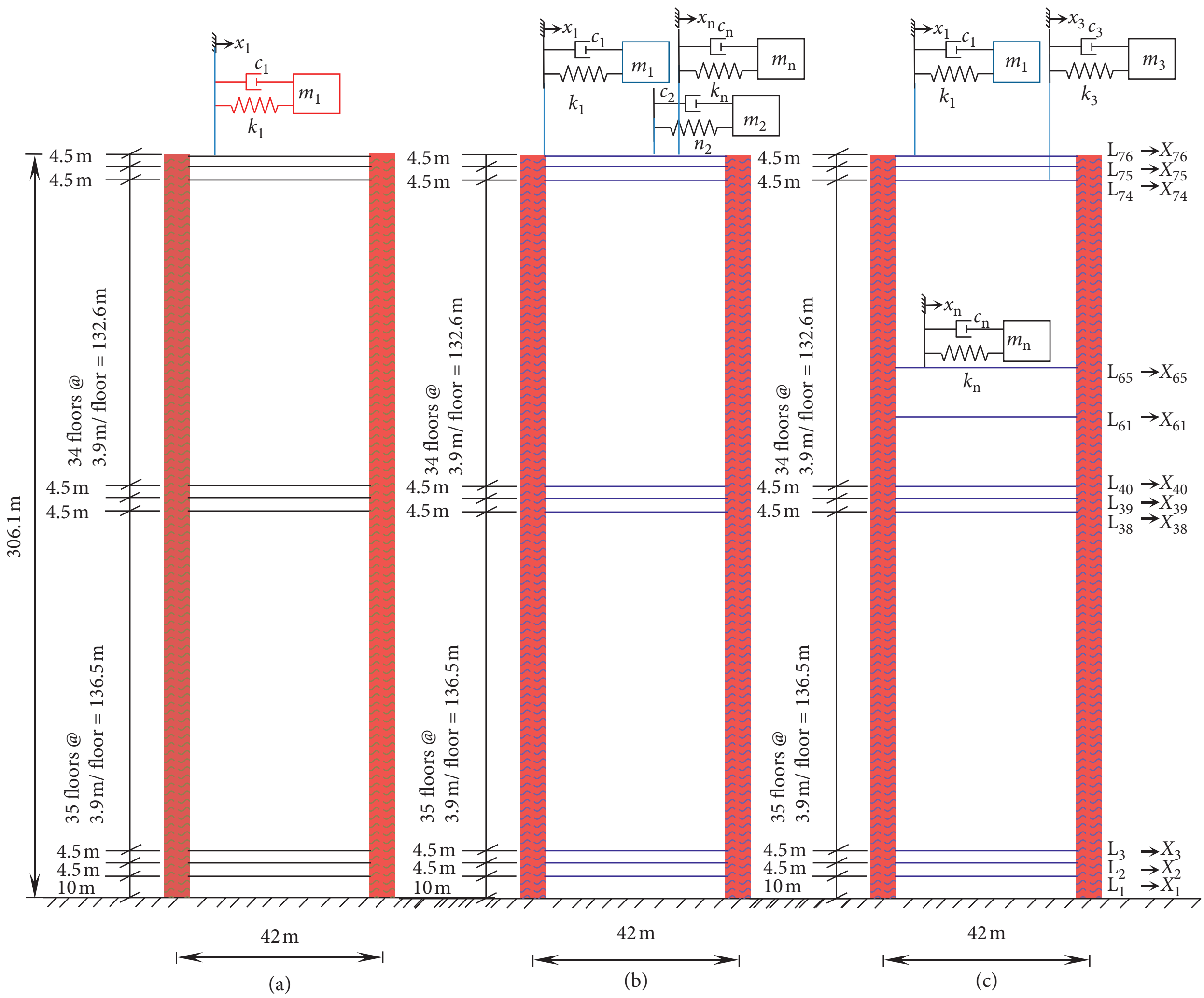

FIGURE 1: Model of benchmark building (a) installed with STMD, (b) installed with MTMDs, and (c) installed with d-MTMDs.

Australia [40, 41]. Samali et al. [41, 42] have given the details of the wind tunnel tests. The results were initially in the form of combined pressure coefficients (CPCs) referenced to the building height [38, 42]. To change these CPCs into wind forces, a suitable mean wind velocity at the i^{th} story of the benchmark structure is applied:

$$F_{ti} = 0.5\rho V_i^2 A C_p, \quad (1)$$

where ρ is the density of air, V_i is the mean wind velocity at the i^{th} floor of the building, A is the corresponding single panel area, and C_p is the combined pressure coefficient. The mean wind velocity at the top of the building was obtained from the reference mean wind velocity, V_r of 13.5 m/sec, at the height of 10 m from the ground [43]. Using equation (1) and the time scale of approximately 1 : 133, the pressure coefficients measured over 27 sec were converted into an hour-long wind force data. These wind force data (along-wind and across-wind directions) can be altered to signify larger or smaller wind velocities by multiplying the given time histories by $(U/V_i)^2$, where U is the desired mean wind velocity at the i^{th} story of the benchmark structure. As recommended by Yang et al. [35], for the performance evaluation of the control systems, only the first 15 mins in 900 s of across-wind data is enough for the computation of building response.

Performance of the devices is checked for both the root mean square (RMS) and peak response parameters at the roof. Cumulative distribution function (CDF) of RMS and peak response are calculated to compare the performances of the controlled and uncontrolled building.

3.1. Response Control of Benchmark Building. The performance of STMD is compared with that of w-MTMDs and e-MTMDs in Figures 2–5. The optimal parameters of the STMD1 are 0.94 and 0.06, respectively, for the frequency tuning ratio and damping ratio for a total mass of 1250 tons. The optimal parameters for w-MTMDs having five TMDs (each with a stiffness of 1064.389 kN/m) are 0.94 and 0.05 for the frequency tuning ratio and damping ratio, respectively. The optimal parameters of the STMD2 are 0.987 and 0.1398, respectively, for frequency tuning ratio and damping ratio for the same mass. Similarly, for e-MTMDs, each TMD has a mass of 250 tons with optimal parameters of 0.996 and 0.090,

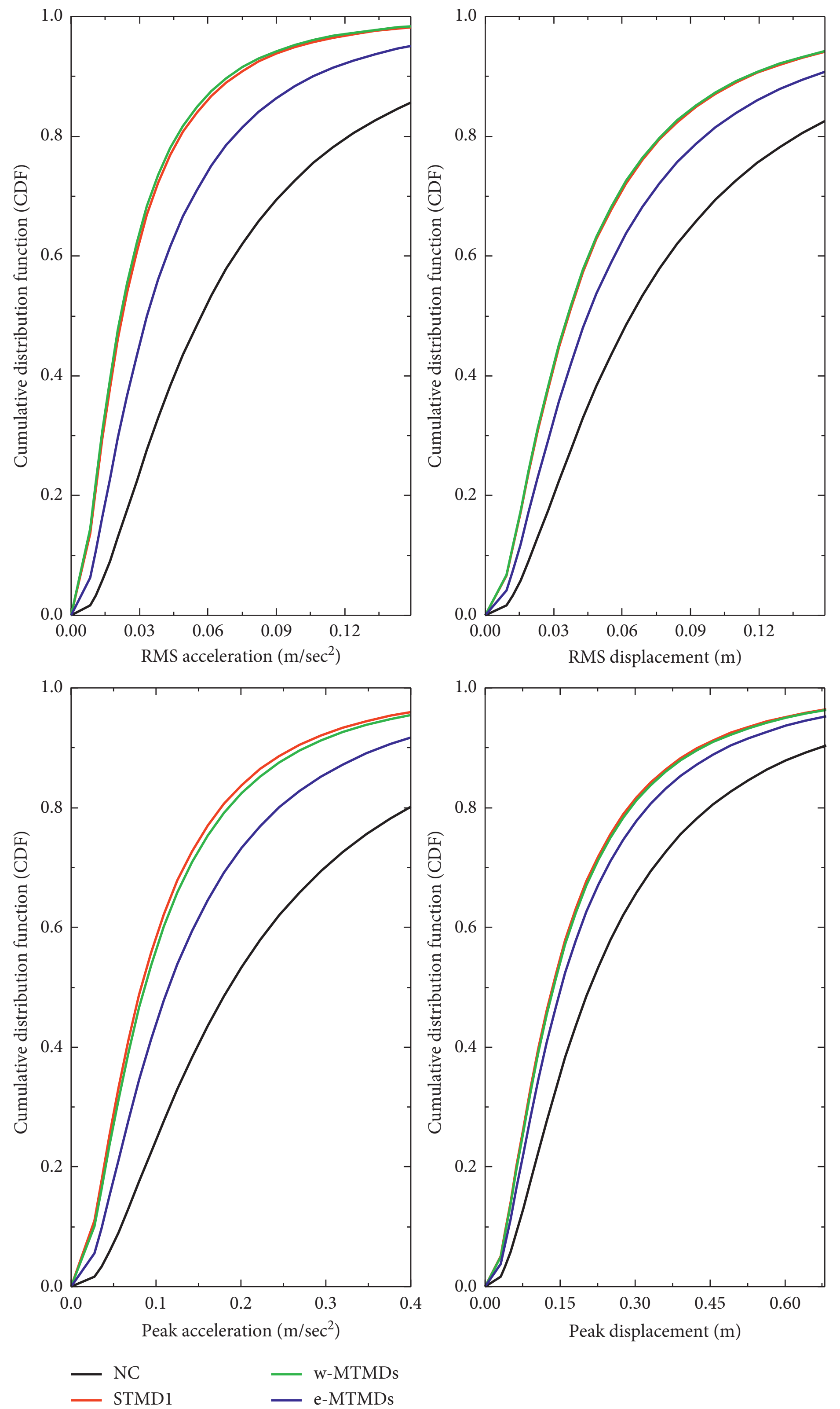

Figure 2: The CDF of responses for the NC, STMD1, w-MTMDs, and e-MTMDs subjected to wind forces.

respectively, for frequency tuning ratio and damping ratio. Figure 2 shows the CDF of different response parameters for the NC (not controlled), STMD, w-MTMDs, and e-MTMDs subjected to wind forces. For comfort of occupants, RMS acceleration should not exceed 0.005 g (g is acceleration due to gravity) more than once in every 6 years on average [45].

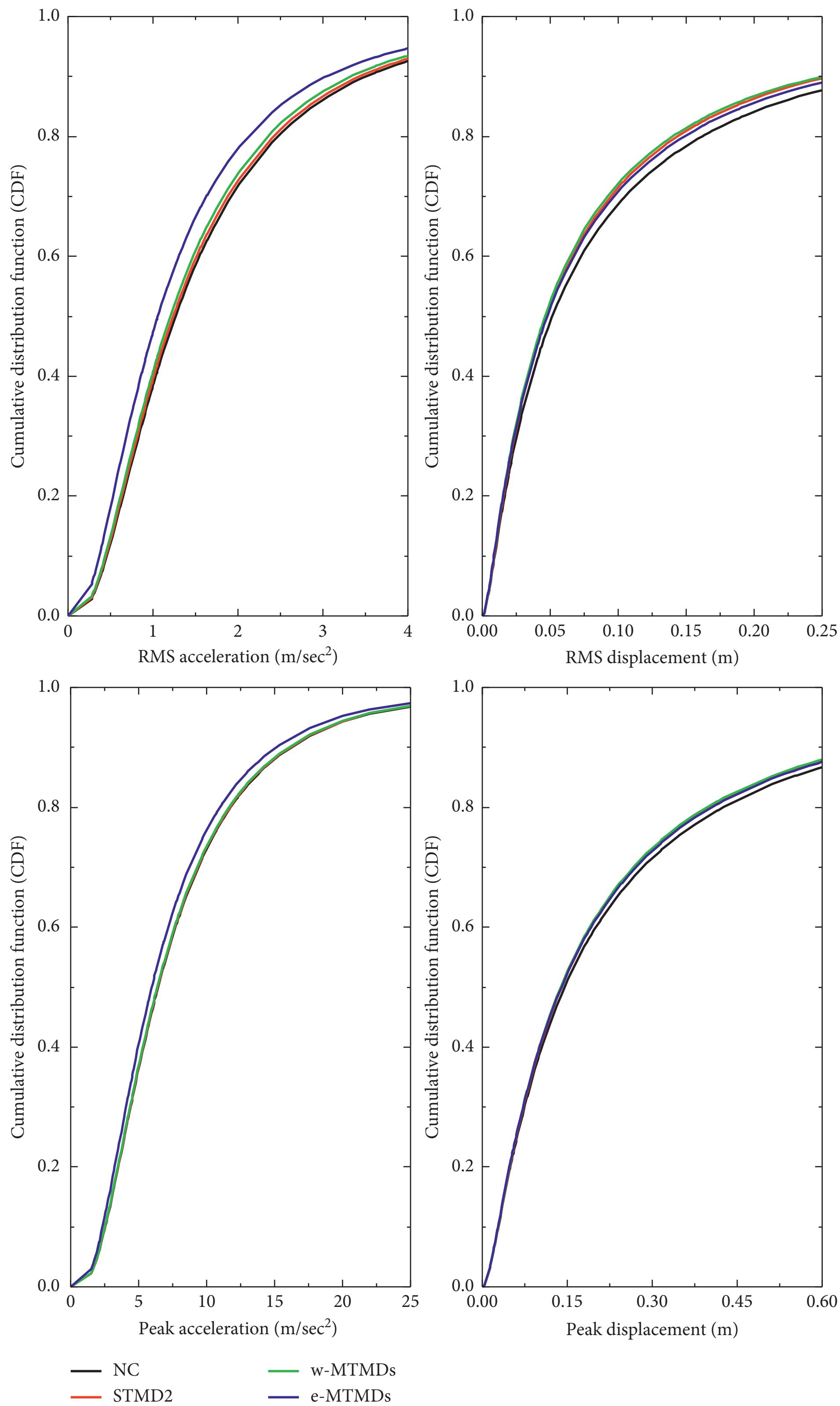

Figure 3: The CDF of responses for the NC, STMD, w-MTMDs, and e-MTMDs subjected to earthquake excitation.

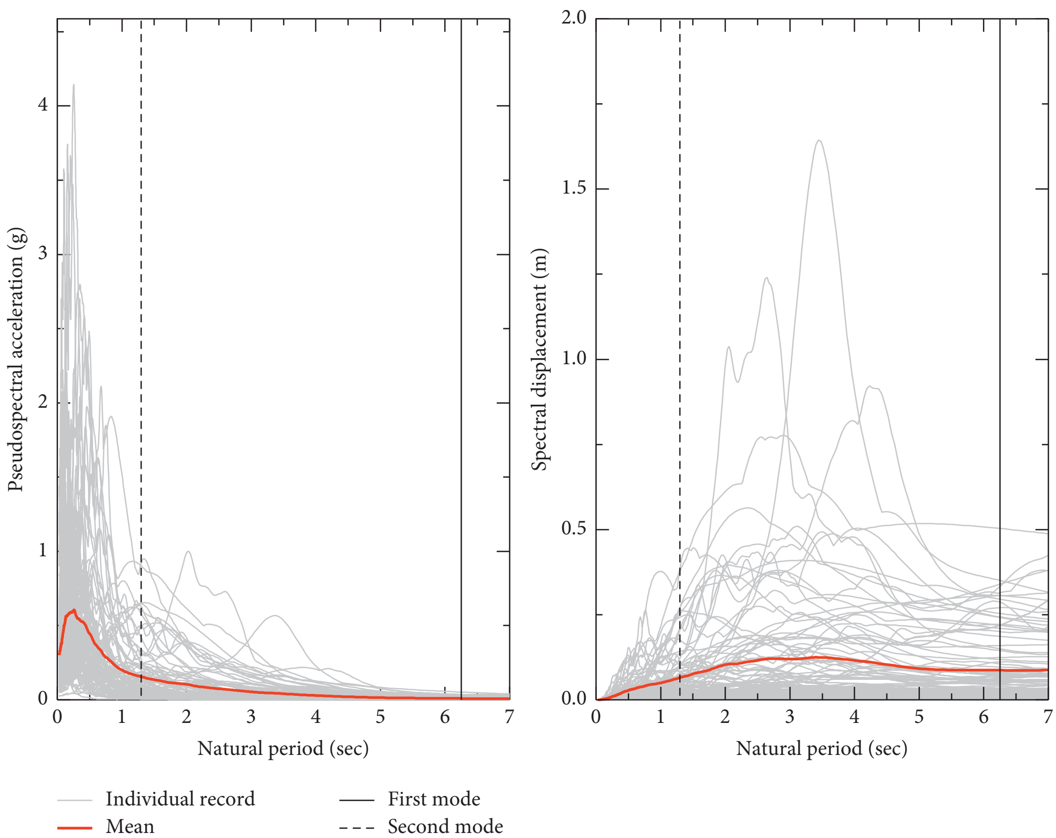

Figure 4: Elastic response spectra (5% damped) of the 100 ground motions used in the study. The average spectra are plotted in red, and the periods of the first two modes of vibration are represented with the vertical lines.

Similarly, peak acceleration should not exceed 0.02 g [46, 47]. In case of NC, there is 50% probability that the building fails the comfort criteria of RMS acceleration. The TMD schemes are quite effective in reducing the probabilities of exceeding the limits. As expected, the performance of w-MTMD is similar to that of the STMD but better than the e-MTMDs. Although the performance of w-MTMD is similar to that of STMD, the former may be preferable for robustness [36]. The e-MTMDs, where all the TMDs on the top floor have the same mass, is found to be the least effective scheme for reduction of wind response. The STMD1 and w-MTMDs are also effective in reducing wind-induced peak accelerations. It is to be noted that the displacement response of the structure is quite large under wind forces. Serviceability criteria require limiting lateral displacements of buildings. To avoid visibly disturbing damages, displacements are recommended to be less than H/500, with H being the height of the building. However, some cracking in partition walls may occur at this displacement. To avoid cracking in infill and partition walls, maximum displacements less than H/1000 are recommended [48]. This corresponds to a roof displacement of about 30 cm. For the range of wind speeds considered here, there is ~35% chance of crossing this limit. The TMDs reduce this probability by as much as half. The TMD schemes are not effective in controlling the acceleration or displacement response induced by ground shaking.

As reported also in references [30, 31], ground shaking induces more displacement than wind in the structure being studied for the range of wind speeds and ground motion considered.

Ineffectiveness of the TMD schemes in controlling earthquake-induced response of the building can be explained by looking at the frequency content of ground motions used in this study.

For this purpose, the dominant frequency of the ground motion [44] is estimated from 5% damped response spectra as

$$T_d = \frac{\sum S_A^2 T}{\sum S_A^2}, \tag{2}$$

where S_A is the pseudospectral acceleration normalized by the peak ground acceleration and T is the undamped natural period. The pseudoacceleration and displacement response spectra of the 100 ground motions used in this study along with their average values are shown in Figure 4.

Dominant ground motion periods normalized by the fundamental period of the structure, which is 6.25 s, is called as the normalized period. Peak response of the structure with one of the control schemes divided by that of the uncontrolled structure is called as response ratio. Figure 4 shows variation of response ratio as a function of the normalized period. All the ground motions have their dominant periods well below the fundamental period of the building, which is not unexpected for such a flexible

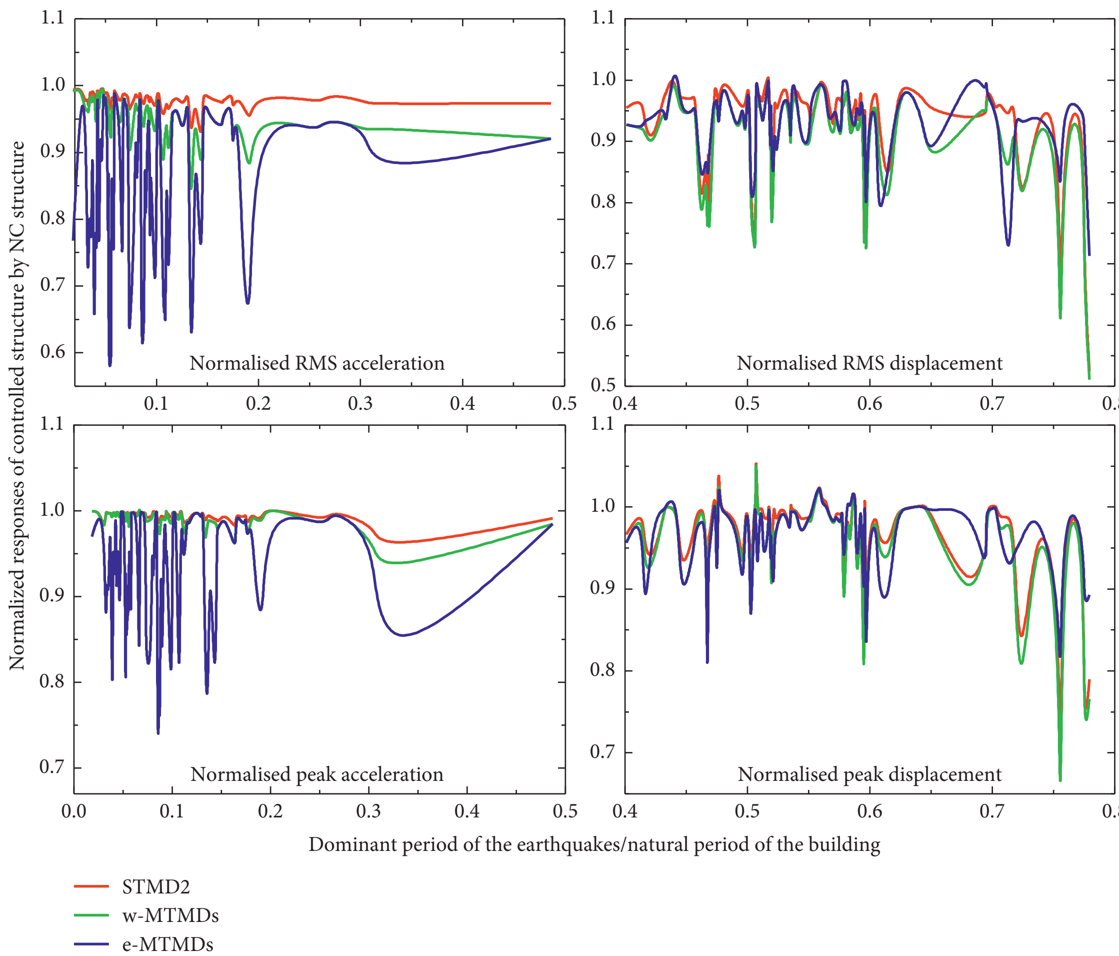

FIGURE 5: The variation of responses for the NC, STMD, w-MTMDs, and e-MTMDs subjected to earthquake excitation.

building. The STMD2, which is tuned to the fundamental mode of vibration, is not effective in controlling structural acceleration for all the ground motions considered here. This is because the fundamental mode of the building is not excited by the ground motions as much as the higher modes. Normalized period corresponding to the second vibration mode is ~0.2. For normalized periods less than about 0.2, the e-MTMDs are effectively controlling higher mode response (see Figure 5).

It is interesting to note that even the STMD2, which is tuned to the first mode, is somewhat effective in controlling displacement response induced by some earthquakes with normalized period in the range 0.3-0.4. This is because displacement response is controlled more by the longer period waves of ground motion, and the normalized periods used in Figure 5 are based on acceleration response spectra. It is also interesting to note that w-MTMDs are more effective than e-MTMDs in controlling earthquake-induced structural displacement. Distributed TMDs are thought to perform better (see, for example, [5, 9]). Figure 6 shows that the performance of TMDs designed for wind does not improve by distributing them along the height when the structure is excited by wind, and Figure 7 confirms that distributed TMDs perform better under both wind and ground shaking.

Advantage of distributed TMDs over all TMDs placed on the same floor is more for ground shaking (Figure 7). When the structure is subjected to earthquake ground motion, multiple TMDs distributed along the height are more effective than TMDs placed on the top floor in controlling acceleration response. It is to be noted that the wd-MTMDs and ed-MTMDs discussed so far were designed according to general formulations available in the literature. These formulations are based on generic structures and might not be optimal for a specific structure.

To explore improvement in performance of distributed TMDs for the structure being studied here, a detailed parametric study was carried out. A range of frequency tuning ratio and damping ratio were considered for a given mass ratio. At the first stage, frequency tuning ratio was assumed to be 1 and optimum damping ratio was identified by considering average displacement response when subjected to the range of wind and ground motions used in this study. Then, the frequency tuning ratio is varied from 0.85 to 1.15, and the optimum value, which minimizes average displacement response, is selected. This process is repeated until an optimal combination of damping ratio and frequency tuning ratio is identified (Figure 8). The optimal solutions are denoted as ed-oMTMDs and wd-oMTMDs. Figure 9 compares these schemes with w-MTMDs and

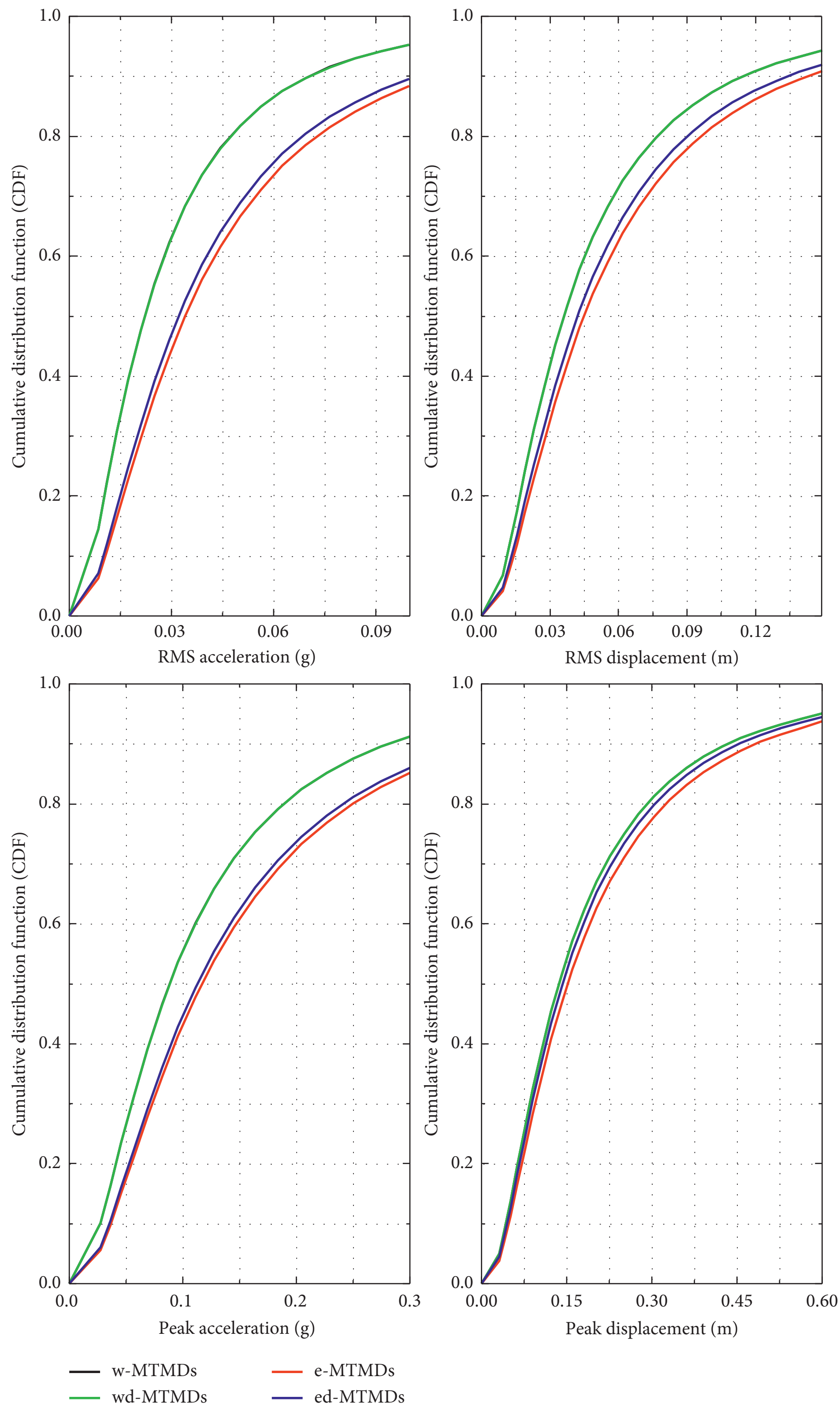

FIGURE 6: The CDF of responses for the w-MTMDs, e-MTMDs, wd-MTMDs, and ed-MTMDs subjected to wind forces.

e-MTMDs. It is observed that wd-oMTMDs and ed-oMTMDs perform much better than wd-MTMDs and ed-MTMDs in controlling acceleration response of the structure.

4. Summary and Conclusions

The results show that the wind-sensitive building is also adversely affected by earthquake ground motions. Therefore,

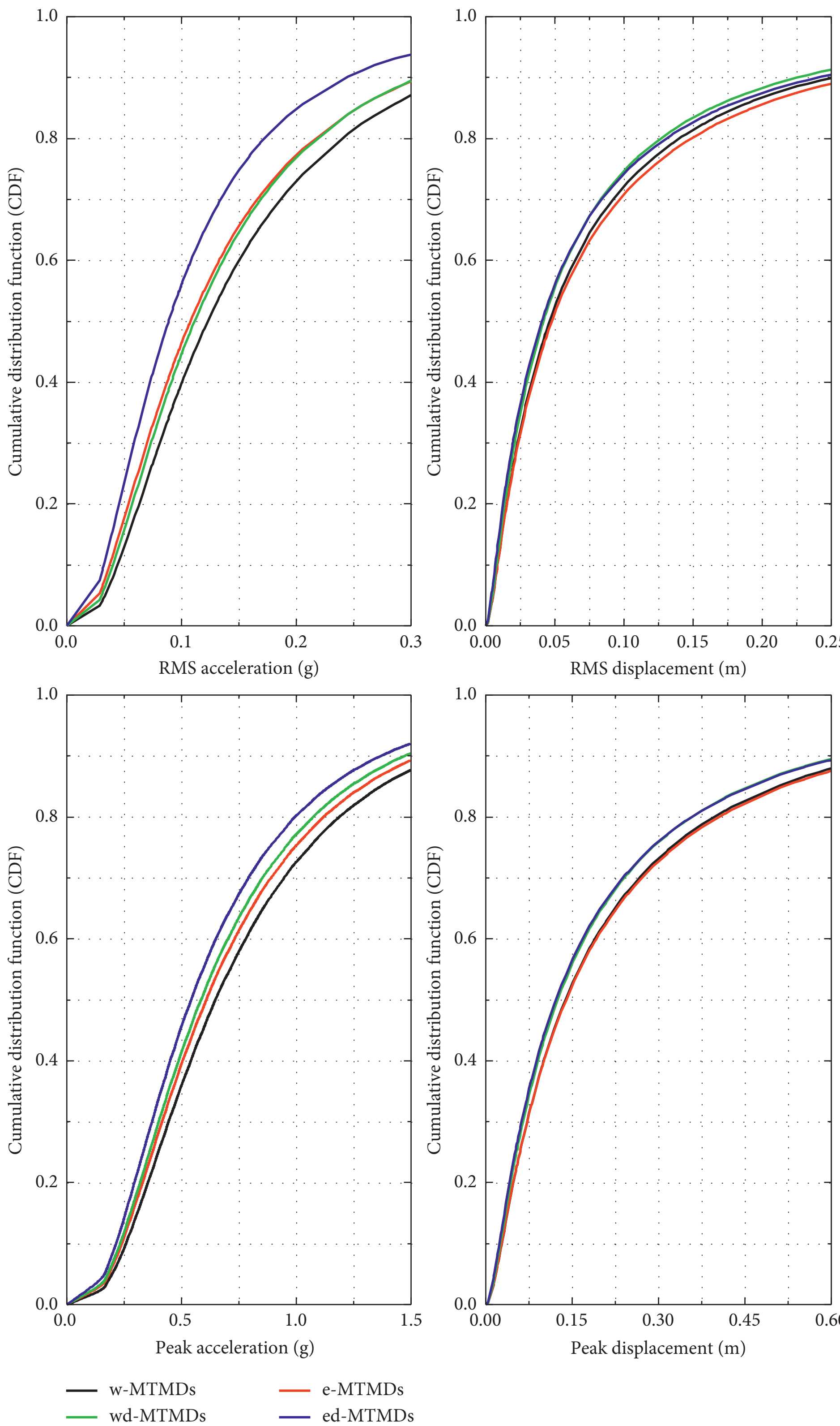

FIGURE 7: The CDF of responses for the w-MTMDs, e-MTMDs, wd-MTMDs, and ed-MTMDs subjected to earthquake excitations.

a multihazard approach is needed for robust design of such structures. In general, the TMD schemes investigated here were more effective in controlling wind response than earthquake response. It was observed that, in some cases, STMDs optimized for wind-induced response control amplified structural displacements when

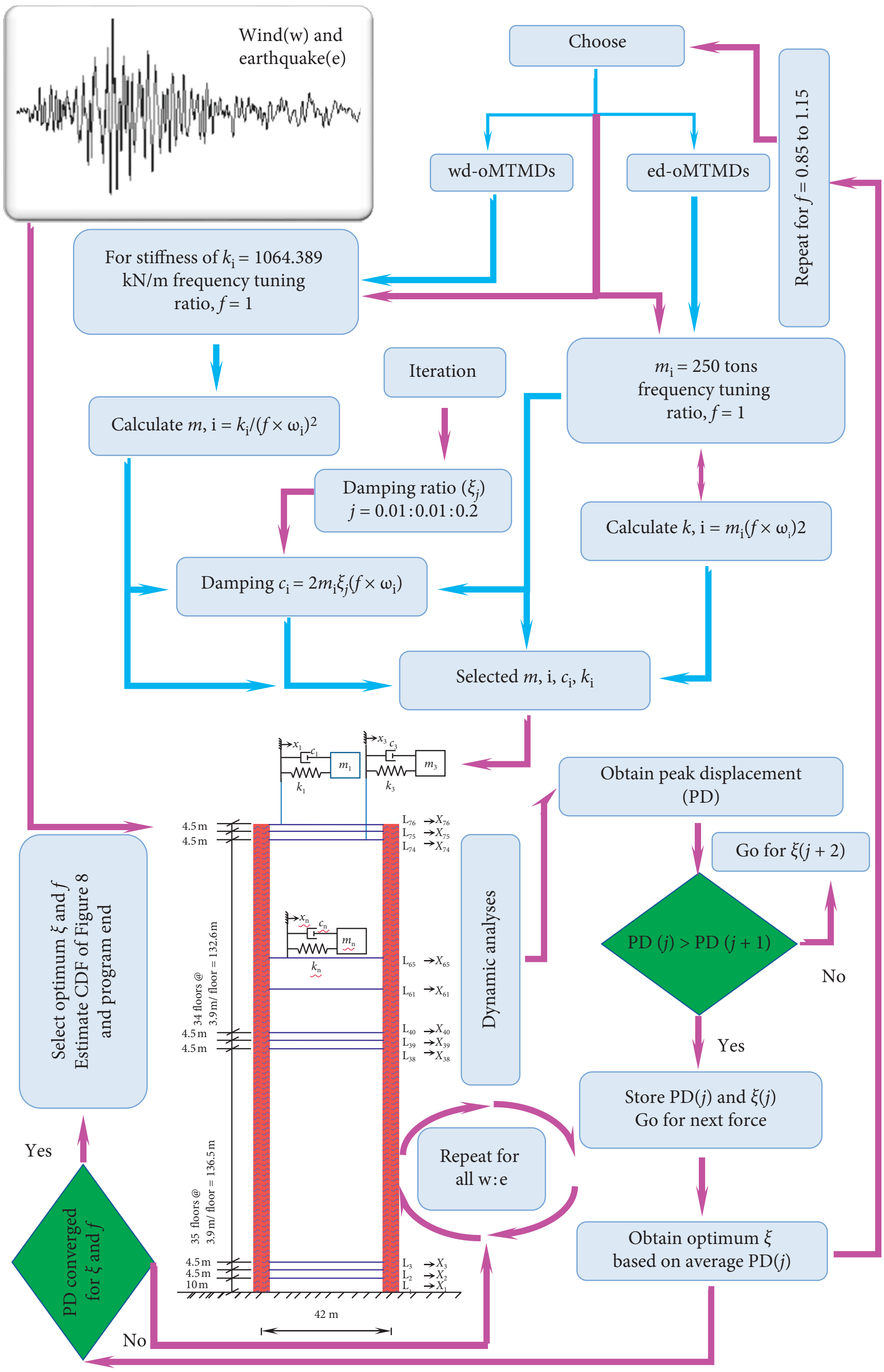

Figure 8: Flowchart for optimum design of d-MTMD for response reduction of buildings under wind and earthquake excitations.

subjected to earthquakes. However, those that were optimized for seismic response mitigation did not show adverse effects when subjected to wind forces. Based on the results presented above, the following conclusions can be drawn :

(i) Tall buildings designed for wind forces can experience severe floor acceleration during earthquakes. This can result in damage to nonstructural contents of the buildings, which may represent a significant percentage of the value of the building.

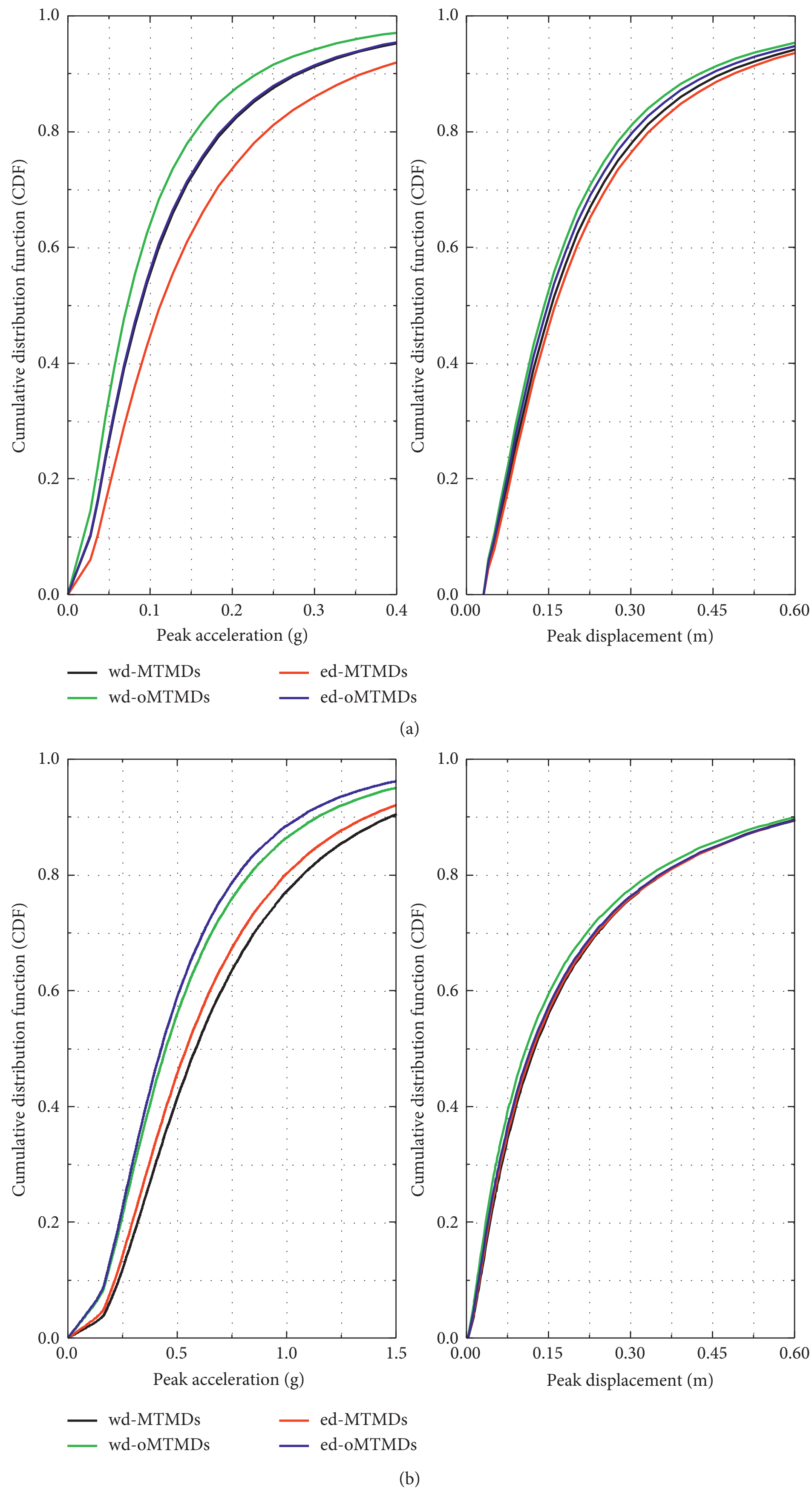

Figure 9: The CDF of responses for the wd-MTMDs, ed-MTMDs, wd-oMTMDs, and ed-oMTMDs subjected to wind (a) and earthquake excitations (b).

(ii) The TMDs are more effective in controlling acceleration response than displacement response when subjected to wind and earthquake ground motion.

(iii) TMD schemes for wind response control optimized using generic formulations from the literature are not effective in controlling seismic response. Optimization therefore needs to be carried out for the specific structure being designed. It is found that when MTMDs are optimized for the specific structure, they can be effective in controlling acceleration response under strong ground motion.

(iv) MTMDs with equal stiffness in each of the TMDs (usually considered for wind response control), when optimized for a given structure, is effective in controlling acceleration response under both wind and earthquake forces. However, if the device is designed with equal mass in every floor, it is less effective in controlling wind-induced floor acceleration. Therefore, when it comes to multihazard response control, distributed TMDs with equal stiffnesses should be preferred over those with equal masses.

Data Availability

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

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Acknowledgments

The authors acknowledge the support from the University of Iceland Research Fund.

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