Mitigating footfall-induced vibration in long-span floors*

T Nguyen†, E Gad and J Wilson
Swinburne University of Technology, Melbourne, Victoria

N Haritos
The University of Melbourne, Melbourne, Victoria

ABSTRACT: *Long-span lightweight floor systems with low damping are susceptible to serviceability problems due to human excitations. This paper discusses two remedial measures to alleviate disturbing footfall-induced vibrations observed on a real office floor of steel-concrete composite construction. One treatment involves stiffening the existing floor beams while the other utilises passive control with an innovative distributed multiple viscoelastic tuned mass damper (TMD) system. Several finite element models reflecting different remedial scenarios are analysed from which the TMD approach and the traditional stiffening technique are found to be comparably effective in reducing the vibration level. However, the proposed TMD solution is far superior, feasible and non-intrusive when applied to existing floors. A probability-based evaluation of the effectiveness of the TMD system is also conducted, taking into account likely variations in the walking force and dynamic properties of both the floor and dampers. The custom-made damper system has been successfully installed on the real floor and performed efficiently in various field tests.*

KEYWORDS: Floor vibration; stiffening; tuned mass damper; probabilistic analysis.

REFERENCE: Nguyen, T., Gad, E., Wilson, J. & Haritos, N. 2014, "Mitigating footfall-induced vibration in long-span floors", *Australian Journal of Structural Engineering*, Vol. 15, No. 1, January, pp. 97-109, http://dx.doi.org/10.7158/S12-061.2014.15.1.

1 INTRODUCTION

Long-span lightweight floor systems are prone to vibrations caused by various human activities. Constraining vibration levels in such floors to meet human comfort criteria is a vital serviceability requirement. Excessive footfall-induced vibrations can cause annoyance, disturbance and discomfort to building occupants. Current design guides normally quantify vibration levels in terms of peak acceleration, root-mean-square acceleration or velocity; and recommend acceptable limits of vibration below which the probability of adverse comments is low (Murray et al, 2003; Willford & Young, 2006; Smith et al, 2009). The vibration thresholds are adjusted for various human activities in different environments. For instance, a peak acceleration limit of 0.5% *g* is typically used for offices while that for shopping malls is about 1.5% *g*, ie. three times higher (Murray et al, 2003).

Traditional techniques to reduce vibrations include modification of structural members and architectural components, thus adjusting the basic inherent stiffness, mass and damping of a floor. These actions can be taken during the design stage rather than after the completion of construction. Bachmann (1992a) mentioned the modification of the design of a three-span footbridge by changing the span ratio to cut half of the response level. Adding material to make larger cross-sections is a commonly used stiffening scheme. Bachmann & Ammann (1987) described the addition of steel plates and filled concrete to existing concrete floor beams of a gymnasium hall. This remedy increased the floor natural frequency by a factor of 1.5 and effectively reduced 95% of the floor response. Stiffening can be done by reinforcing the bottom flange of floor beams or bottom chord of floor trusses with cover plates,

* Paper S12-061 submitted 7/12/12; accepted for publication after review and revision 23/03/13.

† Corresponding author Tuan Nguyen-Huu-Anh can be contacted at tuannguyen@swin.edu.au.

rods or queen post hangers (Murray et al, 2003). Providing additional supports to reduce the structure span can enhance the stiffness and hence frequency of a floor system. The extra supports may be in the form of intermediate steel columns supporting floor girders as applied in an office building reported by Bachmann & Ammann (1987), or an additional steel girder grid under a lively gymnasium floor as discussed by Bachmann (1992a). Another remedial scheme involves the use of damping posts in the form of steel piles with a viscoelastic material attached to the top. This technique successfully eliminated half of the displacement magnitude of a 34 m span ballroom floor at a hotel in Manhattan (Post, 1997). Besides modification of the floor stiffness, reducing the annoying effects of vibrations and relocation of the vibration source are also found to be helpful in some cases.

The aforementioned solutions may have significant shortcomings when attempted on existing floors. Stiffening structural members of in-service floors or providing extra supporting columns may be very intrusive, obstructive and architecturally unacceptable. On the other hand, the use of structural control employing supplemental energy dissipation devices is a more advanced approach to reduce annoying floor vibrations. Different configurations of passive tuned mass dampers (TMDs) have been developed for floor vibration applications with some degree of success. Lenzen (1966) used small dampers in the form of simple spring-mass-dashpot systems hung from the floor beams to successfully eliminate annoying floor vibrations. Allen & Swallow (1975) developed a TMD consisting of a steel box loaded with concrete blocks and supported at each corner by a commercial compression spring within a housing. Bachmann & Ammann (1987) reported a successful installation of eight TMDs on a lightweight floor in an exhibition pavilion whereby the displacement response was effectively reduced by a factor of six. Setareh & Hanson (1992) used five pairs of TMDs in a long-span balcony of an auditorium in Detroit, Michigan, where severe vibration was caused at the resonance of the fundamental frequency of the structure due to audience-participation at rock-music-beat frequency. Webster & Vaicaitis (1992) employed a TMD system consisting of a concrete filled steel box and steel plates suspended by springs and viscous dampers to decrease at least 60% of the vibration during an actual dance event on a long-span, cantilevered, composite floor system of a ball room. Bachmann (1992b) reported the upgrade of a four-span pedestrian and cycle bridge using two TMDs installed on the longest span of the bridge. Tacet Engineering Ltd., Toronto, installed a TMD system on a third-floor gym of a high school building in New York City (Thornton et al, 1990; Velivasakis, 1997). Shope & Murray (1995) developed a non-conventional TMD configuration in which the horizontal steel plate functioned as the resilience element while two rigid containers, which enclosed multi celled liquid filled bladders, served as the damping element. Collette (2002) employed passive TMDs in an indoor suspended footbridge connecting two concrete buildings. The installed TMDs suppressed the acceleration response in both the footbridge and an adjacent meeting box, used for office and client meetings, to acceptable levels for human comfort. In addition to passive control, the implementation of active and semi-active control schemes for floor vibration mitigation has been attempted (Hanagan & Murray, 1997; Setareh et al, 2007; Reynolds et al, 2009).

A conventional passive damper with spring and dashpot can be effective for some floor systems such as a footbridge, stadium, or ballroom where the vibration displacement is large enough to excite such a mechanical device to work. It may not be practically suitable for office floors whose displacement amplitude is much lower. Indeed, if the peak acceleration response of an office floor equals the acceptable limit of 0.5% *g* suggested by the AISC/CISC DG11 (Murray et al, 2003) then the displacement amplitude would be below 0.1 mm, assuming a natural frequency of 4-8 Hz normally found in composite floors. In an attempt to develop a damper particularly suitable for office floors, the authors have designed an innovative configuration of passive TMDs using viscoelastic material. The present paper examines the effectiveness of the proposed damper in retrofitting a real office floor in comparison with a traditional remedial method using a stiffening technique. The paper also discusses a probabilistic evaluation of the floor vibration and the performance of the damper.

2 DESCRIPTION OF CASE STUDY FLOOR

Annoying footfall-induced vibrations were reported by tenants occupying an office floor of steel-concrete construction with framing layout as shown in figure 1(a). The most disturbed floor bay was at the north-west corner of the building where two long perpendicular corridors intersect at the bay centre. A number of physical heel drop and walking tests were conducted to measure the modal properties and response of the floor (Nguyen et al, 2012). In the heel drop test, a person rose onto his toes with his heels about 63 mm off the floor and suddenly dropped his heels to the floor. The acceleration response of the floor was recorded by accelerometers with a sensitivity of 5 V/g located around the floor bay centre. Data analysis performed on the measured heel drop response revealed a natural frequency of about 6.2 Hz and damping ratio of 2.5% to 3% for the problematic bay. Moreover, a calibrated finite element (FE) model of the floor predicted a natural frequency of 6.22 Hz, modal mass of 20,600 kg, and a mode shape as shown in figure 1(b), for the resonant mode of the investigated floor bay. The measured

Figure 1: Case study floor – (a) floor plan, and (b) a mode shape critical to the problematic bay.

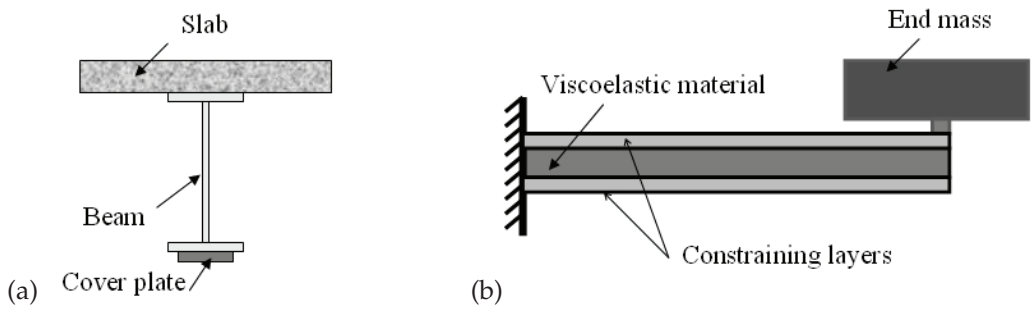

Figure 2: Remedial measures for floor vibration problem – (a) stiffening with cover plate, and (b) sandwich-beam TMD.

floor acceleration due to people walking along the two corridors at a normal pacing rate of around 1.9-2.2 Hz was within a range of 0.5% to 0.7% *g*. This vibration level exceeds the recommended threshold of 0.5% *g* for human comfort in an office environment (Murray et al, 2003); and therefore remedial measures were targeted at reducing the vibration response.

Two remedial measures have been considered. One treatment involves stiffening the existing floor beams with steel cover plates as shown in figure 2(a). The other method utilises passive control with an innovative TMD system consisting of a number of cantilever sandwich-beam TMDs. Each TMD has a layer of rubber as viscoelastic material constrained between two steel plates. A concentrated mass placed at the tip of the sandwich beam contributes most to the mass of the device and facilitates frequency tuning. Figure 2(b) illustrates the different components of a single damper. The modulus of elasticity of the constraining layers is much greater than that of the viscoelastic material. When the damper undergoes cyclic bending, the constrained viscoelastic material layer is forced to deform in shear and hence dissipates energy.

3 SUPPRESSION OF FLOOR VIBRATION BY MEANS OF STIFFENING

3.1 Stiffening scenarios

The frequency of the floor system can be increased by stiffening floor beams and girders with steel cover plates as shown in figure 2(a). The floor beams would need to be jacked up prior to welding of the cover plate so that a composite action between the bottom flange and cover plate can be achieved. As the natural frequency of the girders was found to be 1.5 times higher than that for the beams, stiffening the more flexible members (ie. beams) should be first considered. Two scenarios were investigated:

stiffening beams only, and stiffening both beams and girders. The treated beams and girders are marked as "B" and "G" respectively in figure 1(a).

Analyses were carried out for the cases in which the cover plate width was fixed at 150 mm while its thickness varied from 10 to 50 mm. In all cases, it was assumed that full composite action could be achieved between the cover plates and retrofitted beams.

3.2 Modal analysis

The FE model created for the original floor (unstiffened) was modified to create new models that included the stiffened beams and girders. Modal analysis was then performed on each of the new models corresponding to a stiffening scenario. Examples of the natural modes critical to the stiffened bay are shown in figure 3 for the cases of 30 mm cover plates added to beams only and to both beams and girders. It should be noted that the 6th mode in the modified FE models was found to be the resonant mode of the stiffened bay. The resonant mode defined here is the mode that results in antinodes with maximum modal displacements at the bay of interest. The resonant mode of a particular bay can therefore be different to the first mode obtained from FE modal analysis of the entire multi-bay floor.

Figure 4 shows the resonant natural frequency of the bay obtained for different stiffening scenarios. As can be seen, the thicker the cover plate, the higher the natural frequency. For the same cover plate thickness, stiffening both beams and girders resulted in higher frequency than reinforcing beams only.

Interestingly, it was found that the addition of cover plates altered not only the floor frequency but also the distribution of modal mass and the configuration of mode shapes. For instance, the resonant mode of the investigated floor bay with 30 mm thick plates added to both beams and girders had a natural frequency of 6.99 Hz and modal mass of 33,410 kg. This resonant mode, which resulted in maximum modal displacements at the floor bay centre, was the 6th mode among the many mode shapes of the entire floor. In comparison, the 4th mode shown in figure 1(b) was found to be the resonant mode of the original floor bay (ie. unstiffened) with a natural frequency of 6.22 Hz and modal mass of 20,600 kg. Therefore, in addition to an enhancement in the stiffness and frequency, the stiffening scheme benefits from the redistribution and increase in the modal mass associated with the vibration mode of interest. These factors would contribute to an overall reduction in the floor response.

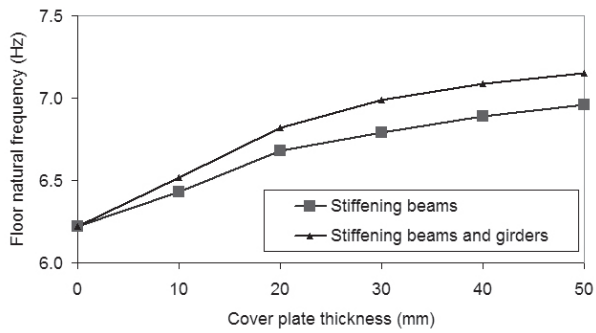

Figure 4: Resonant frequency of stiffened floor bay.

Figure 3: Critical mode shapes for the two different stiffening scenarios – (a) Mode 6 (6.79 Hz), 30 mm cover plates added to beams only; and (b) Mode 6 (6.99 Hz), 30 mm cover plates added to both beams and girders.

3.3 Walking response analysis

The walking excitation can be represented in the FE model by a concentrated force $F(t)$ of equation (1) applied at the centre of the problematic floor bay, ie. where maximum modal displacement occurred, hence representing the worst case:

$$F(t) = P\sum_i \alpha_i \cos\left(2\pi i f_p t + \varphi_i\right)u \quad (1)$$

where P is the walker's weight taken as 800 N. The Fourier coefficients α_i can be taken as 0.5, 0.2, 0.1 and 0.05 for the first, second, third and fourth harmonic components, respectively, of the walking excitation with a footstep frequency of f_p (Murray et al, 2003). Phase angles φ_i can be taken as 0 for the first harmonic and $\pi/2$ for the others (Bachmann & Ammann, 1987). To model a person walking from one end of the floor span to the other, the floor mode shape value u corresponding to various footstep locations along the walking path was incorporated into the forcing function (Nguyen et al, 2011).

In each time history analysis, a critical value for the step frequency f_p was assumed such that the resultant response can be maximised. For instance, figure 5(a) shows the response history of the original floor subjected to walking at a selected footstep frequency of 2.07 Hz whose third harmonic matched the floor natural frequency of 6.22 Hz. Figure 5(b) shows the walking response for the case where the floor was stiffened with 30 mm thick cover plates added to both beams and girders. The natural frequency of this stiffened floor was found to be 6.99 Hz. The critical footstep frequency assumed for this case was taken to be 2.25 Hz rather than the resonant footstep frequency of 1.75 Hz for which the fourth harmonic is resonant with the floor frequency. This is because the resonant step frequency was found to result in a lower response level than for the 2.25 Hz pacing rate. The 2.25 Hz value suggested here can be thought of as a likely upper limit of the footstep frequency generated by normal walking. This proposal stems from the measured gait data reported in Nguyen et al (2011) which revealed that the average step frequency for normal walking is about 1.98 Hz with a standard deviation of 0.13 Hz.

Figure 6 shows the predicted maximum walking response associated with different stiffening scenarios. Each data point in figure 6 was collected from an appropriate acceleration time history, similar to those plotted in figure 5. As can be seen, the thicker the cover plate, the lower the resultant walking response. If the cover plate thickness was increased to 44 mm for beams or 26 mm for both beams and girders, the peak floor acceleration equalled the human comfort threshold of 0.5% *g*. Although these two scenarios require almost the same amount of steel material of about 1300 kg, reinforcing the beams only would involve less jacking and welding work, and hence preferable. When the cover plate thickness was further increased to 50 mm for beams or 30 mm for both beams and girders, a more comfortable vibration level at 0.44% *g* can be achieved. This treatment requires 1470 kg of steel material.

FE investigation was also carried out for the case of half-length reinforcement where the cover plate was applied only to the portion between the quarter points of a beam span rather than extending the full length of the beam. Using 50 mm cover plates for half-length reinforcement of the floor beams was found to reduce the peak acceleration to 0.59% *g*. This vibration level still exceeds the acceptable limit and is about 34% greater than that obtained from the corresponding full-length solution with 50 mm plates (0.44% *g*). However, the 50 mm plate half-length reinforcement can be more effective than a 25 mm plate full-length solution to beams because the latter scenario was found to result in a higher floor acceleration of 0.64% *g*.

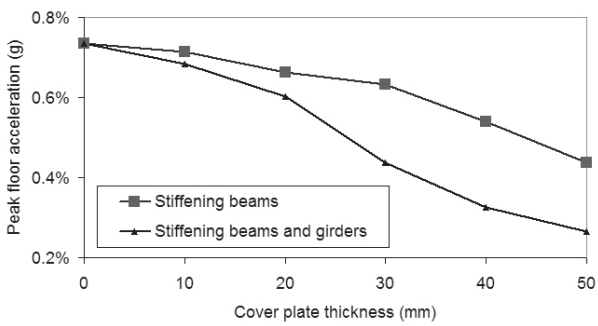

Figure 6: Maximum response of stiffened floor bay.

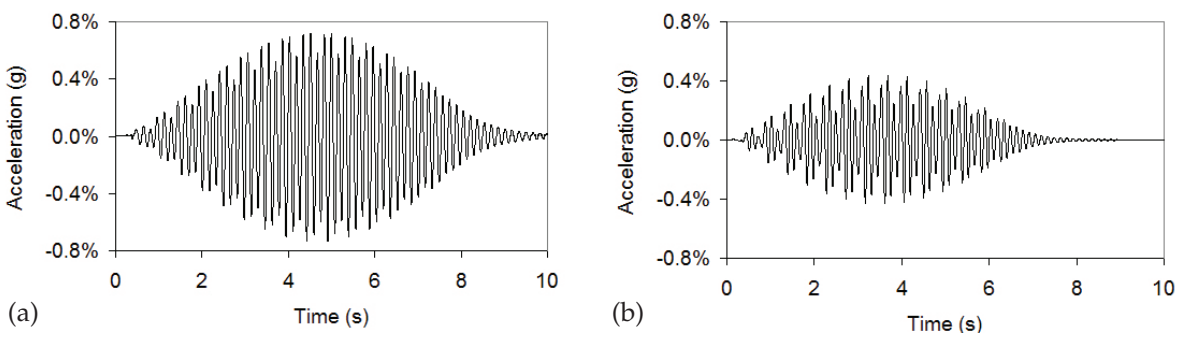

Figure 5: Floor response to a critical pacing rate – (a) original floor, and (b) stiffened floor.

3.4 Discussion on addition of cover plates

The FE investigation suggested that the proposed stiffening technique can mitigate the floor vibration to an acceptable limit. This method would, however, be intrusive when applied to the existing floor as it requires access to the storey below the problematic floor. Another significant drawback is the need to jack up the floor prior to welding of the stiffening cover plate so that the added material can provide additional stiffness rather than just adding its self-weight to the existing structural system. Reinforcement of existing beams requires extensive overhead welding, which is also a disadvantage. Therefore, it was decided not to apply a stiffening technique on the real floor. Instead, a TMD system has been developed and installed successfully on the floor, with details presented in the following sections.

4 VIBRATION CONTROL USING VISCOELASTIC TUNED MASS DAMPERS

4.1 Distributed multiple TMDs

A damper system utilising the sandwich beam concept introduced in section 2 and shown in figure 2(b) was designed for the case study floor. This TMD system can be installed either under the slab (or supporting beams) or on top of the slab within the false floor if present. While TMD installation from above the slab would be most preferable when false floors exist, installation from below would still be much less disruptive than stiffening with cover plates because neither jacking nor overhead welding is required. In either case, the TMDs can be designed to fit within the tight space of the false ceiling or false floor. An innovative distributed multiple TMD system to be fitted within the existing false floor was developed as shown in figure 7. A total of 12 cantilever sandwich beams were distributed in three TMD groups which were fitted at locations close to the floor bay centre. Each group can be considered as a four-arm damper where four sandwich beams were attached to the same base bolted to the concrete floor.

One appropriate type of commercially available rubber was used for the core of the damper. The dimensions of various components constituting the sandwich beam were calculated such that the damper frequency was close to the floor frequency. This resulted in each sandwich beam having a length of about 400 mm with an end mass of 22.5 kg. The modal mass of each TMD including the self-weight of the sandwich beam (two constraining layers and a rubber core) was about 23 kg. The total mass of the 12-damper system including the three supporting bases was around 280 kg. The thickness of each constraining steel layer was 6 mm and the rubber core was about 19-20 mm. The common pluck test was performed to validate the predicted tuning frequency and equivalent viscous damping ratio of the damper. In this test, one end of the damper was suitably mounted while the other was subjected to an initial displacement and suddenly released. The subsequent free vibration of the damper was recorded and analysed. It was found that the damper had an average natural frequency of about 6.2 Hz and damping ratio of 5%. For this particular application as shown in figure 7(b), the false floor cavity is 150 mm deep and installation of all dampers including pre- and post-testing and tuning took just one day.

4.2 FE analysis of floor with dampers

An FE model of the floor with damper elements representing the TMD system was created. The stiffness coefficient and damping coefficient of the

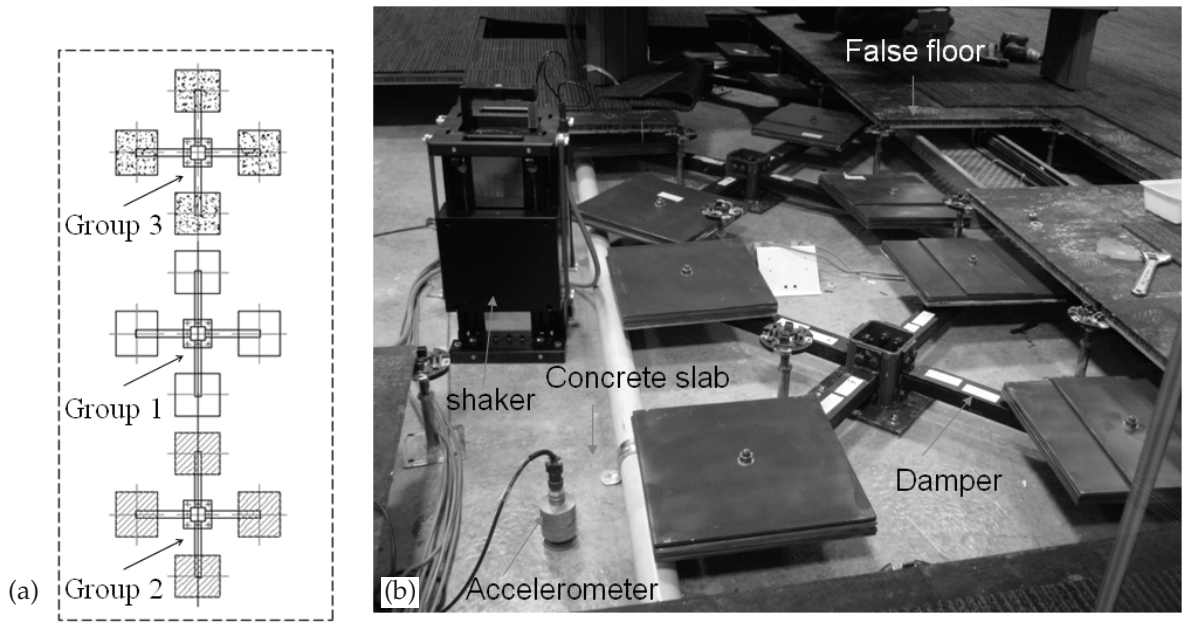

Figure 7: (a) Schematic plan view of TMD system, and (b) dampers installed on the real floor.

damper elements were calculated from the mass, natural frequency and damping ratio of the proposed TMD system. The new resonant frequencies of the damper-fitted floor bay as obtained from FE analysis were 5.8 and 6.6 Hz. The critical step frequencies whose third harmonics coincided with the damper-fitted floor frequencies were then inserted into the forcing function of equation (1). The same walker's weight of 800 N was used as for the cases of the original floor and stiffened floor models. Time history analyses of the retrofitted floor subjected to walking excitations were carried out to obtain the response history from which the peak response value can be extracted. The response to walking in worst case of the damper-fitted floor was estimated at 0.38% *g*, as can be seen in figure 8(a). This response level is about half of that calculated for the original floor as shown in figure 5(a). Furthermore, FE analysis of the floor revealed that the TMD system did not adversely affect the strength capacity of the floor structure. The self-weight of the TMDs was found to increase the bending moment due to dead and live loads of the floor beams by only 0.8% while the corresponding number for the floor girders was even smaller, just 0.4%.

4.3 Field tests

A number of walking tests with pacing rates of around 1.9-2.2 Hz was performed on the real floor fitted with dampers. The measured peak response was in the range of 0.3-0.4% *g*. Figure 8(b) shows a typical measured acceleration trace. Both FE simulation and field testing revealed that the walking response of the damper-retrofitted floor was well below the threshold of 0.5% *g*, ie. the floor is deemed acceptable in terms of human comfort.

The effectiveness of the damper system was further validated by a series of tests using an external electrodynamic shaker. The shaker generated a swept sine excitation with a forcing frequency bandwidth of 5-7 Hz that covered the natural frequencies of the floor with and without dampers. The adopted frequency range can also match the forcing frequency corresponding to the third harmonic of a walking excitation at normal and fast speed that the floor may experience. For comparison purpose, a similar loading scenario in terms of frequency, magnitude and exciting duration was applied to the floor before and after the installation of the dampers. Figure 9 shows the measured floor response in both time and frequency domains. Comparing the response spectra of figure 9(b) obtained before and after the dampers installation reveals that the sharp peak associated with the response of the original floor was lowered and flattened by the dampers. Moreover, it can be seen from figure 9(a) that the dampers reduced the peak floor acceleration by about 40%.

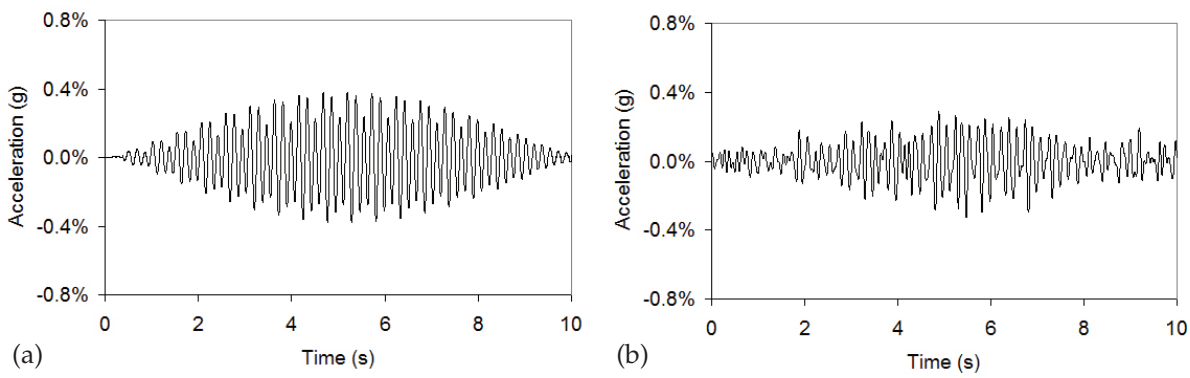

Figure 8: Response of floor with TMDs to walking – (a) predicted from FE model, and (b) measured from field tests.

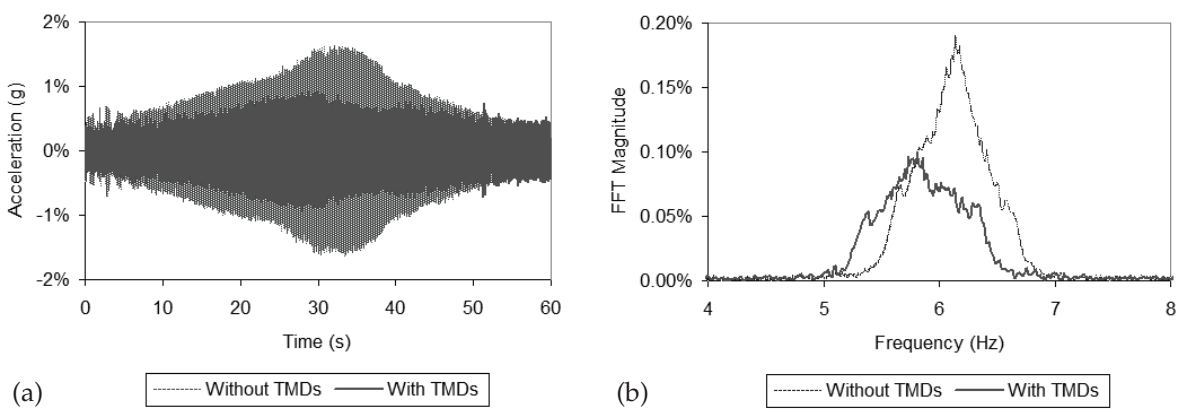

Figure 9: Floor response to shaker excitation with a swept sine wave from 5 to 7 Hz – (a) time domain, and (b) frequency domain.

5 PROBABILISTIC EVALUATION OF PERFORMANCE OF TUNED MASS DAMPERS

5.1 Simplified analysis model for floor with TMDs

A MATLAB program was developed to analyse a simplified model for the floor with dampers, where random values as specified in section 5.2 can be assigned to the input parameters. The floor bay under consideration can be simplified as a single degree of freedom (SDOF) system characterised by the modal properties of the floor. Figure 10 shows a combined system consisting of the floor as a primary structure attached to n TMDs and subjected to a time dependent force $F(t)$.

The governing equation of motion of the general system above can be written as:

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{C}\dot{\mathbf{x}} + \mathbf{K}\mathbf{x} = \mathbf{f} \tag{2}$$

where the mass matrix **M**, stiffness matrix **K**, damping matrix **C**, displacement vector **x** and forcing vector **f** are expressed as follows (Yamaguchi & Harnpornchai, 1993):

$$\mathbf{M} = \begin{bmatrix} m_s & 0 & 0 & \dots & 0 \\ 0 & m_{d1} & 0 & \dots & 0 \\ 0 & 0 & m_{d2} & \dots & 0 \\ \dots & \dots & \dots & \dots & \dots \\ 0 & 0 & 0 & \dots & m_{dn} \end{bmatrix} \tag{3}$$

$$\mathbf{K} = \begin{bmatrix} k_s + \Sigma k_{di} & -k_{d1} & -k_{d2} & \dots & -k_{dn} \\ -k_{d1} & k_{d1} & 0 & \dots & 0 \\ -k_{d2} & 0 & k_{d2} & \dots & 0 \\ \dots & \dots & \dots & \dots & \dots \\ -k_{dn} & 0 & 0 & \dots & k_{dn} \end{bmatrix} \tag{4}$$

$$\mathbf{C} = \begin{bmatrix} c_s + \Sigma c_{di} & -c_{d1} & -c_{d2} & \dots & -c_{dn} \\ -c_{d1} & c_{d1} & 0 & \dots & 0 \\ -c_{d2} & 0 & c_{d2} & \dots & 0 \\ \dots & \dots & \dots & \dots & \dots \\ -c_{dn} & 0 & 0 & \dots & c_{dn} \end{bmatrix} \tag{5}$$

$$\Sigma k_{di} = k_{d1} + k_{d2} + \dots + k_{dn} \tag{6}$$

$$\Sigma c_{di} = c_{d1} + c_{d2} + \dots + c_{dn} \tag{7}$$

$$\mathbf{x} = \{x_s \;\; x_{d1} \;\; x_{d2} \;\; \dots \;\; x_{dn}\}^T \tag{8}$$

$$\mathbf{f} = \{F \;\; 0 \;\; 0 \;\; \dots \;\; 0\}^T \tag{9}$$

In these expressions m, k, c and x are the mass, stiffness coefficient, damping coefficient and displacement, with subscripts s and d referring to the primary structure and the TMD, respectively. The stiffness coefficient and damping coefficient can be computed from the mass, natural frequency and damping ratio using established formulae given in textbooks of structural dynamics (Clough & Penzien, 1993).

To evaluate the effectiveness of the TMD system, the response to walking needs to be calculated for both the floor with dampers and the original floor. For the floor without dampers, which is simplified as a SDOF system, the matrix differential equation (2) becomes the following scalar differential equation:

$$m_s\ddot{x}_s + c_s\dot{x}_s + k_s x_s = F(t) \tag{10}$$

5.2 Input parameters for random vibration analysis

5.2.1 *Gait data*

A biomedical research program has been conducted to investigate the basic spatial and temporal gait measures of about 900 participants of which 90%

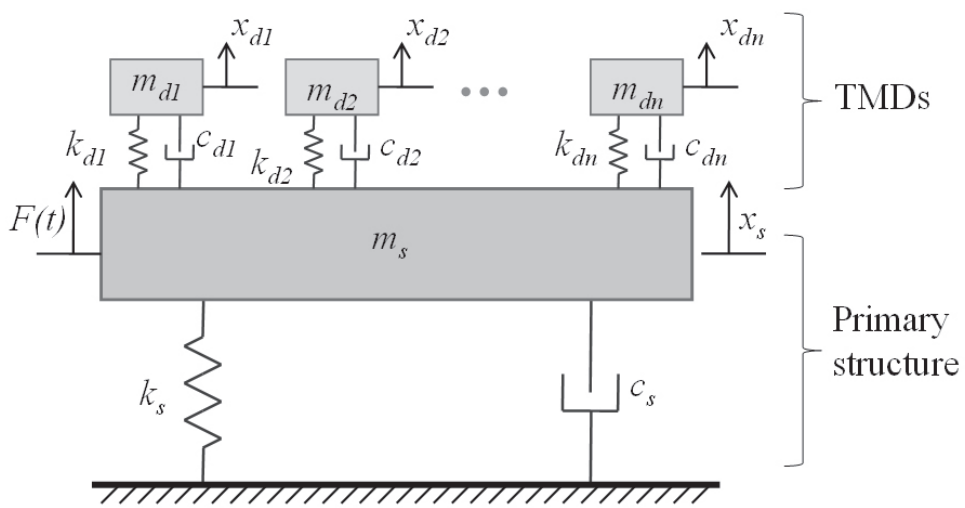

Figure 10: SDOF primary structure combined with multi TMDs.

were primary school-aged children in Australia. The test subjects completed a series of walks at self-selected free (normal), fast and slow gait speeds across electronic walkway systems (Lythgo et al, 2011). The measured data relating to a sample of 90 healthy young adults were statistically analysed to determine some parameters contributing to the characterisation of walking force (Nguyen et al, 2011). The mean step frequency for normal walk was found to be 1.98 Hz with a standard deviation of 0.13 Hz. To investigate intra-subject variability, the standard deviation in the gait parameter was also determined for each test subject, resulting in a standard deviation of up to 0.08 Hz for the step frequency for a single walker. This figure was based on 95% confidence for the sample investigated.

5.2.2 Random simulation

A large number of Monte Carlo simulations were performed to probabilistically predict the floor response. Random values were used with the specified limits below:

1. Modal mass of the floor. The floor natural frequencies could decrease or increase in accordance with an increase or decrease in the floor mass due to the possible range of service loads that the floor would experience. Design guidelines usually recommend a value of 10% to 20% of the nominal live load being considered as contribution to the floor mass when performing dynamic analysis (Murray et al, 2003; Hechler et al, 2008; European Commission, 2006). Using FE modal analysis of the floor, it was estimated that the floor modal mass could be in the range of 19,000 to 23,000 kg due to such a change in the effective live load. This range for modal mass was hence used in the random simulation. The change in modal mass translated to a variation in the natural frequency of approximately between 5.9 and 6.5 Hz.
2. Damping ratio of the floor. Most design guidelines and the relevant literature would estimate the damping ratio of composite floors to be in the order of 2% to 3%. The damping ratio measured on the case study floor was also found to be within this range.
3. Walking force function $F(t)$. Random values for the walker's weight P and step frequency f_p were to be used in the forcing function of equation (1). Design guides usually suggest using a design value of 700-800 N for P (Bachmann & Ammann, 1987; Murray et al, 2003). The random simulation presented in this paper assumed a wider range of 650 to 850 N for the walker's weight. To take into account the inter- and intra-subject diversity in gait parameter, firstly a "basic" step frequency at normal walk was randomly selected. Generally, different basic step frequencies can be assumed in different computer runs corresponding to different walk activities. This was followed by the generation of a set of step frequencies for all footsteps constituting a walking activity from one end of the floor span to the other. These step frequencies vary around the previously selected basic step frequency with a standard deviation of 0.08 Hz, as a result of the intra-subject variability mentioned in section 5.2.1. Figure 11(a) illustrates a simulation case for the step frequency where the floor frequency was randomly selected at 5.92 Hz. It can be seen that a perfect resonance condition would not occur because the randomly generated step frequencies may differ from the theoretical resonant step frequency of 1.97 Hz whose third harmonic matches the floor frequency. Figure 11(b) shows the walking force history calculated using equation (1) for this simulation case in which the walker's weight was randomly taken as 742 N.
4. Damping ratio and natural frequency of the dampers. Although the dampers were dimensioned to have a natural frequency of about 6.2 Hz and damping ratio of approximately 5%, testing of prototype dampers showed that variations in the dynamic characteristics between different dampers can be expected because of tolerances in their manufacture. In the random simulation, the modal mass of each damper was assumed to be constant at 23 kg while the natural frequency and damping ratio can be different

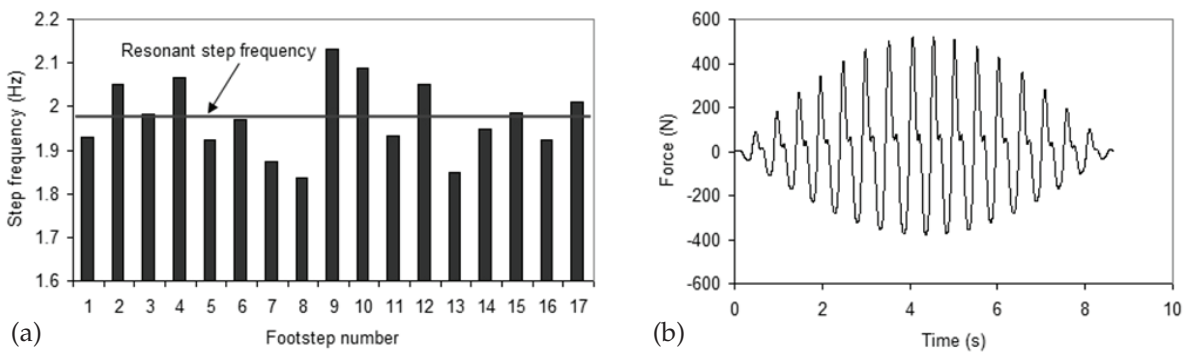

Figure 11: Example of step frequency and walking force for a walk activity – (a) random variation in step frequency, and (b) walking force time history

between 12 dampers and between simulation cases. The natural frequency of a damper was randomly taken in the range of 6.0-6.4 Hz while the damping ratio was assumed to vary within 4.5% and 5.5%.

5.3 Results and discussions

A total of 500,000 different cases for input parameters were created using the MATLAB code. The floor dynamic properties, dampers dynamic properties, and walking force were randomly determined for each computer run, based on the specified ranges in section 5.2.2. Subsequently, a numerical integration method (Clough & Penzien, 1993) was used to solve equations (2) and (10) for the response history of the floor with and without dampers, respectively. The maximum acceleration associated with each resultant acceleration history was then collected.

The response solutions from the 500,000 cases were statistically analysed from which the histograms showing the frequency distribution of the peak acceleration response is obtained as figure 12. The response level of the floor without dampers was found to exceed the acceptable limit of 0.5% *g* in many analysis cases, as can be seen in figure 12(a). In contrast, figure 12(b) reveals a response level of 0.3% to 0.4% *g* for the floor with dampers, in most analysis cases. This vibration level is well within the acceptable limit.

The cumulative probability of the floor response can be expressed by figure 13. It can be seen from figure 13(a) that the 90% fractile peak acceleration of the floor without dampers was 0.67% *g*, which exceeds the threshold of 0.5% *g*. Hence the original floor can be classified as unacceptable in terms of human comfort as per the AISC/CISC DG11. On the other hand, the 90% fractile peak acceleration of the floor with dampers was found to be 0.38% *g* as shown in figure 13(b). If the 95% fractile acceleration was of interest then the corresponding values would be 0.70% and 0.40% *g* for the floor without and with

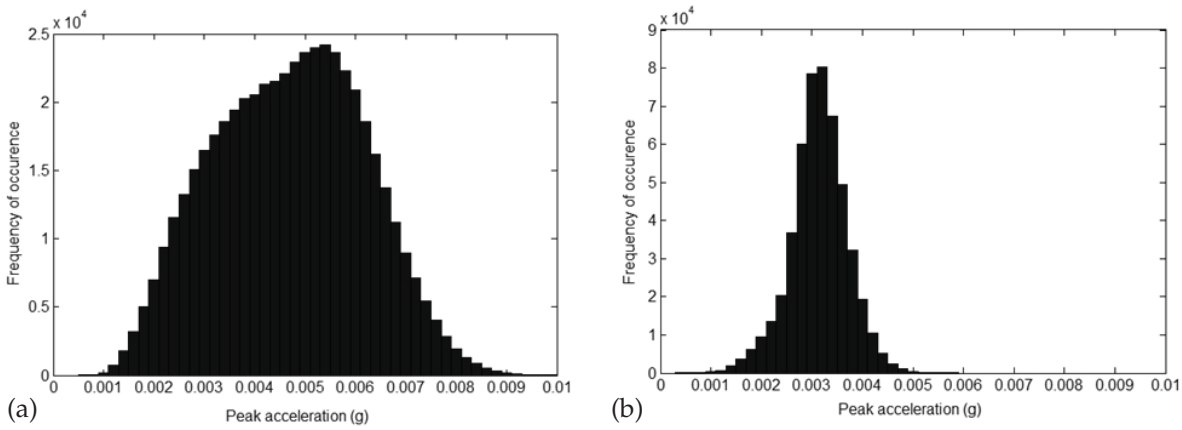

Figure 12: Frequency distribution of peak floor acceleration (500,000 samples) – (a) without TMDs, and (b) with TMDs.

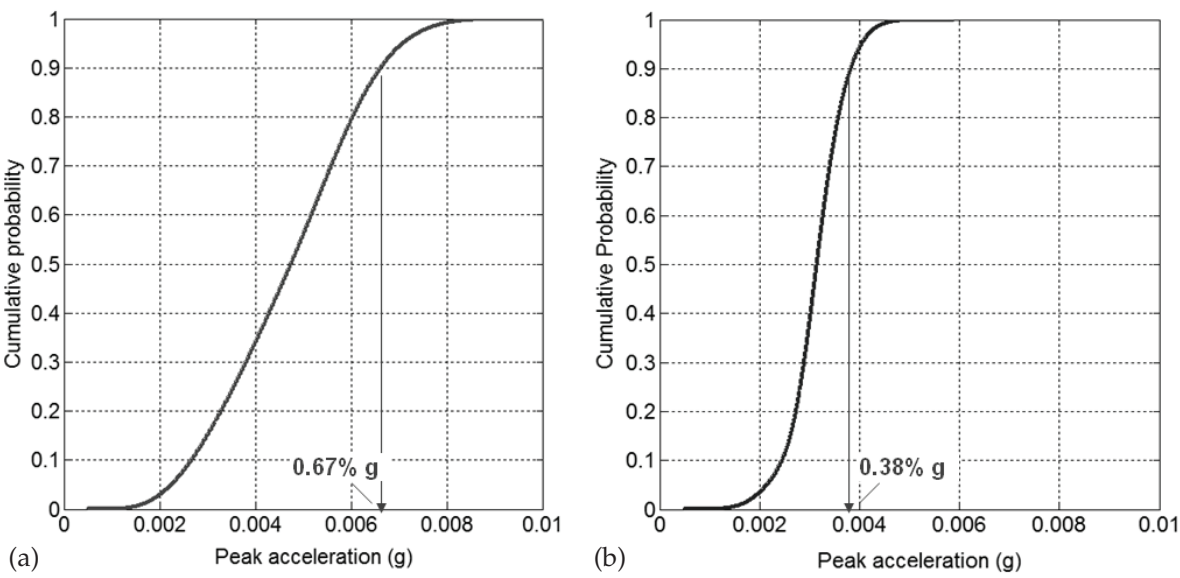

Figure 13: Cumulative probability distribution of peak floor acceleration – (a) without TMDs, and (b) with TMDs.

dampers, respectively. It is clearly seen that the TMD system can successfully reduce the floor response to a tolerable level.

6 CONCLUSIONS

This paper provided a review of current methods of mitigating human-induced floor vibrations. In particular, the paper investigated and compared two remedial measures for a real building floor subjected to annoying vibrations due to human walking. It was found that both methods can enhance the dynamic performance of the floor. Stiffening the floor beams and girders with steel cover plates would increase the floor frequency and modal mass and thus reduce the floor response. Some significant drawbacks associated with this traditional treatment include access to the ceiling space, jacking up the floor and extensive overhead welding. On the other hand, the innovative distributed multi TMD system is a much more attractive and non-intrusive remedial technique when attempted on an existing floor. The amount of steel material required for the damper option (280 kg) is less than 20% of that for the stiffening technique (1470 kg) that provides comparable effectiveness in reducing the floor response. The presented probability-based analysis of floor vibration has considered the likely variations in floor mass, floor damping, TMDs stiffness, TMDs damping, walker's weight and footstep frequency. The effect of damper off-tuning was hence automatically covered by this analysis. Forced-vibration testing revealed a 40% reduction in response due to the addition of the dampers to the floor. Results from all investigations including FE analysis with predetermined parameters, probabilistic analysis with random input parameters, and field measurements all demonstrated that the custom-made damper system can successfully alleviate the floor vibration to a tolerable level for human comfort.

REFERENCES

Allen, D. L. & Swallow, J. C. 1975, "Annoying Floor Vibrations – Diagnosis and Therapy", *Journal of Sound and Vibration,* Vol. 9, No. 3, pp. 12-17.

Bachmann, H. 1992a, "Case Studies of Structures with Man Induced Vibrations", *Journal of Structural Engineering – ASCE,* Vol. 118, pp. 631-647.

Bachmann, H. 1992b, "Vibration upgrading of gymnasia, dance halls and footbridges", *Structural Engineering International,* Vol. 2, No. 2, pp. 118-124.

Bachmann, H. & Ammann, W. 1987, *Vibrations in structures: induced by man and machines,* IABSE-AIPC-IVBH, Zurich, Switzerland.

Clough, R. W. & Penzien, J. 1993, *Dynamics of structures*, McGraw-Hill, New York.

Collette, F. S. 2002, "Tuned Mass Dampers for a suspended structure of footbridges and meeting boxes", *Proceedings of the Footbridge 2002 Conference, Design and Behaviour of Footbridges*, Paris.

European Commission, 2006, *Generalisation of criteria for floor vibrations for industrial, office, residential and public building and gymnastic halls*, RFCS Report EUR 21972 EN, Luxembourg.

Hanagan, L. & Murray, T. 1997, "Active control approach for reducing floor vibrations", *Journal of Structural Engineering – ASCE,* Vol. 123, No. 11, pp. 1497-1505.

Hechler, O., Feldmann, M., Heinemeyer, C. & Galanti, F. 2008, "Design Guide for Floor Vibrations", *EuroSteel 2008 Conference*, Graz, Austria.

Lenzen, K. H. 1966, "Vibration of Steel Joist-Concrete Slab Floors", *Engineering Journal – AISC,* Vol. 3, pp. 133-136.

Lythgo, N., Wilson, C. & Galea, M. 2011, "Basic gait and symmetry measures for primary school-aged children and young adults. II: Walking at slow, free and fast speed", *Gait & Posture,* Vol. 33, No. 1, pp. 29-35.

Murray, T. M., Allen, D. E. & Ungar, E. E. 2003, *Design Guide 11, Floor Vibrations Due to Human Activities*, American Institute of Steel Construction (AISC) and Canadian Institute of Steel Construction (CISC).

Nguyen, T., Gad, E., Wilson, J., Lythgo, N. & Haritos, N. 2011, "Evaluation of footfall induced vibration in building floor", *Proceedings of the 2011 Australian Earthquake Engineering Society AEES conference*, South Australia.

Nguyen, T. H., Gad, E. F., Wilson, J. L. & Haritos, N. 2012, "Improving a current method for predicting walking-induced floor vibration", *Steel and Composite Structures,* Vol. 13, No. 2, pp. 139-155.

Post, N. M. 1997, "Annoying Floors", *Engineering News Record,* Vol. 1997 ENR, pp. 28-33.

Reynolds, P., Díaz, I. M. & Nyawako, D. S. 2009, "Vibration testing and active control of an office floor", *Proceedings of the IMAC-XXVII conference*, Florida, USA.

Setareh, M. & Hanson, R. 1992, "Tuned mass dampers for balcony vibration control", *Journal of Structural Engineering – ASCE,* Vol. 118, No. 3, pp. 723-740.

Setareh, M., Ritchey, J., Murray, T., Koo, J. & Ahmadian, M. 2007, "Semiactive tuned mass damper for floor vibration control", *Journal of Structural Engineering – ASCE,* Vol. 133, No. 2, pp. 242-250.

Shope, R. & Murray, T. 1995, "Using tuned mass dampers to eliminate annoying floor vibrations", *Proceedings of Structures Congress XIII – ASCE*, Boston, Massachusetts, pp. 339-348.

Smith, A., Hick, S. & Devine, P. 2009, *Design of Floors for Vibration: A New Approach (Revised edition),* SCI Publication P354, Ascot, UK.

Thornton, C. H., Cuoco, D. A. & Velivasakis, E. E. 1990, "Taming Structural Vibrations", *Civil Engineering – ASCE,* Vol. 60, No. 11, pp. 57-59.

Velivasakis, E. 1997, "LaGuardia High School Gymnasium, Tuned Mass Dampers 'Dance to the Tune' to Tame Floor Vibrations", *Forensic Engineering: Proceedings of the First Congress – ASCE,* Minneapolis, pp. 198-207.

Webster, A. & Vaicaitis, R. 1992, "Application of tuned mass dampers to control vibrations of composite-floor systems", *Engineering Journal of the American Institute of Steel Construction,* Vol. 29, No. 3, pp. 116-124.

Willford, M. & Young, P. 2006, *A Design Guide for Footfall Induced Vibration of Structures,* The Concrete Society Publication CCIP-016, Trowbridge, UK,

Yamaguchi, H. & Harnpornchai, N. 1993, "Fundamental characteristics of multiple tuned mass dampers for suppressing harmonically forced oscillations", *Earthquake Engineering & Structural Dynamics,* Vol. 22, No. 1, pp. 51-62.

TUAN NGUYEN

Tuan Nguyen is a research assistant at Swinburne University of Technology in Melbourne. He holds a Bachelor of Engineering (Civil) from HCMC University of Architecture (Vietnam), a Master of Engineering (Civil) from HCMC University of Technology (Vietnam) and a PhD from Swinburne University of Technology. He had been working as an academic at HCMC University of Architecture prior to his PhD candidature. His research interests are in the area of structural dynamics and modelling, particularly in the design and control of floor systems for human-induced vibrations.

EMAD GAD

Emad Gad is a Professor in Civil Engineering at Swinburne University of Technology in Melbourne. Prior to this appointment he worked as a senior academic at Melbourne University and research scientist at CSIRO. He has a Bachelor of Engineering (Civil) from Monash University and a PhD from Melbourne University. Emad has research interest and expertise in structural dynamics and modelling. In addition to teaching and research he has also been involved in specialist consulting. He is a Fellow of Engineers Australia and past Chairman of the Structural Branch, Victoria Division.

JOHN WILSON

John Wilson is Dean of Engineering and Industrial Sciences at Swinburne University of Technology in Melbourne. Prior to joining Swinburne in 2005, he was an academic at the University of Melbourne for some 14 years and as a consulting engineer for over 10 years with the SECV and Arups in their London and Melbourne offices. He has a Bachelor of Engineering degree from Monash University, a Master of Science degree from University of California (Berkeley) and a PhD from University of Melbourne. He has a research interest and expertise in structural systems, earthquake engineering, structural dynamics and sustainable structures and has consulted widely in these fields.

NICHOLAS HARITOS

Nicholas Haritos has been a long-standing academic in Civil/Structural Engineering at The University of Melbourne, retiring from the post of Reader and Associate Professor at this institution in 2010. He is now an honorary Principal Fellow in Infrastructure Engineering at The University of Melbourne and a Stage 2 National Assessor with Engineers Australia. He is also currently an Adjunct Professor at Swinburne University of Technology and a sessional staff member at Victoria University in Structural Engineering. Nick's research and consulting interests have centred on the structural dynamics of land and offshore structures and on the capacity assessment of bridges.