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Tuned mass damper effects on the response of multi-storied structures observed in geotechnical centrifuge tests

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ABSTRACT

Tuned mass dampers (TMDs) are widely used to reduce vibrations in structures. However, very little research is available on the experimental investigation of TMDs and their performance in soil-structure systems. In this paper, a series of geotechnical centrifuge tests was conducted to investigate the effects of TMDs on the response of a multiple-storey sway frame structure undergoing dynamic soil-structure interaction (SSI). Structural responses were recorded for a wide range of input motion characteristics, damper configurations and soil profiles. The practicality associated with the use of TMDs in the damping of resonant structures in light of unexpected earthquake characteristics different from design earthquakes was experimentally demonstrated. Tuning a TMD to soil-structure system properties rather than fixed-base structural properties was found to double the improvement in damping and reduce the original peak response by nearly half. The potential effectiveness of a de-tuned mass damper in light of significant SSI was also demonstrated.

1. Introduction

A popular method of mitigating risks from earthquakes to structures is the use of vibration resisting devices. A tuned mass damper (TMD) is one of the simplest and most reliable vibration control devices in existence today and has been widely installed in many structures around the world [1]. It operates through the dissipation of vibrational energy induced in a structure, which is achieved through the combined action of inertial dissipation and material damping [2].

The overwhelming majority of TMDs in use today are linear and passive in nature, the latter meaning that they are not externally driven but that they react solely in response to the motion of the floor in which they are installed. Passive linear TMDs are well understood and have been shown to be very effective and reliable in practice [3]. In the case of fixed-base structures, tuning of the natural frequency of the TMD to the pre-dominant modal frequency of the structure is desired to ensure the damper's optimum operational efficiency [4]. In reality however, inclusion of soil flexibility is expected to result in an overall decrease in stiffness and a different natural frequency of the soil-structure system in comparison to the fixed-base structure [5]. By means of shaking table testing under 1 g conditions Jabary and Madabhushi [6] experimentally demonstrated that TMD performance is optimum when the natural frequency of the TMD is tuned to the pre-dominant modal frequency of the soil-structure system.

With the aim of reducing one or more structural response parameters, past studies into TMDs have focused on the development of analytical expressions for the optimisation of the TMD parameters mass, stiffness and damping. Occasional parametric verifications of such analytical expressions have made reference to very specific model structures with a limited number of defined variables in structural and soil properties. Studies into the response of structures considering their interaction with the foundation soil and the TMD have been performed by several authors [2,7,8]. However, prior to Ghosh and Basu [9] very few authors had looked into the effects of altered structural properties as a result of soil-structure interaction (SSI) on the performance of TMDs in seismic vibration control. Ghosh and Basu [9] investigated SSI effects on the TMD performance in a single-degree-of-freedom structure. Nevertheless, their numerical study was based on many simplifications, most notably their assumption of linearity of the soil's stress–strain behaviour. Furthermore, the efficiency with which TMDs operate in practice is often reduced considerably compared to theoretically developed responses [10]. In addition to these general drawbacks associated with theoretical studies that have been conducted on TMD performance, the bulk of such

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studies considered the long-established use of TMDs in wind-excited structures, with the use of TMDs in seismically-excited structures being a relatively new concept that has not been as extensively explored [11,12].

The aim of this paper is to overcome limitations of theoretical analyses of TMD performance through the experimental investigation of TMD effects on the response of a multiple-storey sway frame structure undergoing dynamic SSI. No such studies have been performed to date using geotechnical centrifuge testing. For the purpose of this study a series of centrifuge tests was conducted on a range of structure–damper configurations. Two different soil profiles and multiple earthquake scenarios were recreated by subjecting various soil-structure models to an extensive range of input motion characteristics.

2. Geotechnical centrifuge modelling

2.1. Principle of centrifuge modelling

Since soil behaviour is highly non-linear and sensitive to stress level, testing under increased gravitational fields may be performed to accurately recreate prototype stresses and strains in a small-scale experimental model. This may be achieved in a geotechnical centrifuge. The need for geotechnical centrifuge modelling arises when the constitutive behaviour of the soil is not fully known or if there is uncertainty about the mechanisms of failure to expect under a given set of loading conditions. In such scenarios physical modelling is preferred to Finite Element Analysis. Contrary to full-scale field testing which is often very expensive and may not be practical when dealing with earthquake related problems, scaled models can be put to effective use in understanding the behaviour of an idealised prototype as described by Madabhushi [13].

The centrifuge tests carried out for this paper were conducted using the Turner beam centrifuge at the Schofield Centre in Cambridge, which is a 10 m diameter 150 g-ton centrifuge. A stored angular momentum (SAM) actuator device was used to simulate a wide range of earthquakes characteristics through shaking a specially designed model container in one lateral direction [14]. The equivalent shear beam (ESB) model container has been used extensively in many centrifuge tests at Cambridge and its performance was compared to a laminar model container by Brennan et al. [15]. The ESB consists of 10 rectangular aluminium rings interspersed by rubber layers. The step-like deformation of the ESB container during shaking limits the restrained soil movement and minimises the reflection of energy from boundary walls to simulate the seismic energy that would radiate away in the field [16].

2.2. Scaling laws

A centrifuge model in flight is subjected to an increased gravitational field which is the product of 1 g and a geometrical scaling factor, N, to which model dimensions are scaled down relative to the prototype. Scaling laws defining the relationships between model and prototype response parameters were derived by Schofield [17] and are shown in Table 1. Unless otherwise stated, parameters presented in this paper are in prototype scale.

Table 1
Scaling laws for centrifuge testing [17].

Parameter	Model/prototype	Dimensions
Length	1/N	L
Time (dynamic)	1/N	T
Time (seepage)	$1/N^2$	T
Mass	$1/N^3$	M
Velocity (dynamic)	1	LT^{-1}
Velocity (seepage)	N	LT^{-1}
Acceleration	N	LT^{-2}
Strain	1	$ML^{-1}T^{-2}$
Stress	1	$ML^{-1}T^{-2}$
Frequency	N	T^{-1}
Area	$1/N^2$	L^2
Volume	$1/N^3$	L^3

2.3. Model

2.3.1. Structure

Sway frame structures are simplified representations of real structures in terms of their horizontal sway behaviour. The centrifuge model structure under consideration for this study is a two-degrees-of-freedom sway frame structure with space for the installation of an adjustable damper on its upper floor. The walls of the structure were slotted tightly into the floors to ensure a high degree of fixity throughout testing. A schematic illustration outlining the dimensions of the structure and an image of the constructed model are shown in Fig. 1(a) and (b). Centrifuge tests were conducted at 50 g, meaning that the model structure with the inclusion of its base foundation resembles a prototype structure of 7.5 m in height. However, no pre-existing or design prototype structure was borne in mind for the design of the small-scale centrifuge model. Instead, the model was used to study the effects of different parameters on a general multiple-degrees-of-freedom sway frame structure.

The side walls and floors of the sway frame model were made out of aluminium alloy 6082-T6 ($E = 70$ GPa, $\sigma_y = 255$ MPa and $\rho = 2700\ kg/m^3$) and the TMD components were made out of steel grade 43 ($E = 210$ GPa, $\sigma_y = 275$ MPa and $\rho = 7840\ kg/m^3$).

The sway frame model has a fixed-base fundamental frequency of 35 Hz (0.7 Hz for the corresponding prototype modelled at 50 g). Post-centrifuge impulse testing on the model structure showed that it had retained the same fixed-base frequency as determined in the pre-testing phase, thus indicating that the model structure experienced linear-elastic behaviour throughout testing.

The structural response parameters considered in this paper are floor acceleration and floor displacement. Another practical parameter is the mass ratio (μ) between the TMD and the structure. For real structures the mass ratios are typically less than 10% for economic reasons [18]. As a result, optimised mass ratios are rarely found in practice. However, for this research the mass ratio $\mu = 27\%$ was used.

2.3.2. Soil

Dry soil conditions were tested in the centrifuge, given that the effectiveness of TMDs in reducing the peak structural response during seismic loading relies on the absence of drastic changes in soil (e.g. liquefaction). The soil under consideration was fine-grained siliceous Hostun sand (HN31). The properties of this sand are outlined in Table 2 [19].

The two soil profiles that were tested consisted of a loose ($D_r = 50\%$) homogeneous bed and a dense–loose–dense ($D_r = 85\%,\ D_r = 50\%,\ D_r = 85\%$) layered bed of varying layer depths. The structure's footing was rested on the soil surface in each case. The total soil depth was consistently modelled to resemble 18.5 m in prototype scale. The relative density of the loose bed was designed to be sufficiently low in order to investigate damping effects on structural response when soil damping is significant. In comparison, the layered bed was designed to better resemble the natural variability of soil conditions with depth

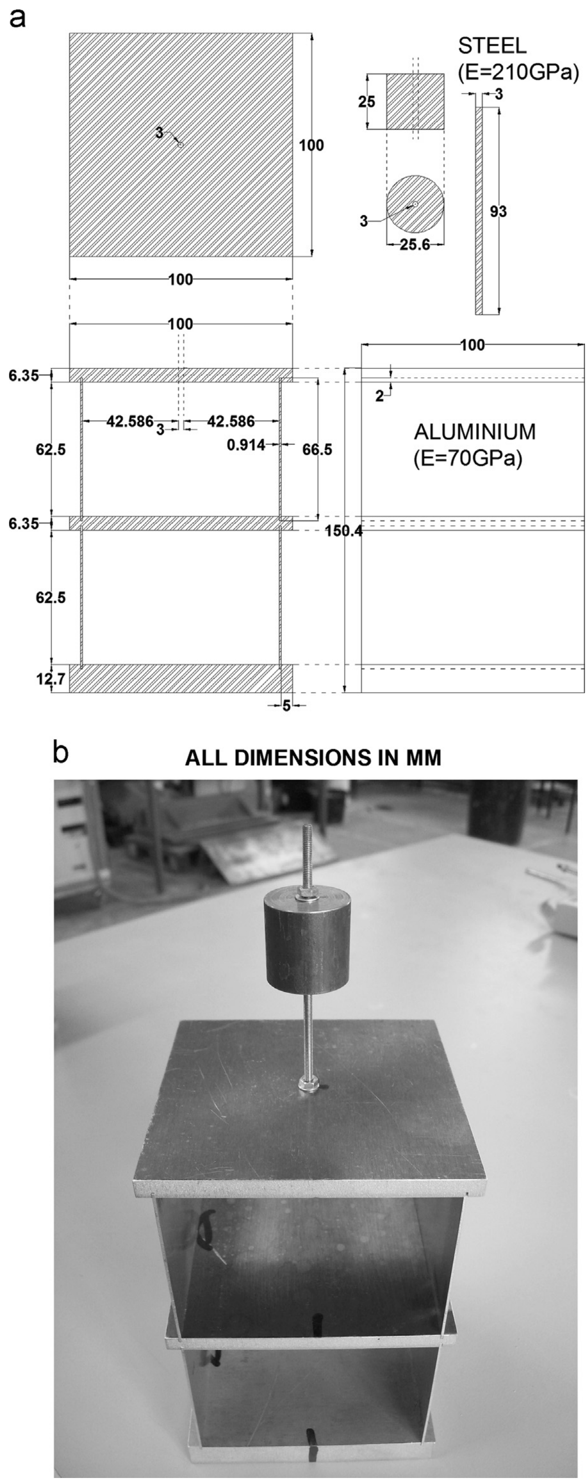

Fig. 1. Centrifuge model dimensions and the constructed centrifuge model.

Table 2
Hostun sand (HN31) properties [19].

Property	Value
d_{10}	0.315 mm [19]
d_{50}	0.480 mm [19]
d_{60}	0.525 mm [19]
G_s	2.65
e_{min}	0.555
e_{max}	1.041

found in reality, in which stiff layers which are generally deemed favourable for construction purposes may be underlain by looser layers. The layer thicknesses of the soil profile were carefully selected in order to avoid the scenario whereby more deeply embedded soil strata would have no role in influencing structural response. The thicknesses of the two upper layers were therefore kept limited, and in particular that of the uppermost dense layer. Taking into account a characteristic dimension (b) for the width of the prototype footing of 5 m, the layers were poured to thicknesses of $0.5 \times b$, b and $2.2 \times b$ using sand pluviation.

2.3.3. Instrumentation

Since centrifuge scaling laws apply to the model in its entirety with the inclusion of any relevant instrumentation, miniaturised transducers and cables were used in limited numbers to minimise their influence on the structural response and avoid reinforcement of the soil. In line with this and to overcome saturated transducer responses, small 120 g micro-electromechanical system (MEMS) accelerometers were installed on the structure and damper instead of bulkier piezo-electric accelerometers. Arrays of piezo-electric accelerometers were positioned in the soil underneath the structure and in the free-field. A linear variable differential transformer (LVDT) was positioned in the free-field to measure soil settlement throughout testing. The LVDT was founded on the sand surface by means of a circular pad footing to ensure a reliable measurement of soil settlement. An air hammer was installed at depth in the soil to generate shear waves. The propagation of these waves through the soil was picked up by a vertical array of piezo-electric accelerometers positioned directly above the air hammer in order to measure the shear wave velocity (v_s) and obtain an estimate for the soil stiffness.

3. Test programme

A series of centrifuge tests was conducted at 50 g to capture the effects of various tuned and de-tuned mass damper configurations on the two-degrees-of-freedom sway frame structure interchangeably positioned on two soil profiles.

Each centrifuge test consisted of multiple flights resembling a range of design earthquakes in which the structural response to unique configurations between the structure and damper was investigated, including configurations in which the damper was de-tuned away from the soil-structure system's fundamental frequency. This was done in order to resemble practical scenarios whereby changes in soil-structure system properties may directly result from an earthquake. The SAM actuator was used to produce a wide range of earthquakes in the form of harmonic input motions of constant frequency as well as frequency sweep earthquakes which roll down from a high frequency to a low frequency and cover a wide frequency spectrum. The size and sequence of earthquakes was kept mostly the same throughout all centrifuge flights in order to enable direct comparison between different tests.

The characteristics of the earthquakes fired are provided in prototype scale in Table 3 for the structure–damper configurations shown in model scale in Fig. 2.

As indicated in Table 3, the duration of all single frequency earthquakes was kept the same at 20 s. In addition to the earthquakes shown in Table 3, air hammer tests were carried out at 50 g and changes in the overall soil depth were recorded during centrifuge swing-ups ($1 \rightarrow 50$ g) and swing-downs ($50 \rightarrow 1$ g). The average shear wave velocity for the overall soil depth measured following Ghosh and Madabhushi [20] is also shown in Table 3 for each centrifuge flight. Only one test with a 'tuned' configuration is necessary to allow for a direct comparison with a 'de-tuned' configuration on the effects of structural response for the same soil

Table 3
Earthquake characteristics for the structure–damper configurations.

Loose sand				
Configuration #	**Frequency (Hz)**	**Duration (s)**	**Max. input acceleration (g)**	**Average shear wave velocity (m/s)**
1 (no TMD)	0.6	20	0.102	154
	1.0	20	0.238	
	1.2→0	80	0.350	
2 (TMD de-tuned to 0.5 Hz)	0.9	20	0.174	194
	1.2→0	80	0.356	
Dense–loose–dense sand				
Configuration #	**Frequency (Hz)**	**Duration (s)**	**Max. input acceleration (g)**	**Average shear wave velocity (m/s)**
1 (no TMD)	0.6	20	0.097	259
	1.0	20	0.262	
	0.7	20	0.149	
	1.2→0	80	0.355	
2 (TMD de-tuned to 0.7 Hz)	0.7	20	0.133	289
	1.2→0	80	0.383	
3 (TMD tuned to 0.76 Hz)	0.7	20	0.123	300
	1.2→0	80	0.388	

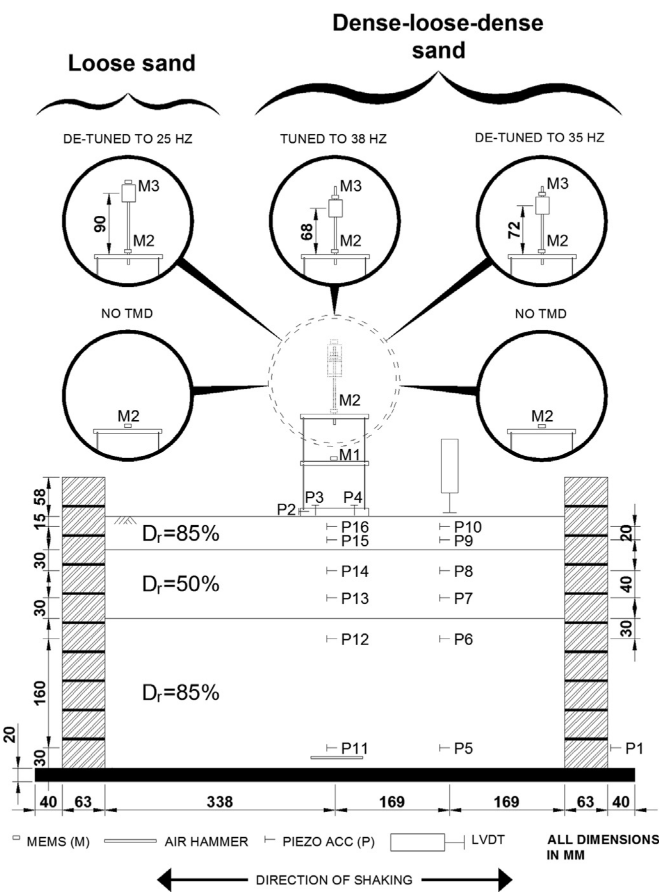

Fig. 2. Model layout for the layered sand bed and all damper configurations.

profile. A small (to minimise soil disturbance) earthquake frequency of 0.6 Hz was fired up front in each centrifuge test to compare the structural behaviour observed for the two soil profiles in the absence of external damping. An earthquake of 1.0 Hz frequency was fired at the same stage in each centrifuge test to obtain the secant shear moduli. For the most part the earthquake frequencies, magnitudes and durations were kept the same to aid comparison between different configurations.

The soil layout in Fig. 2 is specific to the dense–loose–dense sand case, though the instrumentation layout applies to both soil profiles. The loose homogeneous sand bed ($D_r = 50\%$) consists of the same total depth as that shown in Fig. 2.

4. Centrifuge test results

For the remainder of this paper the terms 'loose sand' and 'dense–loose–dense sand' shall be used in direct reference to the different soil profiles that were centrifuge tested. Structural response parameters are analysed in lateral directions. Fast Fourier Transforms (FFTs) are used to transform data from the time domain into the frequency domain. Since FFTs produce complex numbers, the moduli of the FFT ($|FFT|$) is taken to express the Fourier component on the vertical scale of the Fourier Transforms in real units. A Fourier Transform shows how much amplitude of the recorded signal is present at various frequencies. The greater the Fourier components at certain frequencies, the more dominant these frequencies are in the acceleration record. Finally, as effective tools in the analyses of non-stationary signals, combined time-frequency domain analysis in the form of harmonic wavelet transforms is used to observe the energy distribution of the structural response in the time-frequency domain [21].

4.1. Soil-structure system properties

The soil-structure system's natural frequencies were experimentally determined by subjecting the structure without an external damping device to a frequency sweep (1.2→0 Hz) which covers the soil-structure system's fundamental frequency. FFTs of the first and second floor responses to the frequency sweep were obtained for the structure rested on loose and dense–loose–dense foundations. Both FFTs were found to be very similar and one of these FFTs is shown in Fig. 3 for the structure rested on loose sand. Filtering of the accelerometer signals was carried out using an 8th order Butterworth low-pass filter with a Nyquist fraction of 0.01. As the sampling rate used was 10 kHz per channel, this filtering will attenuate all frequencies above 1000 Hz.

The system frequencies for the loose sand case were observed at 0.76 Hz and 2.308 Hz and the system frequencies for the dense–loose–dense sand case were observed at 0.766 Hz and 2.41 Hz. The layered soil profile does not seem to increase the stiffness and hence the natural frequency of the soil-structure system by very much. This may be due to the presence of a loose sand layer below the top dense layer. The thickness of the dense layer is only half the width of the base of the structure. Both the cases of the

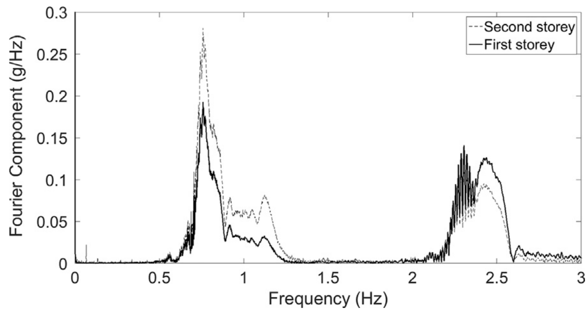

Fig. 3. FFT of structural response to 1.2→0 Hz sweep for loose sand ('no TMD' configuration).

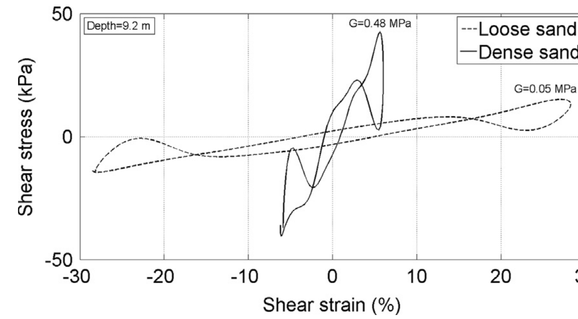

Fig. 4. Stress–strain loops for loose and dense–loose–dense sands.

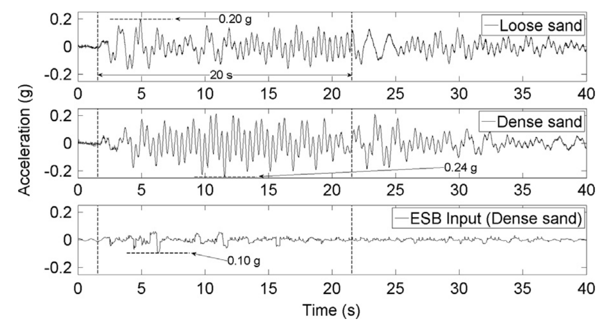

Fig. 5. Acceleration–time histories of the first floor response to a 0.6 Hz earthquake ('no TMD' configuration).

structure on uniform, loose sand and layered soil profile result in a fundamental frequency that is greater than the fixed-base frequency of the structure on its own. This stands in contrast to the expectation that additionally imposed soil presence would lower the natural frequency of the system [5,6,9]. This results from the fact that in the centrifuge test the 2-d.o.f. structure placed on the sand layer is free to undergo a combination of both rocking vibrations and sway vibrations. The structure's rocking mode of vibration has a natural frequency of 0.76 Hz due to the stiffness of the sand layer. This is being picked up in the centrifuge test and is higher than the fixed-base frequency of the structure at 0.7 Hz, which is for sway vibrations only.

In line with expectations, the system's natural frequencies for the same structure and excitation characteristics were found to be lower for the loose sand case. Since the frequency sweep covers only the fundamental frequency of the soil-structure system and not its second-mode frequency, the actual second-mode frequencies may possibly be located at slightly higher frequencies than those found in the tests.

For each of the soil profiles the widths of the frequency spectra are consistent regardless of first and second floor responses. Though – for both soil profiles – whereas the largest Fourier component around the fundamental system frequency is associated with the second floor response, the largest Fourier component around the second-mode system frequency is associated with the first floor response.

Settlement readings from an LVDT positioned on the free-field soil surface show that the maximum changes observed in relative density due to the earthquakes in any given centrifuge flight were $\Delta D_r = 5.9\%$ for the loose sand and $\Delta D_r = 2.1\%$ for the dense–loose–dense sand, though these changes are too small to render a TMD ineffective.

Using the procedure developed by Brennan et al. [22] to estimate the stiffness of the soil in centrifuge tests, a first-order approximation was applied to obtain the secant shear modulus (G) at a prototype depth of $z = 9.2$ m measured from the soil surface. Considering the response of the relatively undisturbed loose and dense–loose–dense sand deposits to a typical single frequency earthquake of 20 s duration (1.0 Hz), the secant shear moduli for typical stress–strain cycles during the earthquake were $G = 0.05$ MPa and $G = 0.48$ MPa for the loose and dense–loose–dense sands respectively. This is shown in Fig. 4. Changes in the shear moduli as a result of soil densification after successive earthquakes were found to be small ($\Delta G \approx 0.01$ MPa).

4.2. Influence of soil on structural response

The horizontal motion of the first floor of the structure was computed in the absence of an external damping device for a single frequency earthquake (0.6 Hz) to capture the influence of the soil on structural behaviour. Both the loose and dense–loose–dense soil profiles were considered. The responses are shown in Figs. 5 and 6. The start and end of the earthquakes are indicated in the acceleration–time histories by vertical dashed lines. In addition, the absolute peak accelerations are highlighted.

The horizontal acceleration motion of the second floor was found to be very similar to that shown for the first floor in Fig. 5.

Clear differences in structural response during the earthquakes and immediately upon termination of shaking can be observed in Fig. 5, especially for the loose sand. Within the first few seconds of excitation the loose sand fails to damp structural responses as much as throughout the remainder of the earthquake duration, resulting in a peak acceleration early on in the record. Generally thereafter, acceleration magnitudes for the structure rested on a loose sand foundation are lower in magnitude compared to those for the dense–loose–dense sand case.

The FFT in Fig. 6 shows significant Fourier components at the excitation frequency (0.6 Hz) and around the fundamental and second-mode frequencies of the soil-structure system.

4.3. Effect of earthquake characteristics on structural response

In order to determine and compare the significance of variations in earthquake excitation characteristics with variations in structure-TMD configurations on structural response, second floor responses were computed for cases involving both types of variations. These are shown in Figs. 7–10. The 'Tuned' cases in Figs. 9 and 10 depict the configuration whereby the TMD is tuned to the fundamental frequency of the soil-structure system.

The following observations can be made from these figures:

For the same earthquake magnitude, changes in the earthquake excitation frequency have a more profound effect on structural accelerations and displacements than variations in the damper configuration. This holds true for both soil profiles considered. As opposed to earthquake characteristics however, damper configurations can easily be modified in order to reduce the likelihood of damage to structures and provide a greater degree of safety to its

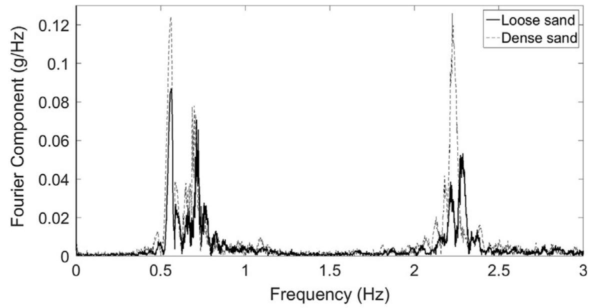

Fig. 6. FFT of the first floor response to a 0.6 Hz earthquake ('no TMD' configuration).

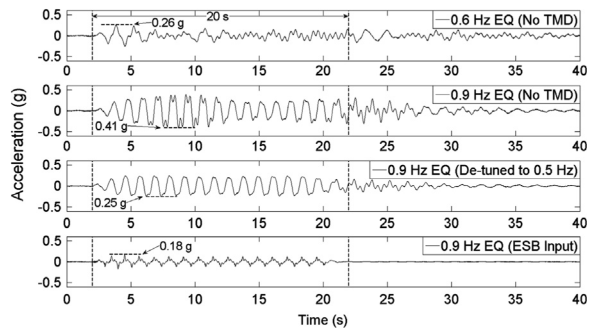

Fig. 7. Acceleration–time histories of the second floor response (loose sand) and the ESB input for varying earthquake magnitudes.

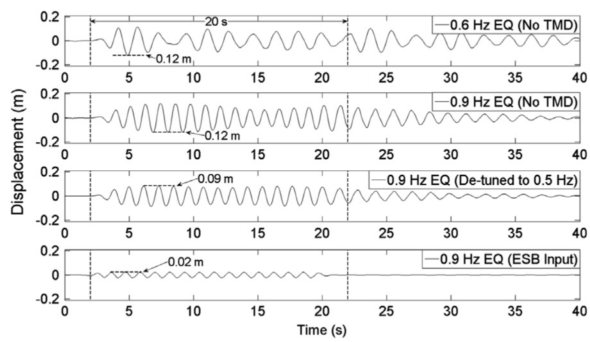

Fig. 8. Displacement–time histories of the second floor response (loose sand) and the ESB input for varying earthquake magnitudes.

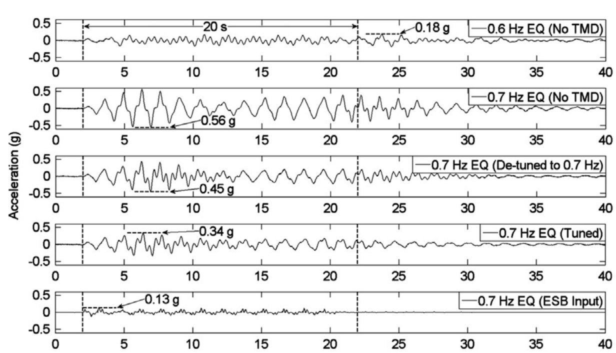

Fig. 9. Acceleration–time histories of the second floor response (dense–loose–dense sand) and the ESB input for the same earthquake magnitude.

occupants when the structure is subjected to seismic loading. From Figs. 9 and 10 in which dense–loose–dense soil conditions are considered, it is evident that it is important for the influence of soil on system properties to be taken into consideration in tuning the

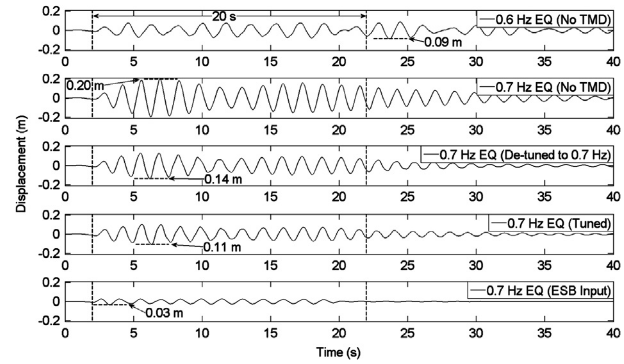

Fig. 10. Displacement–time histories of the second floor response (dense–loose–dense sand) and the ESB input for the same earthquake magnitude.

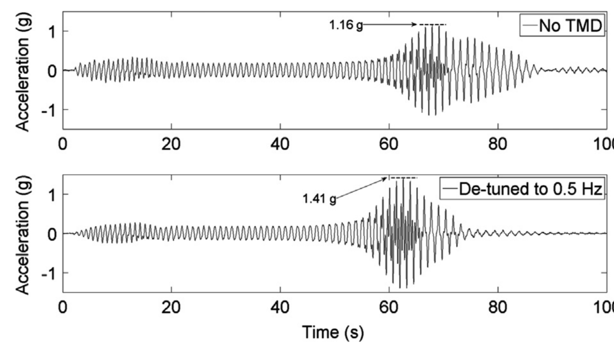

Fig. 11. Acceleration–time histories of the second floor response to a 1.2→0 Hz sweep earthquake for loose sand.

mass damper. Under dense–loose–dense soil conditions, tuning to soil-structure system properties (39% reduction in peak acceleration; 45% reduction in peak displacement) as compared to tuning to fixed-base properties (19% reduction in peak acceleration; 30% reduction in peak displacement) improves damper effectiveness in the attenuation of structural peak responses tremendously. This finding is in line with findings from recent experimental studies conducted by Jabary and Madabhushi [6] on a sway frame structure in a similar test set-up performed at 1 g.

Under loose soil conditions – albeit considering a different earthquake magnitude, thus disallowing direct comparison with previously discussed peak reductions under dense–loose–dense soil conditions – the damper was surprisingly still very effective in attenuating peak structural response (39% reduction in peak acceleration; 25% reduction in peak displacement) despite being de-tuned away from both fixed-base and soil-structure system properties. The significant extent of peak displacement reductions achieved in these cases contradicts the observations made in a study by Ghosh and Basu [9] that a TMD tuned to the fixed-base frequency is completely ineffective in reducing structural displacements when $v_s = 100\ m/s$. Although the loose foundation considered as part of this study was not as soft ($v_s = 194\ m/s$), the potential damping effectiveness of a de-tuned mass damper found in this study shows the importance of the experimental investigation of this problem compared to numerical investigations which require many soil and structural idealisations.

In line with the findings reported above, TMD effects on the response of the sway frame structure were investigated for further variations in structure–damper configurations subjected to the same earthquake events considered. Acceleration–time histories of the second floor responses to various structure–damper configurations are shown in Figs. 11 and 12 for the loose and dense–loose–dense sand foundations respectively, with the FFTs corresponding to the latter shown in Fig. 13. Different to Figs. 7–10 which considered structural responses to single frequency tone burst earthquakes, Figs. 11 and 12 consider structural responses to frequency sweep earthquakes which cover a wide frequency spectrum. As before, the 'Tuned' entries shown in these figures depict the configuration whereby the TMD is tuned to the fundamental frequency of the soil-structure system. The 'De-tuned' entries shown depict the configuration whereby the TMD is tuned to the fundamental frequency of the fixed-base structure and thus soil effects are overlooked. Harmonic wavelet transforms combining the response accelerations and frequencies for the dense–loose–dense soil foundation are shown in Figs. 14 and 15, with the darker regions indicating greater Fourier components for the frequencies.

The following observations can be made:

Investigation of the shaking input to the ESB container in various flights shows that the duration of the frequency sweep earthquake is consistently 80 s. It may therefore be concluded from Fig. 12 that alteration of the structure–damper configuration could cause a drastic change in the time at which the acceleration response peak occurs. This is the result of the system pertaining a different natural frequency and thus responding to a different frequency. This is more apparent from the response to a frequency sweep earthquake containing multiple frequencies than from the response to a single frequency tone burst earthquake.

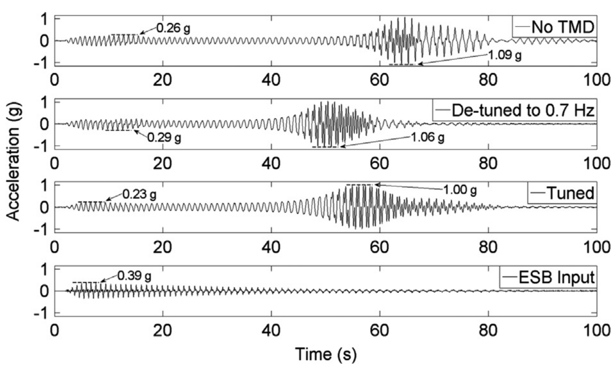

Fig. 12. Acceleration–time histories of the second floor response to a 1.2→0 Hz sweep earthquake for dense–loose–dense sand.

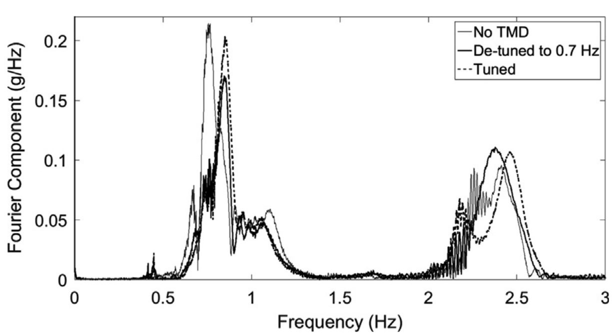

Fig. 13. FFT of the second floor response to a 1.2→0 Hz sweep earthquake for dense–loose–dense sand.

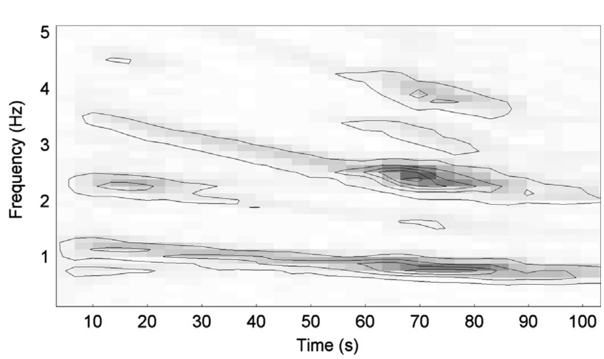

Fig. 14. Harmonic wavelet transform of the second floor response to a 1.2→0 Hz sweep earthquake for dense–loose–dense sand ('no TMD' configuration).

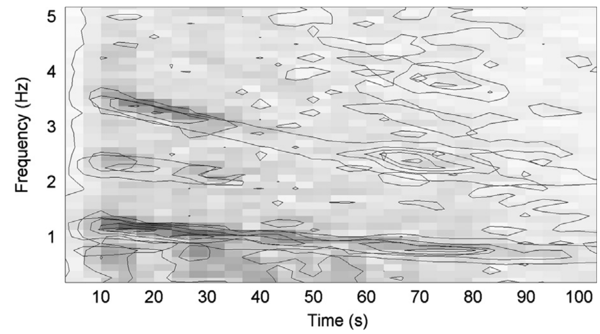

Fig. 15. Harmonic wavelet transform of the ESB container response to a 1.2→0 Hz sweep earthquake for dense–loose–dense sand.

Comparison of the ESB container input traces to structural response in Fig. 12 shows the acceleration response reduction caused by the presence of soil damping within the initial 30 s of the record. The wavelet plots in Figs. 14 and 15 show that the tremendous acceleration response amplification of the structure observed beyond 40 s is the result of the dominant frequency of excitation (ESB input) for this time period overlapping with the soil-structure system's fundamental frequency, giving rise to resonance effects. The acceleration–time histories show that the response amplification caused by resonance is clearly much more significant than the response attenuation resulting from the installation of a TMD. Nevertheless, it is evident that TMDs are still effective in reducing the response of a structure resonating with the frequency of excitation.

The FFT in Fig. 13 indicates that the installation of a damper causes shifts in the frequencies at which the most dominant Fourier components occur. This same phenomenon was observed in 1 g shaking table tests recently performed by Jabary and Madabhushi [6,23] on a multi-storey sway frame structure rested on a sand deposit.

As was seen from the system responses to a range of single frequency earthquakes in Figs. 9 and 10, both damper configurations considered in Fig. 12 show attenuation of the peak acceleration response in comparison to the response of the structure in the absence of an external damping mechanism. As is highlighted within the initial 15 s of the acceleration–time history record of Fig. 12, a de-tuned mass damper may amplify structural response. As is apparent from Fig. 11, the extent to which a mass damper is tuned away from the soil-structure system frequency could greatly influence structural response and potentially cause tremendous response amplification (of up to 22%). This stresses the need for precise tuning and the consideration of the soil-structure system's natural frequency as opposed to the fixed-base structural frequency in doing so.

5. Conclusions

The effects of various damper configurations on the response of a sway frame structure were experimentally investigated in a series of geotechnical centrifuge tests. These were conducted for a range of earthquake characteristics and soil profiles. Excitations consisted of single frequency tone burst earthquakes as well as frequency sweep earthquakes which cover a much wider frequency range. Tuning a mass damper to the fundamental frequency of a soil-structure system was found to effectively reduce peak structural responses for all soil stiffness values considered. The most significant findings from the series of centrifuge tests that were conducted are as follows, in the order of significance:

The centrifuge test results confirm that tuning of a TMD based on soil-structure properties is more beneficial than tuning based on fixed-base structural properties. For the cases investigated in this research, this improvement was from 19% to 39% peak acceleration attenuation. Tuning the TMD to soil-structure system properties halved peak storey displacement.

The frequency to which the TMD is tuned has a strong influence on the structural response both in terms of the temporal location of the peaks and the exposure time to large cycles of shaking.

Even for a de-tuned mass damper, reduction of peak displacement response was still found to be quite significant (25%) for a structure resting on a soft sand deposit. This contradicts findings from previous analytical studies which observed that tuning a TMD to the fixed-base frequency under soft soil conditions would be completely ineffective in reducing structural displacements.

Large amplifications in the structural response were observed due to resonance effects with the application of swept-sine wave input motions. The presence of a TMD in such situations led to a decrease in the structural response for both tuned and de-tuned cases. This demonstrates the practicality of TMDs, particularly in light of unexpected earthquake characteristics that structures may be subjected to in reality.

The extent to which a mass damper is tuned away from the soil-structure system frequency greatly influences structural response and could cause great response amplifications of up to 22% for the cases investigated in this research. This emphasises the need for accurate tuning and the consideration of the soil-structure system's natural frequency as opposed to the fixed-base frequency in doing so.

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