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Experimental Seismic Response Assessment of Tall Buildings With Large Mass Damping

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ABSTRACT

This paper describes an experimental investigation into the seismic performance of a novel integrated damping system. The proposed damping concept mobilises a portion of the building's own mass to generate damping from its differential motion relative to the lateral load-resisting system. In order to assess the viability and effectiveness of the system under seismic loading, experimental investigations are performed using a 1:300 dynamically scaled physical model of a 300 m tall building. The scaled model is developed using a proposed multivariable genetic algorithm optimisation workflow that enables precise design and fabrication while explicitly incorporating the damping system. Harmonic and seismic tests are then carried out on a number of damped and undamped model variations using several ground motion excitations and multiple intensity levels. The experimental results are compared with finite element simulations of both the full-scale prototype as well as a digital twin of the dynamically scaled model. The experimental results demonstrate the ability of the integrated large mass damping system to significantly reduce the structural response, with average peak reductions in accelerations and displacements of about 40% and minimal differential displacements between the lateral load-resisting system and the floors. Complementary numerical studies are additionally used to evaluate the influence of the mass ratio and other key damping parameters and to illustrate the feasibility of partial-height implementation to maximise efficiency and resilience while significantly reducing costs. The findings highlight the effectiveness and robustness of the damping approach for enhancing seismic resilience in tall buildings, with the potential to deliver substantial reductions in both construction cost and embodied carbon.

1 | Introduction

Tall building construction has experienced exponential growth in recent decades and is expected to continue increasing due to space efficiency and commercial benefits. The design of tall buildings is often governed by their dynamic response, and more specifically, by the need to control wind-induced accelerations within user comfort levels. Apart from the conventional

approach of controlling the dynamic response by increasing the mass or stiffness of a building, it is widely recognised that the provision of supplementary damping can lead to a significantly more cost- and carbon-efficient design [1–4]. Incorporating supplementary damping has also been shown to considerably enhance building seismic response. If a redundant damping system is installed, it is also possible to take advantage of the added damping to reduce the seismic strength demands

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Summary

- The study describes an experimental investigation into the seismic performance of a novel large mass integrated damping system.
- A scaled physical model of a 300 m tall building is developed using a proposed multi-variable genetic optimization workflow.
- Harmonic and seismic tests are carried out on damped and undamped physical model variations and compared with numerical simulations.
- The results demonstrate the effectiveness of the proposed damping arrangement in providing substantial reductions in the seismic accelerations and displacements.

and enhance resilience, reducing non-structural damage and downtime [5, 6].

Since the first elastomeric dampers were installed in the World Trade Centre in 1969 to control wind-induced vibrations [7], supplementary damping has been an active topic of research, with numerous systems being proposed for various applications [8]. Damping systems can be broadly categorised into passive, active and hybrid, depending on their level of interaction with the excitations [9]. Among these categories, passive systems are the most widely used, owing to their simplicity, reliability and well-established performance record. While significant advances have been made in active and hybrid systems, they remain less frequently used [10–12].

The most commonly implemented damping systems are arguably tuned mass dampers (TMDs). These are based on the vibration absorber system proposed by Ormondroyd and Den Hartog [13], which introduces a mass with a spring and a damper in parallel to reduce the dynamic response of a structure. The system reduces the response by generating an equal and opposite dynamic force and/or dissipating energy through the damper [14]. Their wide use is owed to their simple and verified behaviour, and their versatility to be added later in the design stage, once wind tunnel test results are available, without changing the dynamic behaviour of the structure. However, despite their extended use, TMDs have several drawbacks as follows: (i) they occupy a significant valuable space at the top of a building; (ii) they are very sensitive to detuning and can only mitigate the response for the specific frequency for which they are tuned, limiting their use for seismic applications; and (iii) they are non-redundant damping systems, and thus their generated damping cannot be used to reduce strength demands [15].

Distributed damping systems aim to maximise the effectiveness of added damping by spreading dampers throughout the structure, forming a redundant system that can reliably mitigate both wind and seismic demands. These systems typically use elastomeric, fluid viscous dampers (FVDs) or hysteretic dampers as bracing elements or coupling core walls. However, most distributed systems rely on overall building flexibility to dissipate energy, making them less effective for tall buildings with low inter-storey drifts. Notable examples include Torre Mayor in Mexico

(98 FVDs with 20 m mega-braces [16, 17]), 111 Huntington Avenue in Boston (FVDs with toggle braces [18, 19]) and 181 Fremont Street in San Francisco (mega-braces with BRBs and FVDs [8]). Other methods include the damped outrigger concept—placing FVDs at outrigger tips [20]—and viscoelastic coupling beams between cores [21]. While these approaches offer substantial design savings, they rely on softening the building by creating alternative load paths that direct forces into the dampers, unlike mass-damping systems that do not require reducing structural stiffness. This softening has a detrimental effect on the response to wind, thereby limiting their practical use at greater heights.

With the aim of overcoming most of the drawbacks of TMDs and other conventional damping systems, such as their lack of redundancy and narrow frequency range, and height limitation, respectively, a different approach was recently proposed by the authors, referred to as ‘Integrated Damping System’ (IDS) [22]. The system mobilises a large portion of the building’s own mass as a damper mass and is hence able to generate large levels of redundant damping. The ability of the system to provide relatively large levels of damping and response reduction has been demonstrated numerically for wind and seismic excitations [23, 24]. While the approach is applicable to all building heights, it is particularly well-suited for tall buildings due to the significant benefits that redundant supplementary damping can provide.

Although the dynamic performance of tall buildings is often dominated by wind-induced effects, driving the widespread use of wind tunnel testing, ensuring robust seismic performance is equally critical, particularly in seismically active regions where even small-to-moderate events can lead to significant financial losses. For a novel damping system like the IDS, which introduces non-classical damping through the interaction of large structural masses, comprehensive validation of its dynamic behaviour under both wind and seismic loading is essential. To this end, aeroelastic wind tunnel testing, explicitly incorporating the system and replicating the actual dynamic behaviour of the prototype building, was identified as a key priority. Unlike high-frequency pressure integration, or high-frequency force balance techniques, aeroelastic testing incorporates the prototype modal properties, mass characteristics and distribution, scaled geometry and damping, thus enabling an accurate prediction of the response of the system and its interaction with the wind flow [25–31]. For seismic performance assessment, shake table testing is a well-established method [32, 33]. Prior studies range from standard buildings and components [34–38] to tall buildings [39–43] and, more specifically, to the evaluation of various damping systems [44–48]. In this work, the same dynamically scaled physical model developed for aeroelastic wind tunnel testing was repurposed for seismic tests. This provided an ideal opportunity to investigate the IDS performance under both critical loading regimes and to directly compare experimental results with prior numerical predictions.

This paper describes an experimental investigation into the response of the IDS, implemented within an illustrative 300 m tall building and subjected to harmonic and dynamic seismic tests. To make optimal use of the dynamically matched physical model, testing is conducted at a scale which is also suitable for separate wind tunnel experiments, thus exploring the feasibility

of using such small-scale models for comprehensive testing. In subsequent sections, the IDS concept is first introduced, followed by a description of the design and fabrication of the dynamically scaled model. Results from harmonic tests are then presented and analysed for both the physical model and a finite element model (FEM) of the physical model. Finally, the response to seismic excitations is assessed, leading to a comparative evaluation of the key parameters influencing the system behaviour and key design considerations.

2 | Integrated Damping System

Passive mass damping systems, and particularly TMDs, have gained significant recognition due to their reliability and efficiency in enhancing building response for serviceability wind excitations. However, their effectiveness is predominantly governed by the ratio between the damper mass and the main structural mass, typically limited to 0.5%–1.0% due to practical constraints. The influence of varying the mass ratio, μ , of the damper mass, m_d , to the main structure mass, m , under a harmonic excitation, p , applied to the main mass, can be examined using the common simplification of a single-degree-of-freedom (SDOF) system with an attached TMD, as illustrated in Figure 1. Figure 1b shows that increasing the mass ratio directly raises the equivalent damping generated, representing the percentage of critical damping present in the system, using optimal damper and tuning frequency parameters according to Den Hartog's formulation [49]. Although Den Hartog's formulation is only applicable in closed-form solution for an undamped main structure, this is extended to this case considering the very small damping present in tall buildings and its negligible influence as verified for both harmonic and random excitations [49, 50]. Likewise, Figure 1c indicates that as the mass ratio increases, the differential displacement between the main mass and damper mass displacements, U and U_d , respectively, for a specific damping level, reduces.

Increasing the mass ratio also has significant benefits in enhancing detuning and overall system robustness. The potential to take advantage of this has been explored in various studies. For example, Matta and Stefano [51] proposed the use of a heavy roof garden as a 'roof-garden TMD', similarly to the concept proposed by Villaverde for a more general application [44]. Mahmoud and Chulawat [52] investigated a hybrid approach combining Floor

Isolation Systems (FISs) with TMDs, creating 'suspended slab systems' acting as TMDs, similar to Xiang and Nishitani's [53] 'in-floor TMDs' concept. Feng and Mita [54] developed a 'mega sub-configuration' system, which proposed using an interior separate part of the building and thus taking significant space, to vibrate either as a conventional TMD or in shear with diagonal FVDs as a separate structure [55, 56]. Moon [57] proposed the use of the façade as a TMD mass, though requiring relatively large differential displacements due to the low mass. Other proposals included partial or intermediate mass isolation [58, 59], which have been successfully implemented for low- and mid-rise buildings, but have limited application for tall buildings. While each approach has its own merits, its practical implementation is often constrained by space limitations, cost and challenges of mobilising sufficient mass.

2.1 | Underlying Concept

To address the limitations of traditional TMDs, their variations and alternative damping approaches, the IDS [22] was recently proposed by the authors. The concept is based on the premise that, as shown in Figure 1, if a sufficiently large mass is mobilised, large levels of damping can be generated with minimal differential displacements. Rather than adding any extra mass, a usable portion of the building is mobilised, and, by connecting it with springs in parallel with viscous dampers, it can resist the static wind loads while controlling accelerations within the damper mass. Due to the large mass mobilised, the IDS departs from the tuning requirement associated with TMDs to maximise damping for a reduced mass and sets the stiffness of the damper mass to ensure that the level of differential displacements between the main mass and damper mass remains within practical serviceability constraints.

Apart from the significant damping generated, another merit of the system is that it does not need to occupy a large dedicated valuable space at the top of the building, or local strengthening to support the extra mass. Moreover, due to the large mass ratio, the system provides enhanced robustness against shifts in structural or excitation frequencies, hence enhancing the building response over a wider frequency range. The combination of this with the redundant nature of the system, extending through several floors, allows the IDS to effectively mitigate wind and seismic strength demands, thus maximising the damping benefits to bring sub-

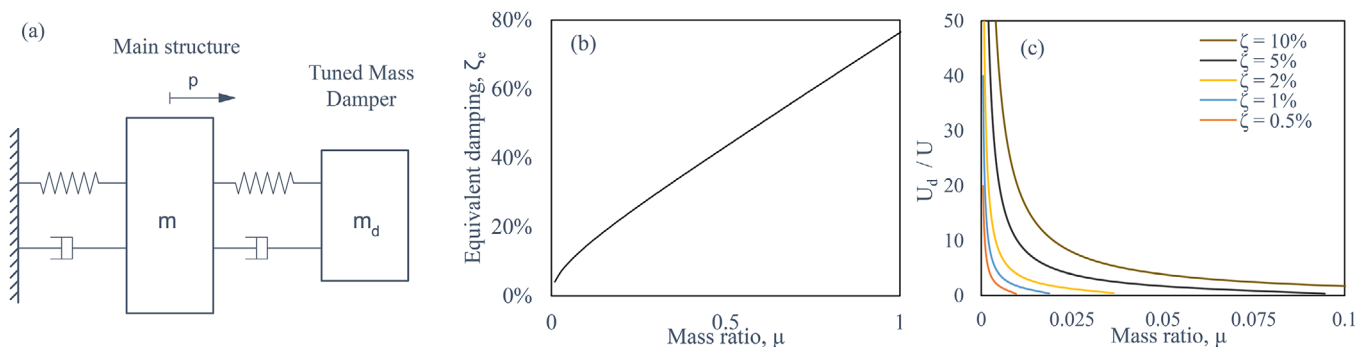


FIGURE 1 | (a) Simplified SDOF with TMD; (b) equivalent damping versus mass ratio under harmonic excitation; (c) ratio of relative displacement amplitude versus mass ratio for different target equivalent damping levels.

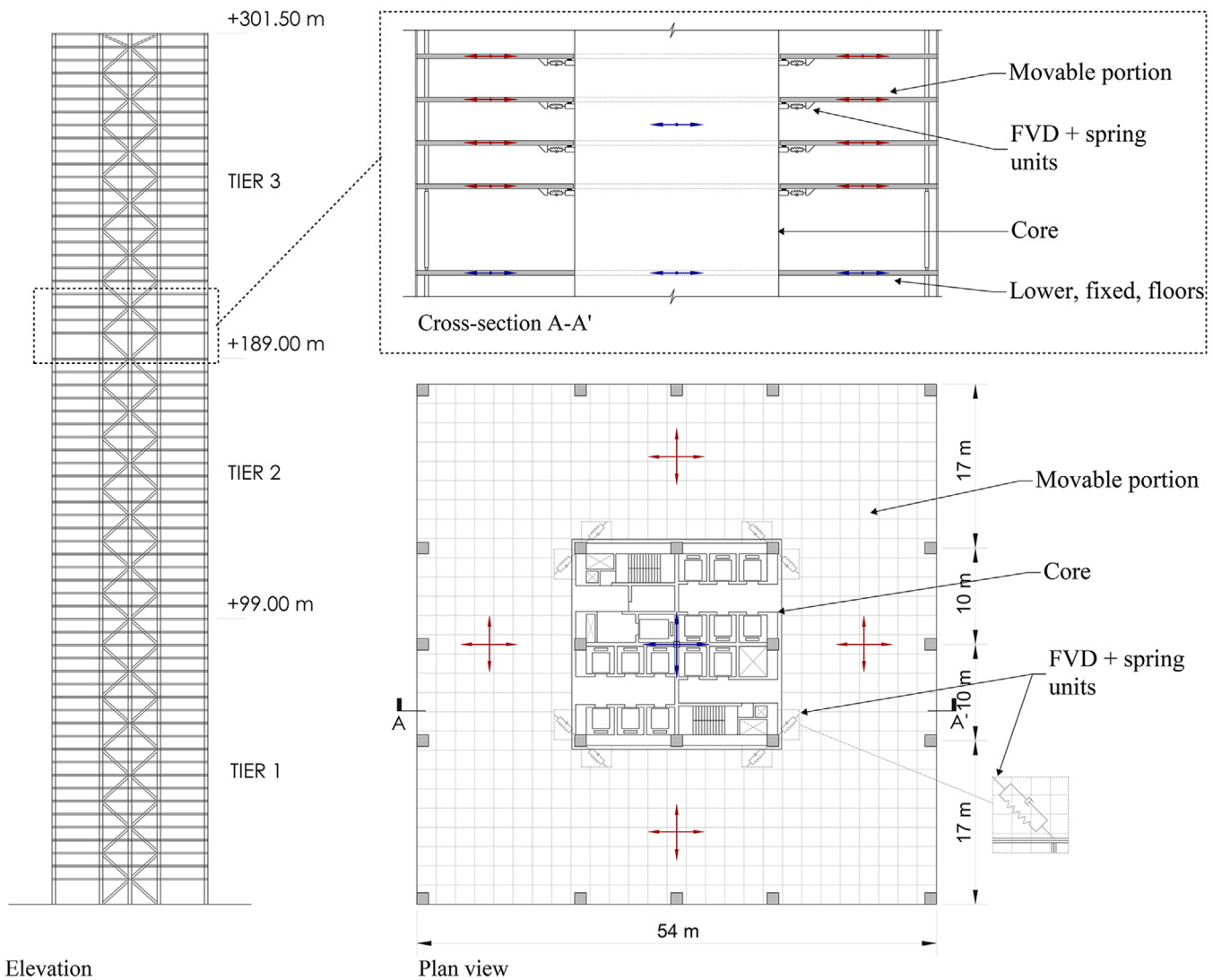


FIGURE 2 | Elevation, plan, and damper arrangement for the representative tall building.

stantial design savings. Numerical studies have demonstrated the potential for IDS to offer significant supplementary damping and response enhancement with minimal relative displacements under both wind and seismic excitations [24, 60].

2.2 | Representative Building

The performance of the IDS is investigated experimentally herein using an illustrative, 65-floor, 300 m tall building, previously assessed numerically [23]. The building, described in Figure 2, is designed as a composite structure, with a steel-braced central core as the lateral stability system, of 20 m × 20 m plan dimensions within a 54 m × 54 m square footprint. The columns adopt concrete-filled steel tubes (CFST) and are designed not to contribute to the lateral load-resisting system. The floors use composite steel/concrete decks with steel beams. The structure was designed following Eurocode provisions [60, 61], with a reference basic wind speed of 21.5 m/s. The dynamic response is characterised by translational first and second modes at a 9.9

s period and an uncoupled torsional third mode at 5 s period. Due to its flexibility, initial assessments indicated excessive wind-induced accelerations, making it an ideal candidate for implementing a supplementary damping system. Following this requirement, the IDS is detailed to engage the office floors in the top third of the building (Tier 3) and create a movement joint between the office floor and the core to allow the office floor, highlighted with red arrows in the figure, to move differentially from the core, shown in blue.

The performance of the system was investigated numerically using a SAP2000 [62] model and showed the potential to enhance the building efficiency and resilience. The baseline case, where the spring stiffness was calibrated to allow a maximum relative displacement of 150 mm under the 10 min mean 50-year wind load [63], led to a 6% increase in damping, significantly reducing the top-floor displacements, inter-story drifts and floor accelerations, under both wind and seismic loads. More detailed information on the building design and numerical results is available in [23, 60].

3 | Scaled Physical Model

Dynamically scaled models, for either wind tunnel or shake table testing, can closely replicate the dynamic response by matching the prototype structure properties following scaling and similarity rules [34]. To achieve this, the scaled model should match the prototype modal properties, mass distribution, geometry scale and damping. Due to the requirement to match all properties at a reduced scale, testing tends to be associated with a significant expense and complexity in terms of both design and fabrication. The aeroelastic models used to assess the wind response can be SDOF or multi-degree-of-freedom (MDOF). SDOF models are typically formed by a rigid model with a tuned rotating base that captures the first mode [31, 64, 65]. MDOF models, on the other hand, can capture the building response with much more accuracy, including the contribution of higher modes, but are significantly more complex [28–30, 66]. The physical model described below was developed for assessing wind as well as seismic response using a single model while explicitly integrating the IDS configuration. A relatively complex MDOF dynamically scaled model was therefore necessary, as simple SDOF or rigid approaches were deemed unsuitable.

3.1 | Model Formulation

To ensure dynamic matching between the physical model and the prototype, each dimensional parameter needs to be scaled following scaling and similarity rules. Once the geometric and velocity factors, λ_L and λ_V , respectively, are defined, keeping the air density, λ_ρ , as 1.0, the remaining parameters can be directly derived as shown in Table 1. Since the dynamically scaled model was intended for seismic as well as wind tunnel testing, these had to comply with additional requirements beyond a maximum size and weight to suit the shaking table limitations. The geometric scale was set at 1/300 to ensure that the model does not block more than 5% of the tunnel cross-section. The velocity scale was set to 1/4 to avoid a scaled model frequency above 10 Hz and

hence vibrating at very high frequencies that could compromise the model integrity.

To respect the modal properties, mass distribution and stiffness of the prototype, it is necessary to match the generalised properties described in Equations (1)–(5) for every mode of interest, n . The number of DOFs is reduced in the physical model, which is common practice for aeroelastic models to reflect the lower scale and meet fabrication constraints. In the equations, the subscript i corresponds to the prototype floor numbers, while j corresponds to the module number in the scaled model [28, 67].

$$\omega_{\text{model}}^2 = \omega_{\text{prototype}}^2 \quad (1)$$

$$\zeta_{\text{model}} = \zeta_{\text{prototype}} \quad (2)$$

$$m_j \varphi_{nj}^2 = \sum_{i=i_1} m_i \varphi_{ni}^2 \quad (3)$$

$$k_j \Delta \varphi_{nj} = \sum_{i=i_1} k_i \Delta \varphi_{ni} \quad (4)$$

$$I_j \varphi_{n\theta j}^2 = \sum_{i=i_1} I_i \varphi_{n\theta i}^2 \quad (5)$$

in which m , k , I and φ_n are the mass, stiffness, rotational mass and n -th modal shape coordinate, respectively, where $\Delta \varphi_{nj} = \varphi_{nj} - \varphi_{nj-1}$, and $\Delta \varphi_{nj} = \varphi_{nj} - \varphi_{nj-1}$ and the summations include the number of floors that are condensed in each module of the scaled model.

3.2 | Design Procedures

As the scaled physical model, illustrated schematically in Figure 3, had to incorporate a movable portion connected via miniature springs and dampers to the lateral stability system, the formulation of Equations (1)–(5) was more intricate than for conventional aeroelastic models. Given this complexity, and to ensure model accuracy while allowing design iterations and accounting for a realistic estimate of how the physical model behaves, a parametric FEM of the MDOF aeroelastic model was developed using Oasys GSA [68] as well as the new GSA-GH component [69] within the visual programming interface Grasshopper [70].

By setting the model parametrically, it was possible to assess the frequencies, nodal masses and mode shapes for each model variation directly within Grasshopper [70]. Using the genetic algorithm optimisation solver Galapagos [71], the model configuration was optimised in terms of element size and setting out, iterating through thousands of potential setups by linking the model results with those from Equations (1)–(5) for each iteration. The algorithm was set to prioritise matching of the first two translational modal frequencies, shapes and mass distributions, followed by secondary translational modes, while ensuring no torsional coupling in the first modes. Even though it was possible, through the parametric procedure, to account for the participation of every node, the model was split into six equivalent DOFs, or nominal nodes, to facilitate fabrication and preliminary design. The optimal vertical segmentation was determined using the same algorithm, ensuring an exact match for the first-mode mass distribution (Equation 3) and close agreement with the second

TABLE 1 | Scaled physical model similarity ratios.

Model parameter	Similarity parameter	Scale ratio
Length	λ_L	1/300
Velocity	λ_V	1/4
Time	$\lambda_T = \frac{\lambda_L}{\lambda_V}$	1/75
Acceleration	$\lambda_a = \frac{\lambda_V^2}{\lambda_L}$	18.75
Damping ratio	λ_ζ	1
Density	λ_ρ	1
Mass	$\lambda_M = \lambda_\rho \lambda_L^3$	$1/2.7 \times 10^7$
Frequency	$\lambda_f = \frac{\lambda_V}{\lambda_L}$	75
Force	$\lambda_F = \lambda_\rho (\lambda_L \lambda_V)^2$	$1/1.44 \times 10^6$
Bending moment	$\lambda_{BM} = \lambda_\rho \lambda_V^2 \lambda_L^3$	$1/4.32 \times 10^8$
Stiffness	$\lambda_K = \lambda_\rho \lambda_V^2 \lambda_L$	1/4800
Damper coefficient	$\lambda_C = \lambda_\rho \lambda_V^2 \lambda_L \lambda_T$	$1/3.6 \times 10^5$

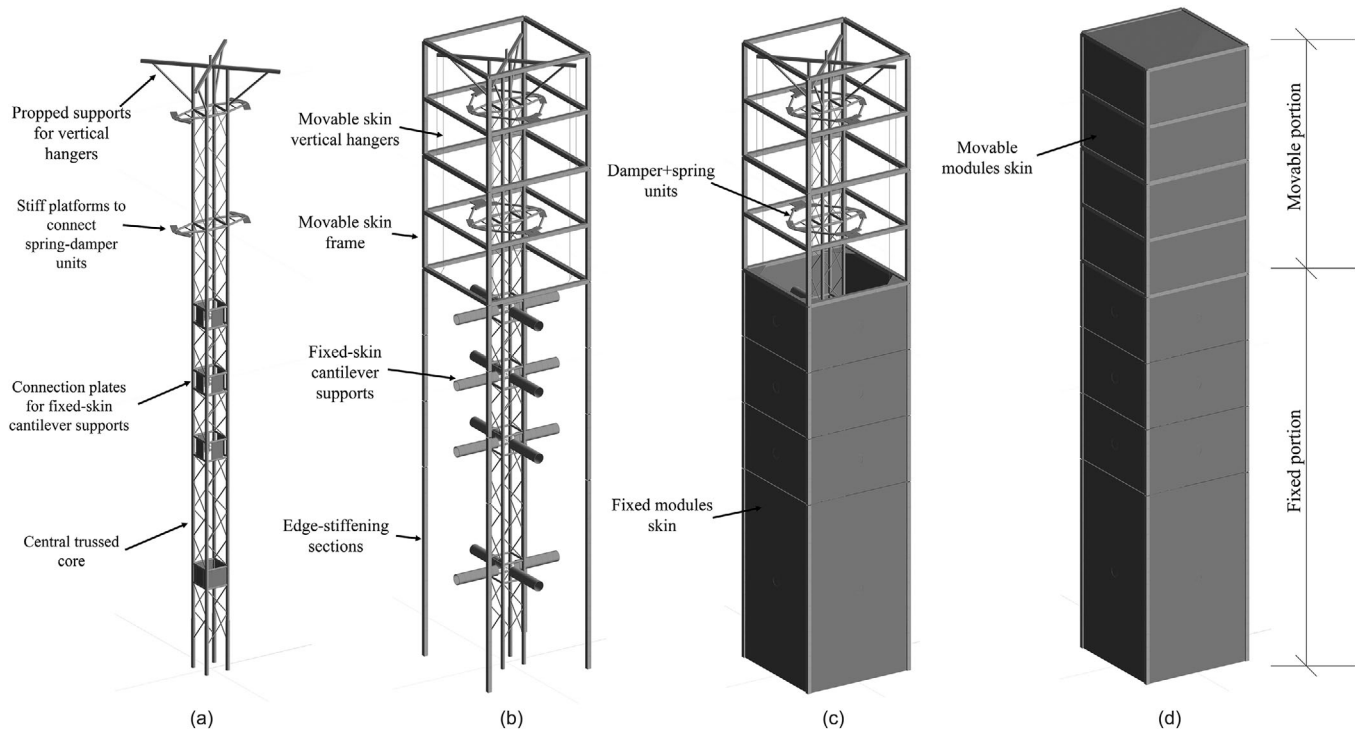


FIGURE 3 | Finite element representation of the scaled physical model at different stages: (a) central core, (b) core with outer frame, (c) core with lower skin and (d) complete model.

mode. The optimisation resulted in a model with six modules, where the lower four represent the fixed portion and the top two represent the movable section (Figure 3). The original Grasshopper script is shared as an accompanying online [Supporting File](#).

Following preliminary investigations, the scaled model was designed with a central braced core for lateral stability and a removable perimeter skin forming the building envelope. The top movable portion is suspended from a propped cantilever that extends from the core, with horizontal restraint provided by parallel miniature springs and dampers at the centre of the module. Positioned at a 45° angle, the dampers control translational movement in both directions while preventing torsional rotation. The skin in the lower four fixed modules is supported via outriggers that cantilever from the core on its four sides. Vertical gaps of 1.5 mm are included between the skin panels to avoid contact between the modules when vibrating at large amplitudes and to prevent the outriggers from contributing to the lateral stability of the building. Figure 3 illustrates the configuration of the model at different stages.

The structural frame was fabricated from soldered brass circular hollow sections of varying sizes along the height, tuned to match the optimisation results. The skin was made from Perspex due to its high stiffness and transparency, and was detailed as removable with countersunk bolts to allow model tuning and modification. Lead weights and 3D-printed accelerometer supports were used. Each module skin was independently attached and separated by a 1.5 mm gap to prevent unintended stiffness contributions. The model was designed to allow the removal and modification of the damper and spring units to test additional configurations. These included a case where a rigid bar is incorporated, recreating the

behaviour of a conventional rigid building, as well as three different spring stiffnesses of 0.78, 1.9 and 2.27 N/mm, to match target differential displacements of 410, 170 and 140 mm, respectively, under the EN 1991-1-4 [63] 50-year wind load applied as a static load, in a manner similar to that used during the calibration of the numerical analysis. An additional configuration (referred to as ‘topOnly’ later on) included fixing the lower block with a rigid bar and allowing only the top module to move, reducing the movable portion to only the top 1/sixth with a 1.9 N/mm spring stiffness. The miniature dampers were carefully chosen to minimise friction and static stiffness, and calibrated using an in-house setup to achieve the scaled target damping coefficient of 65 Ns/m.

3.3 | Fabrication and Calibration

The complexity and sensitivity of the scaled physical model required an exceptionally well-controlled fabrication process, exceeding the already high standard adopted for aeroelastic or shake table models. Key structural elements were tested to verify their stiffness and elastic modulus, and the weight of all additional components was measured to precisely account for their contribution in the model generation parametric procedure. A view of the completed model is shown in Figure 4. The model behaviour was assessed by performing free vibration tests in both principal directions at key assembly stages and compared against a FEM of the physical model at the same stage to detect any deviation. The modal parameters at the complete stage are given in Table 2.

As indicated, the aeroelastic model closely replicates the target scaled properties of the prototype. The first natural frequencies

TABLE 2 | Modal parameters of the prototype building and scaled physical model.

Prototype building 3D MDOF FEM model				Scaled physical model target properties		Scaled physical model measured properties	
Mode	Type	Period (s)	Frequency (Hz)	Period (s)	Frequency (Hz)	Period (s)	Frequency (Hz)
1	Trans. X	9.9	0.10	0.13	7.5	0.13	7.58
2	Trans. Y	9.9	0.10	0.13	7.5	0.13	7.70
3	Torsion	5.0	0.20	0.067	14.9	0.10	10.1
4	Trans. X	2.4	0.42	0.032	31.2	0.038	26.0
5	Trans. Y	2.4	0.42	0.032	31.2	0.038	26.2

**FIGURE 4** | Scaled physical model with rigid bars joining the skin and the core.

were 7.58 Hz in the X direction and 7.70 Hz in the Y direction, differing by only 2% from the 7.5 Hz target. The main deviation was observed in the torsional mode, attributed to the top two movable blocks, which concentrate much of their mass around the perimeter (in the skin and its framing) due to fabrication constraints when incorporating the movable mass, resulting in higher rotational inertia than the prototype.

Figure 5 illustrates the free vibration displacement time history and first mode shape in the X and Y directions for the rigid case at the final completed stage. The model intrinsic damping, extracted using the logarithmic decrement technique [72], was 0.4% in the X direction and 0.6% in the Y direction. Although this was lower than the target of 0.8% corresponding to the prototype building (as mentioned in [23]), by performing sensitivity studies at different vibration amplitudes, it was possible to observe that the actual values range between 0.6% and 1.2% and thus considered to be acceptable.

3.4 | Instrumentation Details

The response of the dynamically scaled model was measured using nine miniature dual-axis accelerometers with amplified analogue output and a 2000 Hz sampling frequency. The sensors

were strategically positioned to capture X and Y displacements at four key levels (L2, L4, L5 and L6), monitor the torsional response of both the core and the skin, assess overturning effects and track differential displacements between the core and the skin. Their specific location and axes are shown in Figure 6.

4 | Harmonic Tests

4.1 | Methodology and Finite Element Model of the Physical Model

The harmonic and dynamic seismic tests were carried out at the Structures Laboratories of Imperial College London by connecting the model to an Instron hydraulic actuator. The precision of the dynamic actuator was first examined under harmonic and seismic pilot excitations to assess its operational amplitude and frequency range. This was undertaken by comparing the input excitation with the actuator's own output and an external laser tracker, of type optoNCDT 1800–ILD 1810-50, with a large stand-off distance, a resolution of 5 μm and a sampling rate of 2.5 kHz. The laser sensor was retained for all harmonic and seismic tests for additional verification. The actuator was able to accurately reproduce harmonic motions up to 25 Hz with a minimum amplitude of 0.05 mm. Despite the high frequencies present in the seismic excitations, due to the 1:75 frequency scaling factor (Table 1), the accuracy of the actuator output was verified for the frequencies of interest for seven selected strong motion excitations. The scaled physical model was attached to the head of the actuator via a stiffened horizontal platform, as shown in Figure 7. It should be noted that, for ease of reference, all results are given in full-scale prototype units.

The tower model was first subjected to a series of harmonic tests at varying frequencies to obtain further insights into the system behaviour. Harmonic tests are particularly useful in providing a simple means to characterise the response and allow a direct comparison with a FEM of the physical model. To assess the response of the system at resonance, the test included a frequency sweep, typically ranging from 6.5 to 8.3 Hz, with varying frequency steps to capture the fundamental natural mode. These were performed with a base amplitude of 0.11 mm, which was determined from the FEM of the physical model to be the largest amplitude the model could resist without risking any potential damage when vibrating for a sustained period of time in the rigid case under resonance conditions. Each test was recorded for 30 s, equivalent

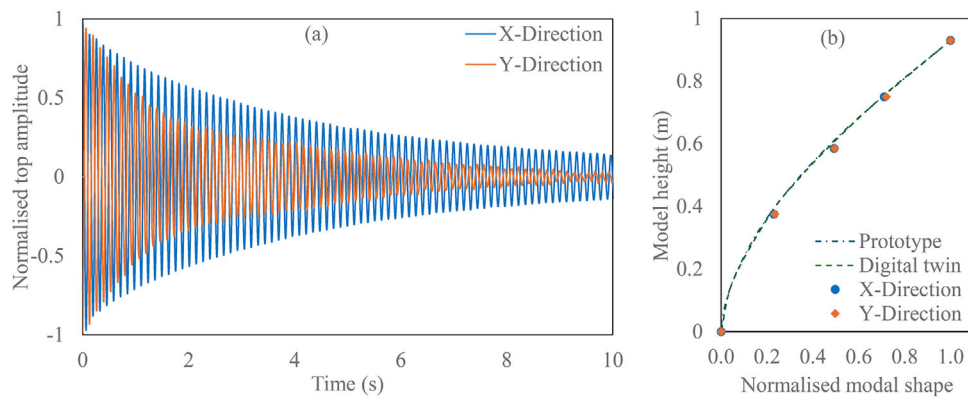


FIGURE 5 | Free vibration decay results: (a) displacement time history and (b) extracted mode shape.

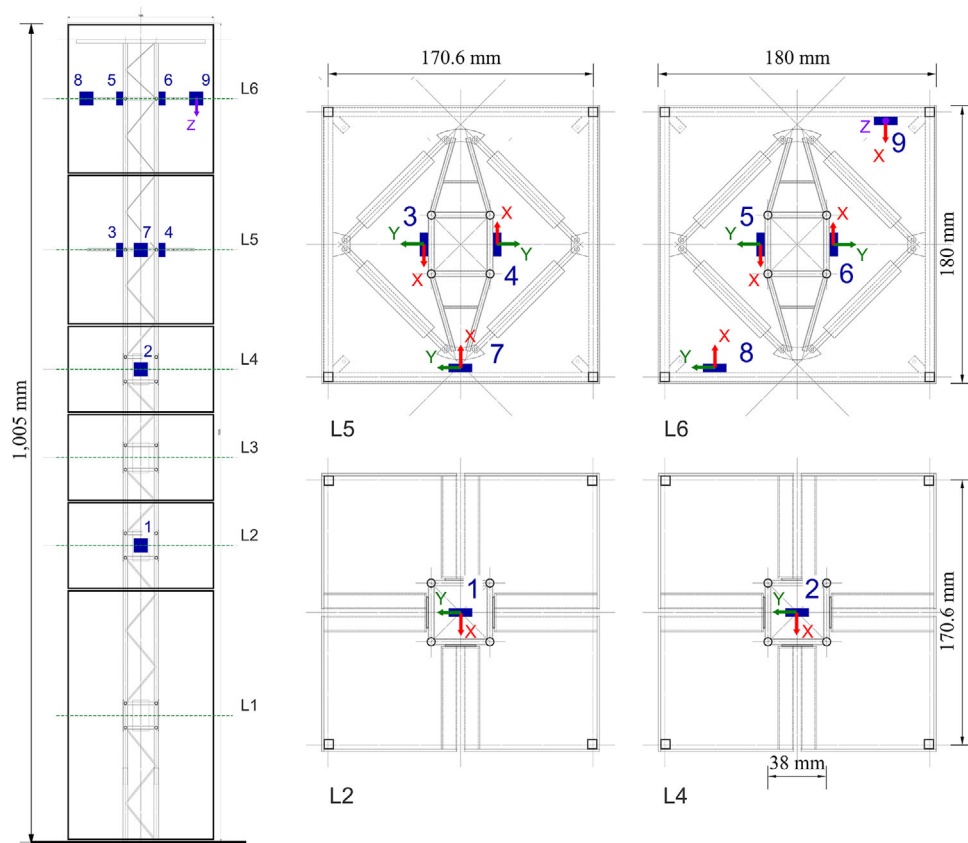


FIGURE 6 | Location and axis orientation of accelerometers in the scaled physical model.

to 37.5 min at full scale, to allow the test to reach steady state conditions.

4.2 | Forced Vibrations

The response of the system was first examined for the rigid configuration, given that it results in the largest amplitude of vibration due to the reduced damping at resonance conditions, and as a way of capping the vibration amplitude and verifying the results of the FEM of the physical model. By performing an initial sweep excitation, it was possible to verify the first natural frequency of the physical model as 7.58 Hz. This was only 0.6%

different from the target frequency of 7.5 Hz (Table 2), which validated the suitability of the proposed parametric physical model generation procedure. By comparing the displacement time-history response of the physical model and its FEM, it is observed that the model experiences 13% larger displacements (Figure 8). This difference in displacement, equivalent to 1.33 mm of movement at the top of the scaled model, results from possible slight inherent flexibilities in the physical model connections when compared to those of its FEM, as well as very small rotation, equivalent to about 0.05° at the mounting base platform.

Once the behaviour of the rigid scaled physical model was verified against its corresponding FEM, it was first interesting to observe

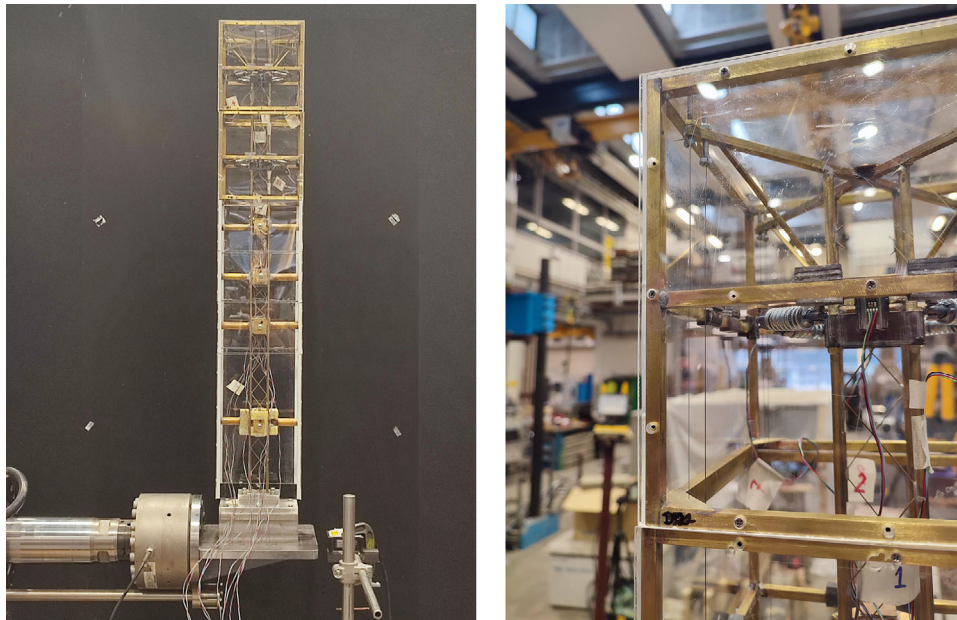


FIGURE 7 | View of scaled model mounted on the actuator, and close-up of part of the model.

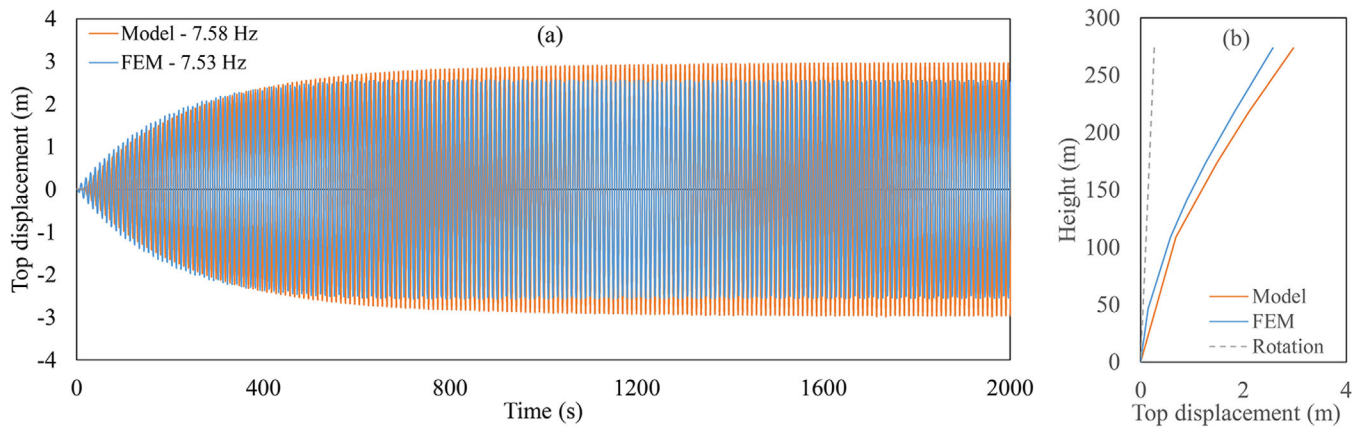


FIGURE 8 | Comparison between physical model and its corresponding FEM: (a) displacement time history at resonance and (b) maximum displacements along height.

how the core and skin behave under harmonic excitations. As shown in Figure 9a, for the four stiffness configurations at resonance and for an initial test amplitude of 0.11 mm, the skin, or movable mass, does not behave like the mass of a perfectly tuned TMD, but only has a small offset with respect to the core. The skin follows the same frequency as the core, yet with a slightly higher vibration amplitude. The configurations with a softer spring stiffness show a slightly longer phase offset and, as described later, result in larger damping and hence reduced vibration amplitude. When vibrating at 10 Hz (Figure 9b), the core incorporates the higher frequency in its vibration, and the movable mass has a significantly smoother response, helping to mitigate the overall building vibration. Notably, despite the initial excitation amplitudes being calibrated in the FEM of the physical model to give comparable top displacements for the rigid configuration at both frequencies, the four damped configurations exhibit significantly lower vibration levels under resonance than at 10 Hz. This suggests that, for the same structural response target, the

damped configurations are more effective in suppressing the vibrations, even though the excitation amplitude at resonance (0.11 mm) is substantially lower than that at 10 Hz (1.316 mm, at model scale). This contrast indicates a potentially higher effectiveness in attenuating vibrations near resonance compared to higher-amplitude, non-resonant excitations.

4.3 | Sweep Excitations

The system can be further characterised by investigating the response under a harmonic sweep around the natural frequency. This was performed for the five configurations, with an amplitude of 0.11 mm, and a wide frequency range on both sides of the natural frequency to enable the use of the half-power bandwidth method [72] to estimate the equivalent damping in the system. The peak vibration amplitude at steady state for each frequency and configuration, and for Levels 5 and 6 (Figure 6), is shown

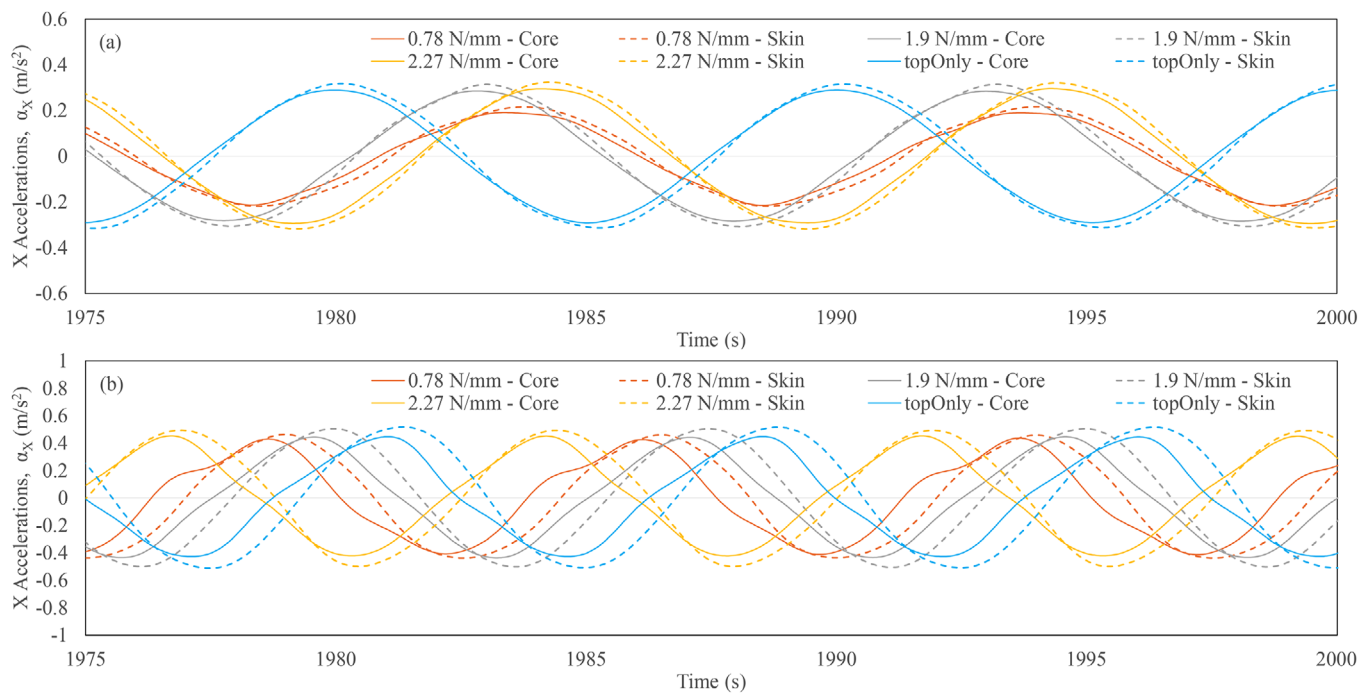


FIGURE 9 | Top core acceleration response history under harmonic excitation at (a) resonance and (b) 10 Hz.

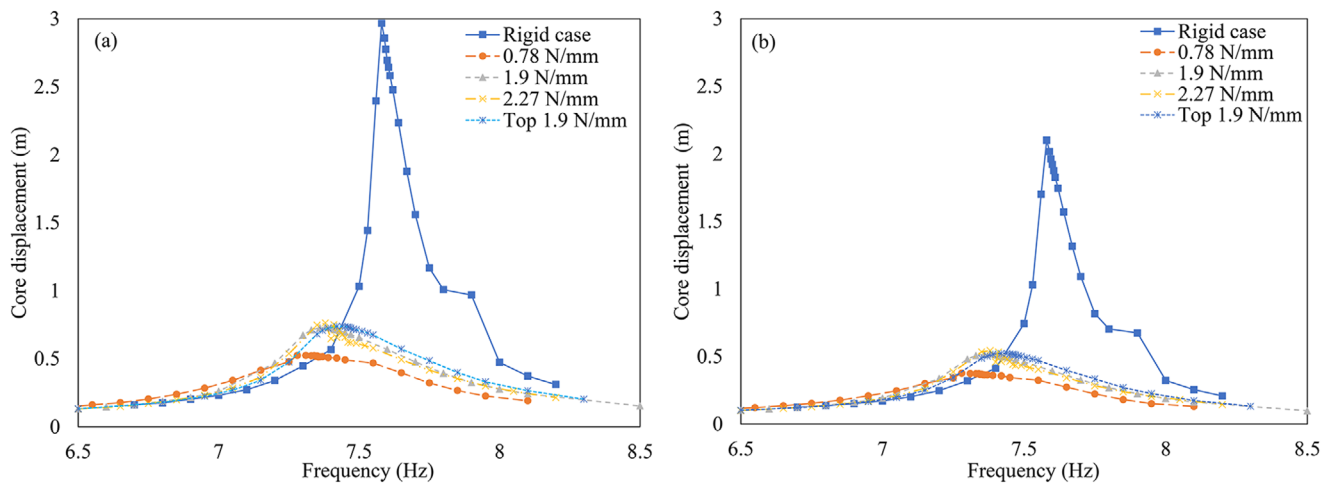


FIGURE 10 | Core peak displacement of physical model at steady state for varying frequency at (a) Level 6 and (b) Level 5.

in Figure 10. The results indicate a drastic reduction in peak response in all the damped cases. While the rigid configuration reaches a prototype peak top displacement of 3 m, equivalent to $H/100$, the damped cases range from 0.5 to 0.75 m, equivalent to $H/400$ to $H/700$, verifying the considerable potential of the system to substantially reduce the response amplitude.

From the results, it is possible to determine the equivalent damping of the system. For the rigid case, this gives an equivalent damping of 0.7% of the critical, slightly below the target of 0.8%. For the damped configurations: 4.1% damping is obtained for the 0.78 N/mm spring stiffness case, 2.9% is attained for the 1.9 N/mm case, 2.3% is found for the 2.27 N/mm case and 2.9% for the top-only case with the 1.9 N/mm springs. Additional insights can also be obtained by examining the differential displacements between

the core and the skin, as depicted in Figure 11 for Levels 5 and 6. Although the nominally rigid case shows up to 85 mm differential displacements, it is worth noting that this corresponds to movements of less than 0.3 mm at model scale. These result from infinitesimally small flexibilities in the physical model, coupled with the long load path between the core and skin accelerometers, depicted in Figure 6, combined with the sharp vibrations observed for the rigid case under resonance. As expected, the damped configurations show increasing differential displacements with increased vibration amplitude. The four damped cases exhibit very similar levels of differential displacements, which explains the similar equivalent damping values obtained. The top-only configuration shows negligible movement at Level 5 and the largest movement at Level 6, which vibrates at a similar amplitude as in other cases, with only the top module to counteract the

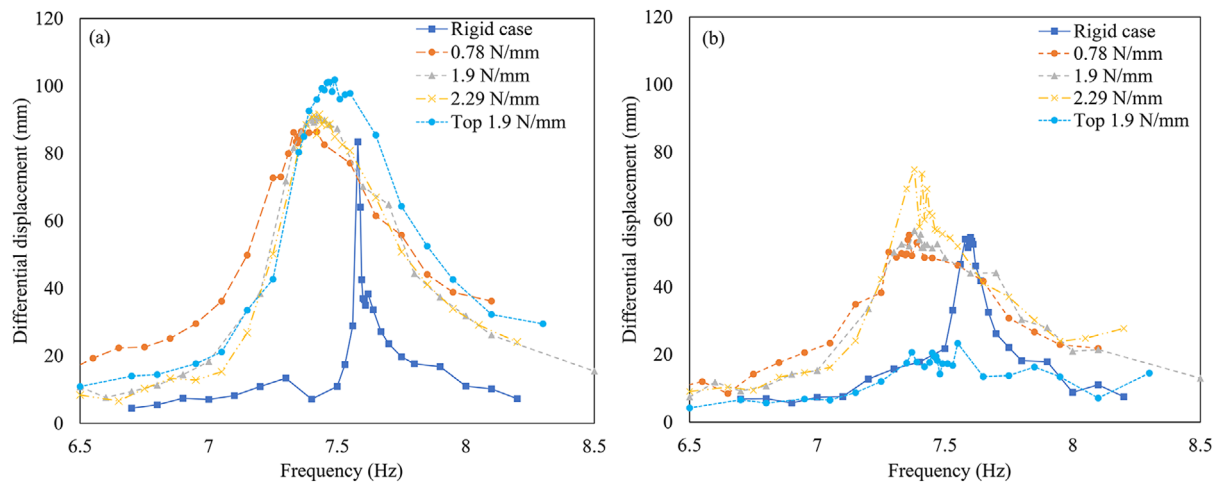


FIGURE 11 | Differential displacement between the core and skin of the physical model at steady state at (a) Level 6 and (b) Level 5.

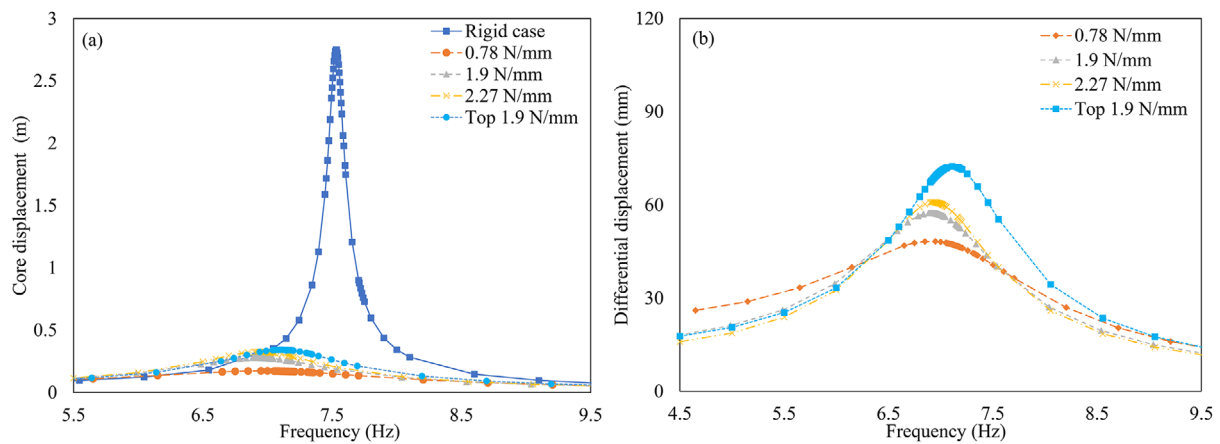


FIGURE 12 | Level 6 response of FEM of physical model at steady state in terms of (a) core peak displacement and (b) differential displacement between skin and core.

response. Even though the softer 0.78 N/mm spring shows smaller differential displacements, it is worth noting that these are also a function of the overall vibration amplitude. For matching vibration amplitudes, the softer horizontal stiffness would naturally result in larger differential displacements.

Overall, the calculated equivalent damping values are lower than those obtained from the prototype building, estimated as 6% for an equivalent baseline spring of 1.9 N/mm [23], and those from the FEM of the physical model due to stiction in the dampers and small frictions between components. As shown in Figure 12a, when subjected to the same harmonic sweep, the FEM of the physical model displays a smoother response and a larger difference for each spring stiffness. The differential displacements between the core and the skin show a similar behaviour to the experimental tests (Figure 11a), with larger values for the larger vibration amplitudes, yet with a similar order of magnitude and smoother behaviour (Figure 12b). From the values obtained, it is evident that, for an excitation that imposes nearly H/100 displacement in a conventionally rigid building, the IDS is able to reduce the top displacement to H/850 with only 75 mm differential displacement between the core and the skin.

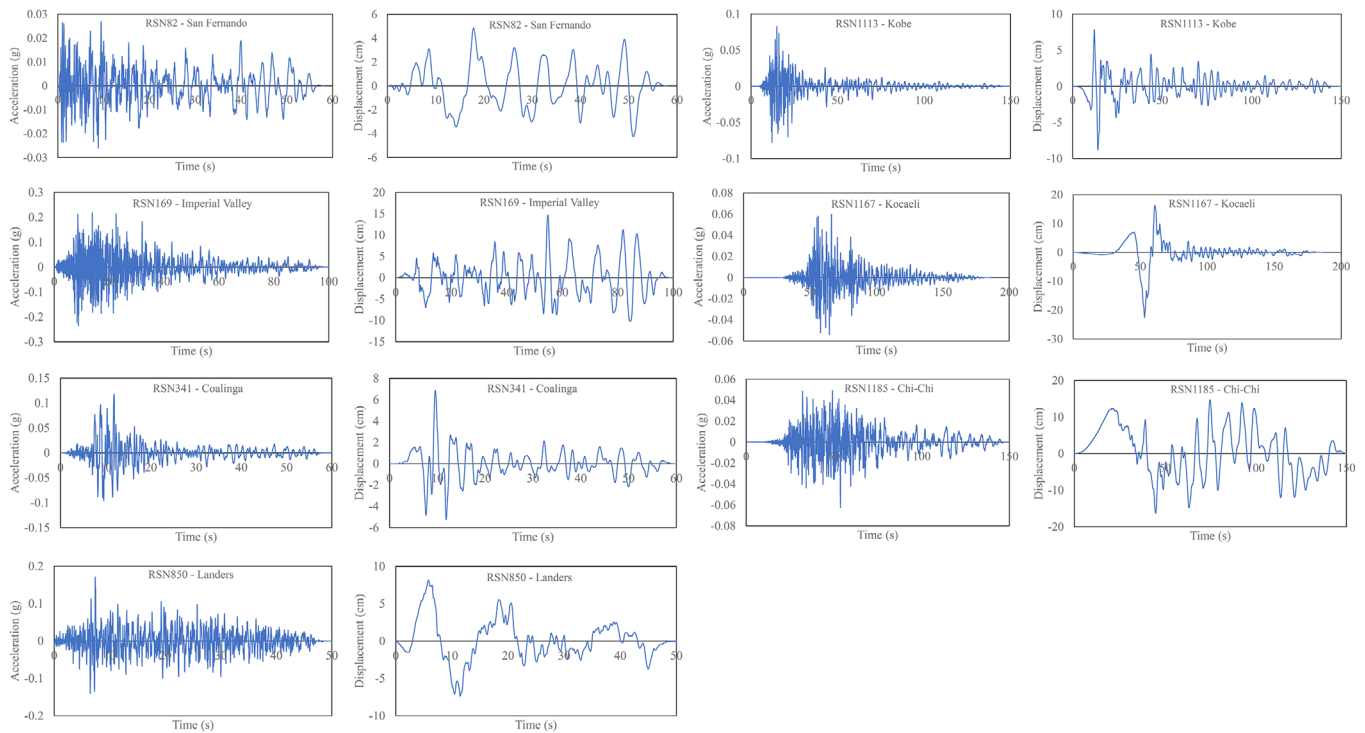
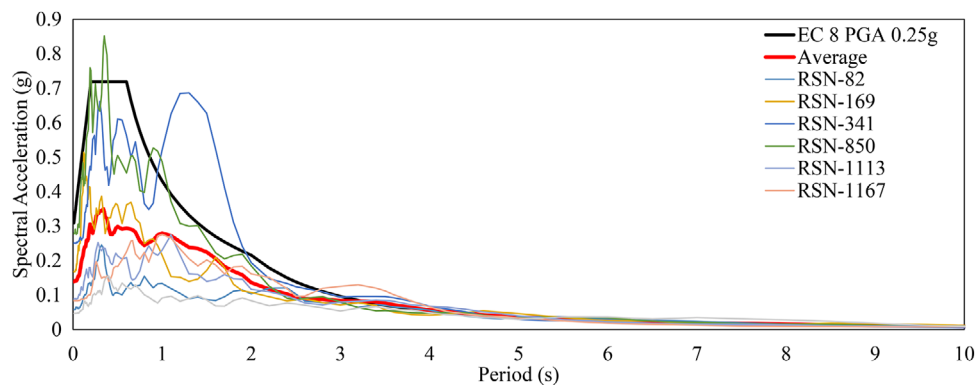
5 | Seismic Tests

5.1 | Loading Procedures

To examine the seismic response of the system, the scaled physical model was evaluated under seven natural ground motion records selected from the Pacific Earthquake Engineering Research–Next Generation Attenuation (PEER NGA) database [73] and applied at the base. These were chosen to have different characteristics and to closely match the EN 1998-1 Type 1 Soil C target spectrum [74], minimising the mean square error over a period range of 2–10 s, that is, $0.2T_1$ to T_1 , when scaled to match the EC8 spectrum with a peak ground acceleration (PGA) of 0.25 g and earthquake magnitudes, M , ranging from 6 to 8. To investigate the system response under different amplitude levels without notably altering the time-history characteristics, the records were applied at three scaled intensity levels: low, moderate and high. The low intensity corresponded to a direct EC8 Type 1 spectrum with a PGA of 0.25 g. The moderate and high intensities were obtained by scaling the records by factors of 3 and 6, matching EC8 Type 1 spectra with PGA of 0.65 g and 1.3 g, respectively. Table 3 gives the characteristics of the records, Figure 13 shows their acceleration and displacement time histories and Figure 14 illustrates the

TABLE 3 | Characteristics of selected ground motion records.

Record sequence number (RSN)	Earthquake name	Year	Magnitude, M	Distance (km)	V_{s30} (m/s)	PGA (g)
82	San Fernando	1971	6.61	68	248	0.037
169	Imperial Valley-06	1979	6.53	22	242	0.428
341	Coalinga-01	1983	6.36	38	294	0.180
850	Landers	1992	7.28	21	359	0.234
1113	Kobe	1995	6.9	21	256	0.107
1167	Kocaeli	1999	7.51	145	399	0.078
1185	Chi-Chi	1999	7.62	59	198	0.082

**FIGURE 13** | Unscaled acceleration and displacement time histories of the seven selected records.**FIGURE 14** | Scaled response spectra of the seven selected strong motion records for the low intensity level.

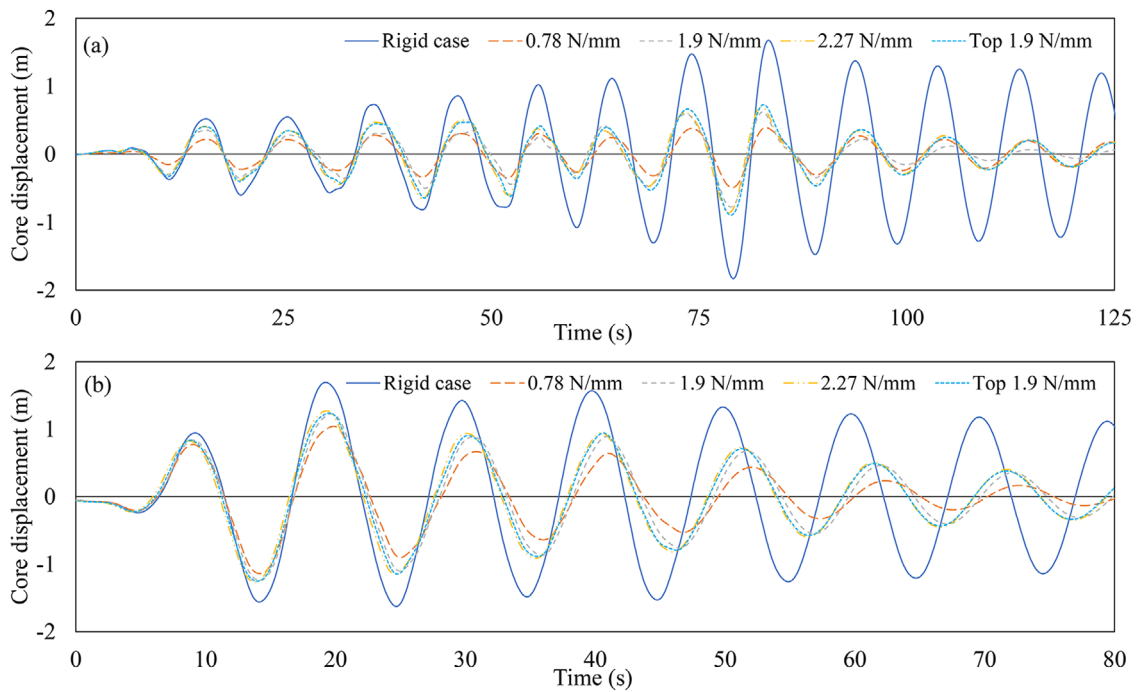


FIGURE 15 | Top core displacement time history of the five configurations under (a) moderate intensity RSN-169 Imperial Valley record and (b) high-intensity RSN-850 Landers record.

low-intensity level scaled response spectra compared to the EC8 Type 1 spectrum.

5.2 | Response Characteristics

This section discusses the results obtained by subjecting the scaled physical model to the three sets of seven ground motion records. The overall system performance is first evaluated by examining the time-history response of the five configurations to two representative ground motions: the moderate intensity RSN-169 Imperial Valley and the high-intensity RSN-850 Landers records (Figure 15). As observed, the rigid configuration exhibits significantly larger prototype displacements, reaching up to 1.829 m (H/164) and 1.695 m (H/177), respectively. These values are approximately 136% and 37% greater than those recorded for the baseline damped configuration with a 1.9 N/mm spring stiffness, which produces displacements of 0.775 m (H/386) and 1.238 m (H/242), during the moderate intensity RSN-169 Imperial Valley and the high-intensity RSN-850 Landers records, respectively. All damped configurations result in substantial reductions in response compared to the rigid case. As expected, the softer 0.78 N/mm spring stiffness achieves the greatest reduction in peak displacement, with values reduced to 36% and 15% of those of the baseline damped case, for a 1.9 N/mm spring stiffness, while the top-only configuration, where only the top module is permitted to move differentially, exhibits the least reductions which are only 3% and 1% above the baseline damped case displacements. For cases where the vibration amplitude increases gradually at resonance, the damper mass is mobilised and vibrates slightly out-of-phase, providing a significant response mitigation, as indicated by the peak near 80 s. However, for cases where the peak building response concentrates in the first oscillation cycles, such as for pulse excitations, the system is still effective in mitigating

the response, but, as expected, offers a smaller reduction. The fundamental natural frequency is slightly extended, consistent with the overall softening of the structure resulting from the decoupled mass.

The average maximum displacement across the seven ground motion records is also examined along the building height for each of the five configurations and three intensity levels, as shown in Figure 16. The plot also includes the response of the physical model FEM rigid and 1.9 N/mm cases under the same excitations. Compared to the rigid case, all damped configurations show a clear reduction in displacement throughout the structure, with the most pronounced effect occurring at the top. This trend is attributed to the greater effectiveness of the IDS in mitigating first-mode vibrations. While the softer 0.78 N/mm spring configuration shows a more substantial response reduction at lower amplitudes, the differences between each case diminish as the amplitude increases, indicating a slight amplitude dependency. Furthermore, by comparing the physical test results to those of its equivalent FEM, it is possible to verify the good match between both models, with similar amplitude levels, response reduction and deformed shape.

The maximum top displacement reduction ratios of the four damped configurations for each of the seven selected ground motion records and three amplitude levels compared to the rigid case are shown in Table 4. The results show that all four damped configurations achieve significant reductions in top displacement across all records and intensities. On average, the top displacements are reduced by up to 39% for the 0.78 N/mm case, 33% for the 1.9 and 2.27 N/mm cases, and up to 31% for the top-only configuration. While the numerical simulations do not exhibit amplitude dependency in the reduction ratios or overall response, the tests suggest an increasing trend in reduction ratios

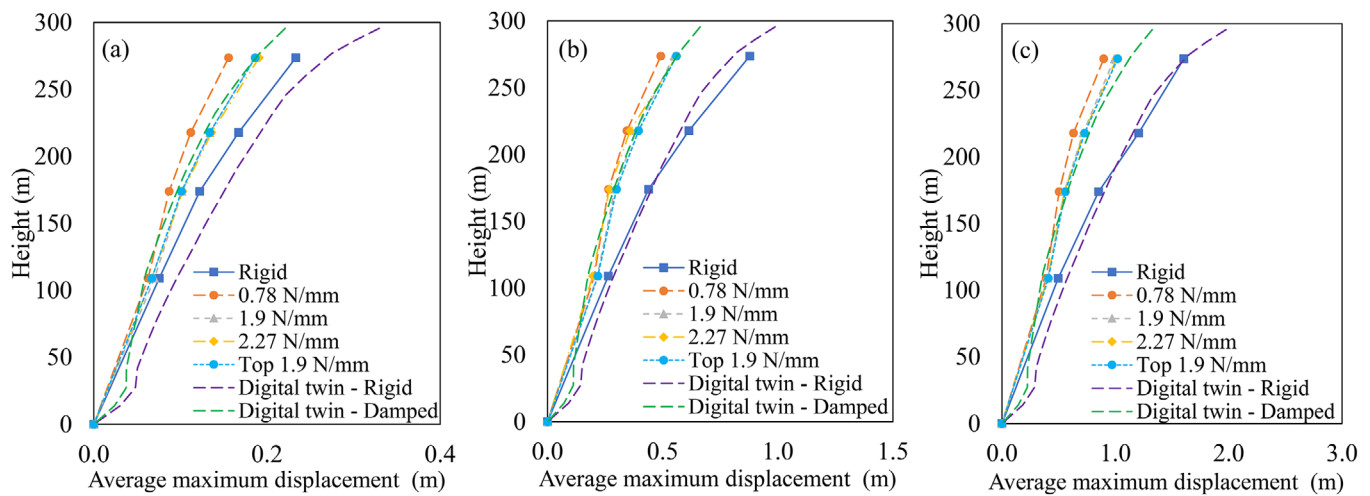


FIGURE 16 | Average maximum displacement over the seven records for the three considered intensities: (a) low, (b) moderate and (c) high.

TABLE 4 | Top displacement reduction for damped configurations versus the rigid case.

Record	0.78 N/mm			1.9 N/mm			2.27 N/mm			Top 1.9 N/mm		
	Low	Mod	High	Low	Mod	High	Low	Mod	High	Low	Mod	High
RSN82	32%	35%	41%	14%	21%	27%	15%	20%	28%	14%	21%	27%
RSN169	56%	58%	57%	35%	53%	53%	33%	52%	50%	34%	51%	51%
RSN341	25%	41%	34%	10%	36%	28%	-6%	17%	36%	22%	29%	23%
RSN850	27%	31%	33%	14%	22%	27%	11%	19%	25%	15%	23%	26%
RSN1113	30%	44%	48%	20%	32%	41%	22%	31%	37%	18%	30%	37%
RSN1167	3%	6%	11%	4%	5%	8%	1%	3%	5%	4%	4%	7%
RSN1185	45%	53%	53%	31%	48%	48%	29%	48%	48%	31%	47%	46%
Average	31%	38%	39%	18%	31%	33%	15%	27%	33%	20%	29%	31%

with amplitude, as shown earlier in Figure 16. This effect is particularly evident between the low and moderate intensity levels, and less so between moderate and high levels, likely due to the initial slight friction being overcome and the damped mass becoming more effectively engaged in mitigating the response. Although the friction forces seem to be overcome for larger amplitudes, the results highlight the importance of minimising friction forces in practice to maximise the system benefits in terms of added resilience for frequent low-magnitude events.

The tests also show that the differential displacements between the core and the skin, or movable mass, remain mostly below those initially estimated with the static wind load, except for the high-intensity case. The differential displacement at Level 6, which exhibits the largest values, is presented in Figure 17 for each record, configuration and intensity level.

The average differential displacements across all records and intensities are as follows: (i) for 0.78 N/mm spring stiffness (Table 5): 29, 132 and 241 mm; (ii) for 1.9 N/mm: 21, 134 and 276 mm; (iii) for 2.27 N/mm: 16, 111 and 198 mm (iv) and for the top-only case: 30, 164 and 317 mm. While the dampers are considered linear for simplicity, it is worth noting that

for cases of very high seismicity, and where large differential displacements occur, non-linear damper exponents or bi-linear damper coefficients could be easily incorporated to ensure that the differential displacements remain within specific practical constraints [23].

The favourable performance of the IDS observed through the scaled physical model tests aligns with previous findings from numerical assessments [22, 23], demonstrating the system's effectiveness and offering additional valuable insights into its behaviour. As summarised in Table 5, the implementation of the IDS can lead to substantial reductions in seismic demands, with the potential to deliver significant cost savings and enhanced resilience in tall building design.

6 | Comparative Assessments

To complement the harmonic and seismic tests, the validated digital-twin FEM is employed to investigate the response of the system to variations in key parameters, namely the mobilised mass, damper coefficient and spring stiffness, as shown in Figure 18. The response is evaluated in terms of the level

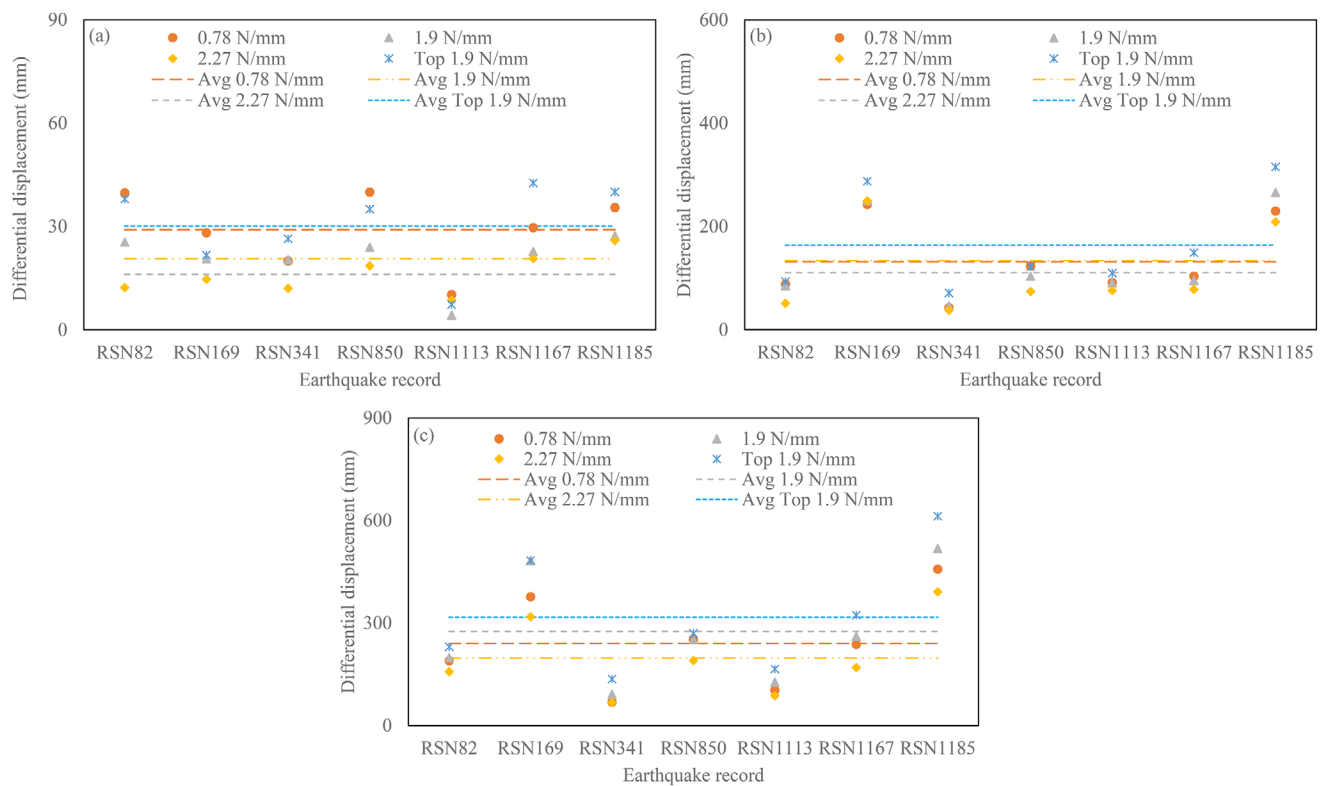


FIGURE 17 | Differential displacement between the core and the skin at Level 6 for the selected earthquake records and the three intensities: (a) low, (b) moderate and (c) high.

TABLE 5 | Average of the maximum accelerations, displacements and differential displacements over the seven selected records for each of the five configurations and three intensity levels.

Case	Low intensity			Moderate intensity			High intensity		
	Disp. (m)	Acc. (m/s ²)	Diff. disp. (mm)	Disp. (m)	Acc. (m/s ²)	Diff. disp. (mm)	Disp. (m)	Acc. (m/s ²)	Diff. disp. (mm)
Rigid	0.233	0.1502	—	0.878	0.537	—	1.604	0.928	—
0.78 N/mm	0.156	0.1047	29	0.492	0.357	132	0.897	0.687	241
1.9 N/mm	0.186	0.1118	21	0.549	0.374	134	0.990	0.697	276
2.27 N/mm	0.191	0.1069	16	0.505	0.328	111	1.010	0.680	198
Top only	0.186	0.1282	30	0.559	0.433	164	1.011	0.792	317

of supplementary damping generated by the system, which is quantified using the half-power bandwidth method.

6.1 | Influence of Damper Mass

The ratio of the damper mass to the main structure mass is considered the parameter that most strongly influences the performance of both TMD and IDS. Unlike for the IDS, this is limited to conventional TMDs due to practical constraints. It is, therefore, useful to assess the effect of mobilising a larger mass. Figure 18a illustrates the equivalent damping levels for increasing mobilised mass ratios across the four damped configurations, with the damper coefficient held constant as in the calibrated model.

As expected, all configurations exhibit an increase in damping with higher mass ratios. This is, however, more evident for the 0.78 N/mm spring stiffness, and less so for the 1.9 and 2.27 N/mm cases, which peak at a mobilised mass ratio of 180%, achieving 10.4% and 8.3% equivalent damping, respectively (Figure 18a). The optimum response of the system is achieved when the damper mass is tuned to the frequency of the building. Hence, there would be no further benefit in terms of larger damping in increasing the mass ratio beyond that point, as the damper mass would be detuned from the optimum configurations. It is important to point out that while the equivalent damping values calculated show the response with the current damper coefficient, the optimum value changes for each spring and mass setting and would thus result in an even higher equivalent damping. It is also interesting to note that the top-only damped case, with a 200% mass, equivalent to both modules, shows a larger equivalent damping

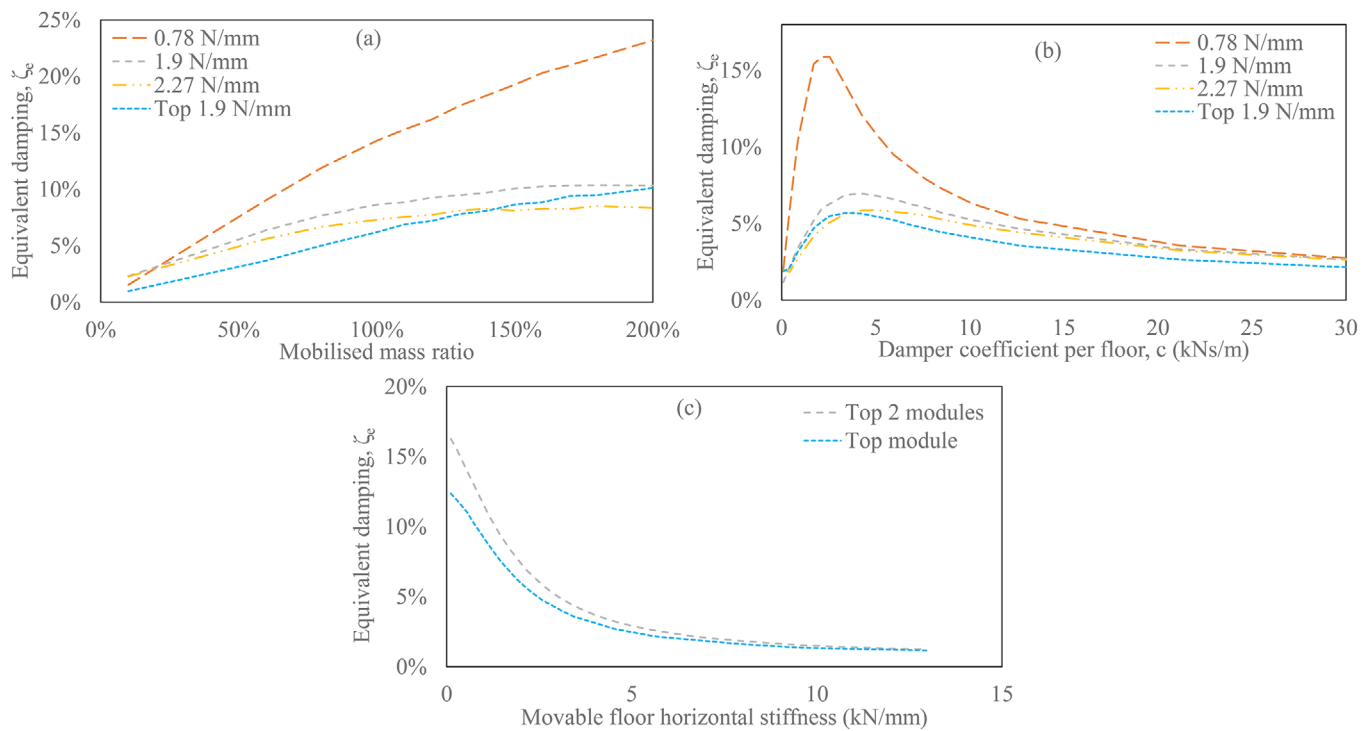


FIGURE 18 | Equivalent first mode damping for varying (a) mobilised mass ratio, (b) floor damper coefficient and (c) floor horizontal stiffness for the cases mobilising the ‘top’ or ‘top 2 modules’.

value than that of the same spring stiffness of 1.9 N/mm, for the 100% mass, which mobilises the same mass. Although the total mass is increased in the 200% case, the fact that the damper mass is placed higher up in the model results in a significant benefit.

6.2 | Influence of Damper Coefficient

The influence of the damper coefficient for the four tested configurations is shown in Figure 18b. It is evident that each spring stiffness has an optimal damper coefficient that maximises the level of damping. Nonetheless, as shown in the figure, the values are relatively close to each other, underlying the robustness of the system to provide a nearly optimum response even if the damper coefficient is not perfectly tuned to the optimal value.

6.3 | Influence of Spring Stiffness

Finally, the equivalent damping levels achieved for different full-scale floor horizontal stiffnesses when mobilising the top module or the top two modules, equivalent to 12 or 24 floors in the prototype, are shown in Figure 18c. As expected, a more flexible movable floor relative to the core would lead to increased differential displacements, a longer damper mass period, and higher equivalent damping values. This trend holds for both configurations. Notably, although the configuration that mobilises only the top module has approximately half the mass of that of the top two, it achieves only about 20% less damping. This can be attributed to the enhanced effectiveness of engaging the uppermost floors, which are more responsive to first-mode vibrations. Given that the top module, or 12 floors, configura-

tion entails roughly half the implementation cost, this finding suggests a potentially attractive trade-off for certain applications. Performing a cost-benefit analysis for each specific application can therefore help identify the optimal number of floors to mobilise, maximising the system efficiency and resilience while minimising implementation costs.

7 | Conclusions

This paper examined the experimental response of a proposed large-mass IDS using a scaled physical model of an illustrative 300-m tall building under harmonic and seismic excitations. The results provided valuable insights into the response of the building incorporating the IDS, validating its capability to bring substantial benefits due to enhanced building efficiency and resilience. The response was investigated using a 1:300 dynamically scaled model that explicitly incorporates the IDS and could also replicate a conventional rigid structure without altering the main dynamic characteristics. To accurately replicate the dynamic response of the prototype while enabling the explicit incorporation of the damping system, the study adopted a novel genetic algorithm-based workflow for parametric model generation that drastically eases the complexities associated with the design and fabrication of dynamically scaled models. The characteristics of the scaled physical model were verified through free vibration tests and shown to provide a nearly perfect match to the prototype building.

The experimental response was subsequently evaluated under seismic excitations using seven selected records, at three different intensity levels, and for five different configurations by adjusting

the spring-damper settings that allow the floor to move differently from the core. These configuration included the following: (i) a rigid 'non-damped' building; three cases with different horizontal spring stiffnesses to reflect different levels of motion, namely, (ii) 0.78 N/mm, (iii) 1.9 N/mm and (iv) 2.27 N/mm at the model scale; and allowing the top third of the building to act as damper mass; and (v) a case where only the top sixth of the building is allowed to move differently with a baseline spring stiffness of 1.9 N/mm.

From the harmonic tests, it was possible to verify the behaviour of the physical model when compared to the numerical simulations, assess the relationship between the core and the movable skin response, including the level of differential displacement and phase offset, and determine the equivalent level of damping provided by the physical model. Importantly, by examining the response under the seismic excitations and comparing the results from the four damped configurations to those of a conventionally rigid counterpart, it was possible to observe the large levels of response reduction enabled by the IDS. The maximum top displacement was reduced by an average of up to 31%, 38% and 39% for the seven selected records at the low, moderate and high intensities considered. The differential displacements between the core and the skin were also shown to remain within serviceability limits and only exceeded conventional environmental service detailing allowances for the very large intensity events, where average top displacement was reduced from H/187 down to H/334. The results from the experimental testing were also compared to those obtained from the digital simulations and were observed to be a relatively close match, thus validating the feasibility of using dynamically scaled or aeroelastic models for combined wind and seismic testing for conventional buildings and those explicitly incorporating complex damping systems.

Further assessment of the sensitivity of the model and role of the three key parameters, namely the mass, damper coefficient and spring stiffness, was carried out using the FEM of the physical model, and their influence was compared in order to inform practical implementation and illustrate the robust nature of the system. For the current application, it was shown that implementing the IDS in the top sixth of the building resulted in a very similar response to that obtained with implementation in the top third, albeit halving the costs. This was therefore identified as a potentially more cost-efficient approach for this application, highlighting the benefits of performing cost-benefit analyses that respond to the particularities of each specific case.

Overall, the results of this experimental investigation demonstrate the significant potential of the proposed large-mass integrated damping arrangement in substantially enhancing the efficiency and resilience of tall buildings. This can offer direct cost and carbon savings and thus have a significant contribution in the path to transition to a Net Zero built environment, where the benefits of urban density are combined with lean and efficient tall building designs.

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Data Availability Statement

Some of the data or models used in this study are available from the corresponding author upon reasonable request.

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Supporting Information

Additional supporting information can be found online in the Supporting Information section.

Supporting File 1: eqe70050-sup-0001-SuppMat.zip