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Control Action of a Tuned Mass Damper in Mitigating Earthquake-Induced Structural Pounding Between Building Floors

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Abstract
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Abstract

Passive control devices are widely used in the reduction of lateral vibrations of building structures. The control of these lateral vibrations is vital in preventing collisions between adjacent building structures during a seismic event. These collisions modify the dynamic behavior of the intervenient structures and cause substantial local damage. In this way, the implementation of a Tuned Mass Damper (TMD) can be a solution in mitigating the earthquake-induced building pounding. However, there are concerns regarding the effectiveness and practical applicability of solutions for this purpose. In fact, non-linear inelastic behavior of building structures, expected during earthquakes, is one of these concerns that should be considered in the assessment of the TMD effectiveness. Hence, this study addresses the investigation of the control action effectiveness of a TMD in reducing the lateral displacements of one of two building structures modelled with elastic and inelastic behavior that are prone to earthquake-induced structural pounding. Results show that TMD is effective in reducing displacements, pounding forces, and number of impacts, under elastic building behavior. However, when inelastic behavior is considered the TMD becomes less effective.

Author keywords

Earthquake-induced structural pounding; Non-linear inelastic behavior; Tuned Mass Damper (TMD)

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Control Action of a Tuned Mass Damper in Mitigating Earthquake-Induced Structural Pounding Between Building Floors

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Abstract. Passive control devices are widely used in the reduction of lateral vibrations of building structures. The control of these lateral vibrations is vital in preventing collisions between adjacent building structures during a seismic event. These collisions modify the dynamic behavior of the intervenient structures and cause substantial local damage. In this way, the implementation of a Tuned Mass Damper (TMD) can be a solution in mitigating the earthquake-induced building pounding. However, there are concerns regarding the effectiveness and practical applicability of solutions for this purpose. In fact, non-linear inelastic behavior of building structures, expected during earthquakes, is one of these concerns that should be considered in the assessment of the TMD effectiveness. Hence, this study addresses the investigation of the control action effectiveness of a TMD in reducing the lateral displacements of one of two building structures modelled with elastic and inelastic behavior that are prone to earthquake-induced structural pounding. Results show that TMD is effective in reducing displacements, pounding forces, and number of impacts, under elastic building behavior. However, when inelastic behavior is considered the TMD becomes less effective.

Keywords: Tuned Mass Damper (TMD) · Non-linear inelastic behavior · Earthquake-induced structural pounding

1 Introduction

The ultimate goal in the investigation of earthquake-induced structural pounding between adjoining building structures is its mitigation. Different solutions and methods were proposed in recent years [1] and may be classified according to Valles and Reinhorn [2] as methods to prevent pounding, to strengthen structures, and techniques to reduce pounding effects in the structures.

The establishment of a minimum gap size, the target of innumerous investigations [3, 4] related with methods to prevent pounding, is viewed as the most natural and efficient way to avoid pounding, despite inherent concern with practical applicability. More methods can be the implementation between the adjacent buildings of link elements,

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rubber bumpers, or collision shear walls at the locations of contact [5, 6]. The addition of lateral stiffness in the structures constitutes the method to strengthen the structures [7]. Finally, techniques that reduce pounding are related to the employment of devices that provide supplemental energy to the structures and enhance the dynamic performance of the structures. Tuned Mass Dampers (TMD) [8, 9], Shared TMD (STMD) [10], and Magneto-Rheological dampers [11, 12] are examples of such devices that can effectively reduce dynamic vibrations and reduce pounding between adjacent buildings.

This study intends to assess the effectiveness of a TMD applied to the most flexible structure to reduce its lateral displacements and avoid structural impacts with the neighboring building structure. The two structures are first modeled in OpenSees [13] using a concentrated plasticity approach, in which plastic hinges with finite length are positioned at the ends of columns and beams. To simplify the current model, adequate for a parametric study that comprises the calculation of pounding forces, an optimization procedure with a Genetic Algorithm (GA) is carried out to calibrate the parameters of a macro hysteresis model developed in MATLAB [14]; such OpenSees model will attempt to replicate the non-linear inelastic behavior of the structural floors. This simplification will allow the use of unusually small time-steps in the simulations, though typical in pounding problems, to capture and calculate pounding forces.

The scenarios considered for comparison are earthquake-induced structural pounding between the floors of two three-story reinforced concrete (RC) building structures with elastic or inelastic behavior. The TMD optimal mass is assessed so to minimize the number and magnitude of the pounding forces.

Results show that the TMD can be effective in reducing lateral displacements, magnitude of pounding forces, and number of impacts. However, when inelastic behavior is considered, the TMD becomes less effective and may not be a good solution.

2 Numerical Formulation

2.1 Finite Element Model with Finite Length Lumped Plasticity

The numerical models built in OpenSees are briefly presented in the next paragraphs. Two RC three-story structures are considered: Building 1, with less mass and stiffness per story; and building 2, with more mass and stiffness per story. Tables 1, 2 and 3 present the steel reinforcement characteristics of the structural elements (beams and columns), while Fig. 1 illustrates the buildings and their relative positions. Moreover, Fig. 1a depicts the locations of critical sections (plastic hinges) and identification of general sections of columns and beam elements; it also includes a legend explaining the terms presented in the Tables. The constitutive laws of confined and unconfined concrete with compressive strength of 30 MPa, and steel with yield strength of 500 MPa applied to the respective fibers of the corresponding sections are also presented in Fig. 1a. The dimensions of the columns and beams' sections of building 1 are 25 × 25 and 35 × 25 cm^2, and of building 2 are 30 × 30 and 40 × 30 cm^2, respectively. The slab thicknesses of buildings 1 and 2 are, respectively, 15 and 20 cm.

Table 1. Steel reinforcement of the columns' critical sections in Buildings 1 and 2.

Building 1						
St.	Corner		Internal		Edge	
	Longitudinal	Transversal	Longitudinal	Transversal	Longitudinal	Transversal
1	C.4Ø12 + Ft.4Ø12; F.4Ø12 + Ft.4Ø12	2lØ6//8	C.4Ø16 + Ft.4Ø12; F.4Ø16 + Ft.4Ø12	2lØ6//8	C.4Ø20 + Ft.4Ø12; F.4Ø20 + Ft.4Ø12	2lØ6//8
	C.4Ø12; F.4Ø12	2lØ6//8	C.4Ø16; F.4Ø12	2lØ6//8	C.4Ø20; F.4Ø12	2lØ6//8
2	C.4Ø12; F.4Ø12	2lØ6//8	C.4Ø16; F.4Ø12	2lØ6//8	C.4Ø20; F.4Ø12	2lØ6//8
	C.4Ø12; F.4Ø12	2lØ6//8	C.4Ø16; F.4Ø12	2lØ6//8	C.4Ø20; F.4Ø12	2lØ6//8
3	C.4Ø12; F.4Ø12	2lØ6//8	C.4Ø12; F.4Ø12	2lØ6//8	C.4Ø12; F.4Ø12	2lØ6//8
	C.4Ø12; F.4Ø12	2lØ6//8	C.4Ø12; F.4Ø12	2lØ6//8	C.4Ø12; F.4Ø12	2lØ6//8
Building 2						
1	C.4Ø12 + Ft.4Ø12; F.4Ø12 + Ft.4Ø12	3lØ6//8	C.4Ø16 + Ft.4Ø12; F.4Ø16 + Ft.4Ø12	3lØ6//8	C.4Ø16 + Ft.4Ø12; F.4Ø16 + Ft.4Ø12	3lØ6//8
	C.4Ø12; F.4Ø12	3lØ6//8	C.4Ø16; F.4Ø12	3lØ6//8	C.4Ø16; F.4Ø12	3lØ6//8
2	C.4Ø12; F.4Ø12	3lØ6//8	C.4Ø16; F.4Ø12	3lØ6//8	C.4Ø16; F.4Ø12	3lØ6//8
	C.4Ø12; F.4Ø12	3lØ6//8	C.4Ø16; F.4Ø12	3lØ6//8	C.4Ø16; F.4Ø12	3lØ6//8
3	C.4Ø12; F.4Ø12	3lØ6//8	C.4Ø12; F.4Ø12	3lØ6//8	C.4Ø16; F.4Ø12	3lØ6//8
	C.4Ø12; F.4Ø12	3lØ6//8	C.4Ø12; F.4Ø12	3lØ6//8	C.4Ø16; F.4Ø12	3lØ6//8

Table 2. Steel reinforcement of the beams' critical sections in Building 1.

St.	Longitudinal	Transversal
1/2	T.3Ø12 + Sp.2Ø10; B.3Ø12	2lØ6//8
	T.3Ø12 + Sp.2Ø12; B.3Ø12	2lØ6//8
	T.3Ø12 + Sp.2Ø12; B.3Ø12	2lØ6//8
	T.3Ø12 + Sp.2Ø10; B.3Ø12	2lØ6//8
3	T.2Ø10 + Sp.2Ø10; B.3Ø12	2lØ6//8
	T.2Ø10 + Sp.2Ø10; B.3Ø12	2lØ6//8
	T.2Ø10 + Sp.2Ø10; B.3Ø12	2lØ6//8
	T.2Ø10 + Sp.2Ø10; B.3Ø12	2lØ6//8

Building 1 has 57085 kg and 49022 kg of story and roof masses, respectively. Building 2 has 96265 kg and 84715 kg of story and roof mass, respectively.

Buildings 1 and 2 have an elastic stiffness per floor of 39675.8 kN/m and 109695.6 kN/m, respectively: resulting in natural periods of 0.515 s, 0.186 s, and 0.131 s for building 1; and 0.404 s, 0.146 s, and 0.103 s for building 2. These previous natural periods were calculated based on the lateral stiffnesses of the columns per story, and were consistently evaluated with the modal analysis carried out in OpenSees.

Table 3. Steel reinforcement of the beams' critical sections in Building 2.

St.	*X*-direction		St.	*Y*-direction	
	Longitudinal	Transversal		Longitudinal	Transversal
1/2	T.2Ø16 + Sp.2Ø10; B.2Ø16	2lØ6//8	1	T.2Ø16 + Sp.1Ø16; B.2Ø16	2lØ6//8
	T.2Ø16 + Sp.1Ø16; B.2Ø16	2lØ6//8		T.2Ø16 + Sp.2Ø12; B.2Ø16	2lØ6//8
	T.2Ø16 + Sp.1Ø16; B.2Ø16	2lØ6//8		T.2Ø16 + Sp.2Ø12; B.2Ø16	2lØ6//8
	T.2Ø16 + Sp.1Ø16; B.2Ø16	2lØ6//8		T.2Ø16 + Sp.1Ø16; B.2Ø16	2lØ6//8
	T.2Ø16 + Sp.1Ø16; B.2Ø16	2lØ6//8	2	T.2Ø16 + Sp.1Ø16; B.2Ø16	2lØ6//8
	T.2Ø16 + Sp.2Ø10; B.2Ø16	2lØ6//8		T.2Ø16 + Sp.1Ø16; B.2Ø16	2lØ6//8
3	T.2Ø12 + Sp.1Ø16; B.2Ø16	2lØ6//8		T.2Ø16 + Sp.1Ø16; B.2Ø16	2lØ6//8
	T.2Ø12 + Sp.1Ø16; B.2Ø16	2lØ6//8		T.2Ø16 + Sp.1Ø16; B.2Ø16	2lØ6//8
	T.2Ø16 + Sp.1Ø16; B.2Ø16	2lØ6//8	3	T.2Ø10 + Sp.1Ø16; B.2Ø16	2lØ6//8
	T.2Ø16 + Sp.1Ø16; B.2Ø16	2lØ6//8		T.2Ø10 + Sp.1Ø16; B.2Ø16	2lØ6//8
	T.2Ø12 + Sp.1Ø16; B.2Ø16	2lØ6//8		T.2Ø10 + Sp.1Ø16; B.2Ø16	2lØ6//8
	T.2Ø12 + Sp.1Ø16; B.2Ø16	2lØ6//8		T.2Ø10 + Sp.1Ø16; B.2Ø16	2lØ6//8

2.2 Shear-Type Model with Macro-hysteresis Model

A simplified model is now developed, as an attempt to replicate the structures' overall dynamic non-linear behavior modeled with OpenSees in the previous subsection.

The simplified model is based on lumped masses at the story levels connected by non-linear shear springs and dashpots, as further illustrated in Figs. 2 and 4.

Hence, the dynamic response will be governed by inter-story shear deformations, where the stories are floor rigid diaphragms and the columns' ends are restrained in terms of rotation. These kinds of models are mostly used to simulate global rather than local effects of structures regular in elevation, leading to overestimations of the story stiffness, strength, and ductility capacity.

The non-linear spring hysteretic behavior is herein considered governed by a smooth hysteresis model (Fig. 2), developed by Sivaselvan and Reinhorn [15], a further modification of the original Bouc-Wen model.

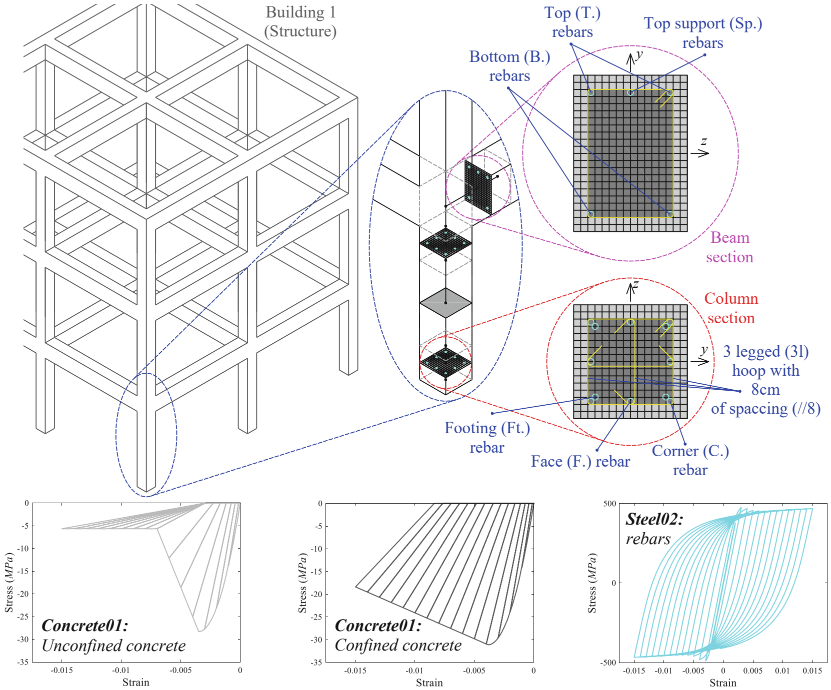
This hysteretic model can represent the macro-hysteretic behavior of moment-resisting frames with reasonable accuracy [16, 17]. The model has the advantage of being defined by the following differential equation

$$\dot{f}_r^* = k_{hyst}\dot{x} \Leftrightarrow \dot{f}_r^* = \dot{x}(R_K - a)k_0\left\{1 - \left|\frac{f_r^*}{f_{r,Y}^*}\right|\left[\eta_1 \mathrm{sgn}\left(f_r^*\dot{x}\right) + \eta_2\right]\right\} \quad (1)$$

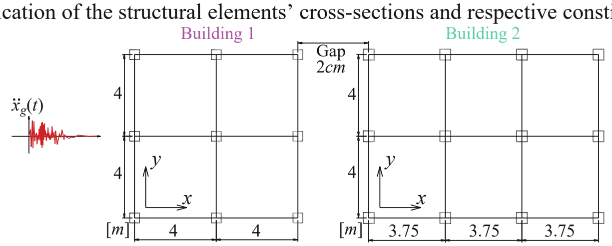
in which k_{hyst} is modified and changed to k_H, to include the pinching effect

$$k_H = k_{hyst}k_{slip-lock}/\left(k_{hyst} + k_{slip-lock}\right) \quad (2)$$

where $f_r{}^*$ and $f_{r,Y}{}^*$ are the hysteretic force and corresponding yielding force, x is the displacement, k_0 is the elastic stiffness, k_H is the total non-linear stiffness, k_{hyst} is the



(a) Identification of the structural elements' cross-sections and respective constitutive laws.



(b) Plan view of the buildings under study and their relative position.

Fig. 1. Equivalent diagonal strut macro-modeling representation.

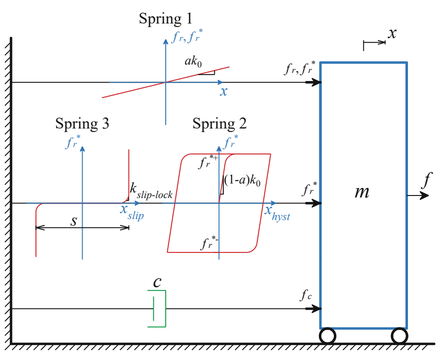


Fig. 2. Lumped mass model with non-linear hysteretic springs.

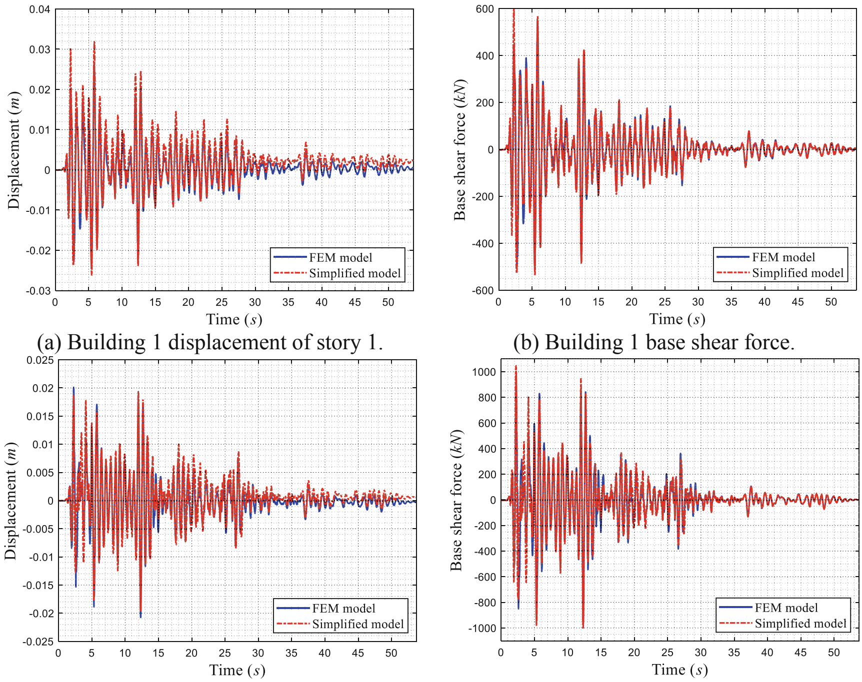
hysteretic stiffness, $k_{\text{slip-lock}}$ is the slip-lock stiffness, R_K is related with stiffness degradation, a is the post-yield to initial stiffness ratio, n_W is the parameter controlling the smoothness transition from elastic to post-yielding range, and η_1 and η_2 are parameters controlling the shape of the unloading path. More information about the hysteretic parameters can be found in [15] and were also detailed by Folhento *et al.* [18].

The yield forces per story were calculated through non-linear static analyses of each story of each building with the corresponding axial forces acting on the columns. The hysteretic parameters' values were obtained by an optimization procedure using a GA, minimizing the Root Mean Square Error between the story shear forces and the displacements of the simplified and refined models.

Figure 3 shows the base shear force and the first story displacement responses of buildings 1 and 2 under El Centro's earthquake (PGA: 2.76 m/s^2) [19].

The restoring force can then be computed by [20]

$$f_r = f_r^* + a k_0 x \tag{3}$$



(a) Building 1 displacement of story 1.
(b) Building 1 base shear force.
(c) Building 2 displacement of story 1.
(d) Building 2 base shear force.

Fig. 3. Comparison between Finite Element Model (FEM) and simplified models' responses.

and is introduced in the restoring forces vector, $\underline{F}_r$, in the dynamic equilibrium equation of the multiple degrees of freedom (MDOF) system

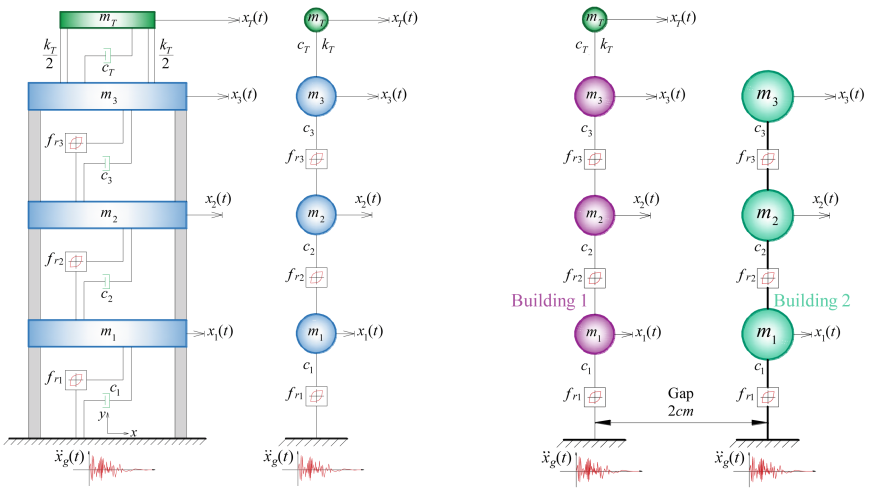
$$\underline{\underline{M}}\ddot{\underline{X}}(t) + \underline{\underline{C}}\dot{\underline{X}}(t) + a\underline{\underline{K}}\underline{X}(t) + \underline{F}_r^*\left[\dot{\underline{X}}(t)\right] = -\underline{F}_p(t) - \underline{\underline{M}}\lambda_g \ddot{x}_g(t) \tag{4}$$

where: $\underline{\underline{M}}$, $\underline{\underline{C}}$, and $\underline{\underline{K}}$ are, respectively, the mass, damping, and the stiffness matrices; $\underline{X}(t)$, $\underline{\dot{X}}(t)$, and $\underline{\ddot{X}}(t)$ are, respectively, the displacement, velocity, and acceleration vectors; $\underline{\ddot{X}}_g(t)$ is the dynamic loading vector, and $\underline{\lambda}_g$ is the influence coefficient vector. Damping was obtained by a linear combination of the mass and stiffness using the frequencies and damping ratios of the first two modes of vibration.

Equation (4) is derived for each building and contact is detected when there is an interpenetration between the buildings. The pounding force vector, $\underline{F_p}$, comprises the pounding forces per story and are computed based on the Kelvin-Voight impact model [21] (disregarding the negative pounding force at the end of the impacts), considering a coefficient of restitution of 0.65, common in structural pounding problems and an impact stiffness calculated as the axial stiffness of the stiffer floor. The problem was mathematically formulated using the state-space formulation, and the fixed-step 4th order Runge-Kutta method was used for integration of differential equations. A bigger time-step was used, $\Delta t = 5 \times 10^{-3}$ s; but for a condition of proximity of the structures, to better capture the pounding force, a smaller time-step was used, $\Delta t = 5 \times 10^{-5}$ s.

2.3 Tuned Mass Damper Optimal Parameters

As mentioned, a TMD is applied to the most flexible structure as shown in Fig. 4.



(a) TMD applied to an MDOF system. (b) Pounding problem layout under study.

Fig. 4. Representation of the pounding problem controlled with a Tuned Mass Damper.

The optimum parameters of the TMD applied to building 1 are derived according to Sadek et al. [22], which considers a procedure similar to single degrees of freedom (SDOF) in determining the optimum tuning ratio, f, and damping ratio of the TMD, ξ_T. The ratio between the TMD mass, m_T, and the generalized mass for the fundamental mode for a unit participation factor is obtained as follows

$$\mu = m_T / \phi_1^T \underline{\underline{M}} \phi_1 \tag{5}$$

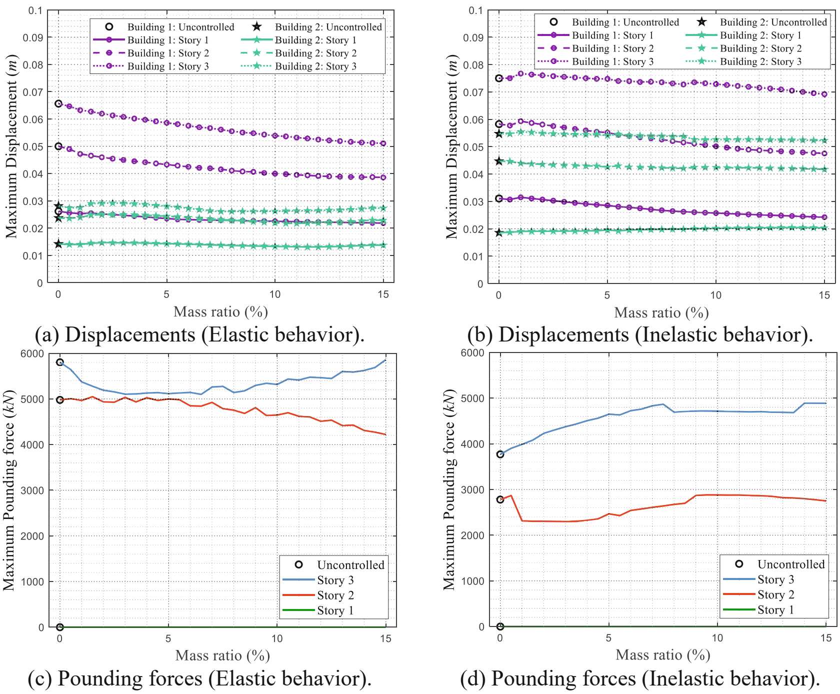
where ϕ_1 is the first mode shape. Parameters f and ξ_T are, respectively, obtained by

$$f = \frac{1}{1+\mu\Phi}\left(1 - \xi\sqrt{\frac{\mu\Phi}{1+\mu\Phi}}\right) \quad \text{and} \quad \xi_{\mathrm{T}} = \Phi\left(\frac{\xi}{1+\mu\Phi} + \sqrt{\frac{\mu}{1+\mu}}\right) \tag{6}$$

in which Φ is the amplitude of the fundamental mode of vibration for a unit modal participation factor calculated at the TMD location, i.e., the top floor of building 1.

3 Discussion of Results

Results, for elastic and inelastic building behavior, are shown in Figs. 5 and 6; they present the maximum absolute displacements and pounding forces, and the number of impacts per story of each building, versus the TMD to the structure mass ratios. The buildings were subjected to the El Centro earthquake above-mentioned. The TMD performance in mitigating earthquake-induced pounding was thus assessed by varying the μ from 0.5% to 15%. Results are now discussed based on Figs. 5 and 6.



(a) Displacements (Elastic behavior). (b) Displacements (Inelastic behavior).

(c) Pounding forces (Elastic behavior). (d) Pounding forces (Inelastic behavior).

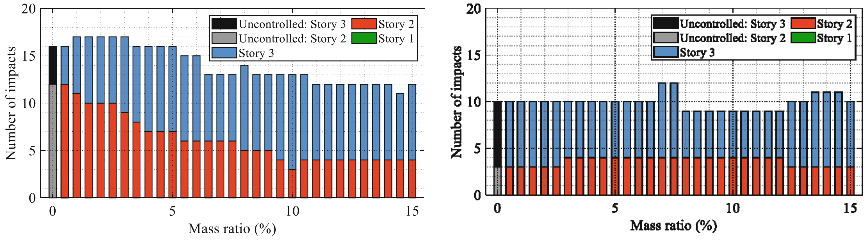
Fig. 5. Absolute maximum responses versus the TMD to structure mass ratio.

The effectiveness of the TMD is more visible when the structures have elastic behavior, revealing decreases of the maximum displacements of building 1, and number of impacts (Figs. 5a and 6a). However, increases in the maximum displacement of building 2 and pounding forces at the top stories are also verified in Fig. 5a and 5c.

When inelastic behavior is considered (Figs. 5b, 5d and 6b), decreases of the maximum displacements of both buildings are verified, mainly, in the controlled structure, and some decreases in the magnitude of the pounding forces at the second story. Nevertheless, increases were verified of the magnitude of the pounding forces, especially, at the top stories. The number of impacts remained almost constant.

A mass ratio of 15% which corresponds to 20 tons of TMD mass would add substantial weight to building 1, leading to great demands of strength and ductility in the structure, viz, at the top story. Based on the overall results, a value of μ between 5% and 7% would lead to the best results taking into account the practical limitations.

To further assess the effectiveness of a TMD in building structures under these conditions, more earthquake actions should be considered, adjusted to a seismic region to account for the seismic effect and different gap sizes between the structures. Due to the inherent inelastic behavior in building structures during a seismic event, a TMD may not be appropriate in mitigating pounding, being more adequate the use of adaptative devices such as semi-active vibration control devices, e.g., MR dampers.



(a) Elastic behavior. (b) Inelastic behavior.

Fig. 6. Number of impacts per story versus the TMD to structure mass ratio.

4 Conclusions

The present study reveals that the TMD applied to the flexible structure reduces its maximum displacements, and the number and magnitude of pounding forces. However, this TMD was not as effective in reducing the effect of earthquake-induced structural pounding when the inelastic behavior of the structures is considered compared to the elastic case. A mass ratio of the TMD between 5% and 7% led to the best results in reducing the dynamic responses and pounding effects. Further studies should consider different control strategies in mitigating earthquake-induced pounding, and additional seismic actions and gap sizes to better support these conclusions.

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11:10 – 12:10	**Keynote 2**	**"Intelligent humanoids: From labs to real world"** Carlos Balaguer, University Carlos III, Spain *Chair: Luís Palma*	Room A
12:10 – 14:00		Lunch	
14:00 – 15:40	**ThA2.SS**	**Geometric and analytic methods for nonlinear data** *Chairs: Fátima Silva Leite and Maria Barbero-Liñán*	Room A
14:00 – 14:20	88	A 4th-order variational problem on SO(3) Authors: Margarida Camarinha Abstract: In this paper we consider a fourth-order variational problem on SO(3) and study its stationary points. This gives rise to a higher-order interpolation method that is one step ahead of the cubic interpolation method for rigid body orientation.	
14:20 – 14:40	30	Optimization on Stiefel Manifolds Authors: Markus Schlarb and Knut Hüper Abstract: Explicit matrix-type formulas for gradient and Hessians of smooth functions on the compact real Stiefel manifold with respect to a whole class of (pseudo-)Riemannian metrics are presented. This includes explicit formulas for corresponding normal spaces and associated orthogonal projections. It turns out that some well-known formulas are reproduced, moreover, it is shown that they are even valid in a much bigger context. All proofs are included, some of them for the first time. A numerical experiment is added as well.	
14:40 – 15:00	18	Variational problems on Riemannian manifolds with constrained accelerations Authors: Alexandre Anahory Simoes and Leonardo Colombo Abstract: We introduce variational problems on Riemannian manifolds with constrained acceleration and derive necessary conditions for normal extremals in the constrained variational problem. The problem consists on minimizing a higher-order energy functional, among a set of admissible curves defined by a constraint on the covariant acceleration. In addition, we use this framework to address the elastic splines problem with obstacle avoidance in the presence of this type of constraints.	
15:00 – 15:20	63	Best fitting geodesic going through the Riemannian mean Authors: Luís Machado and Fátima Silva Leite Abstract: Our main objective here is to derive alternative normal equations for the geodesic fitting problem on compact Lie groups and Grassmann manifolds. This is achieved by constaining the solution to pass through the Riemannian mean of the data, a property which is shared by their Euclidean counterparts and also by using variations of geodesics by geodesics to solve the corresponding optimization problem.	
15:20 – 15:40	31	Dynamical Component Analysis: Matrix Case and Differential Geometric Point of View Authors: Philipp Romberger, Monika Warmuth, Christian Uhl and Knut Hüper Abstract: Dynamical component analysis, a data-driven dimensionality reduction and subspace detection method for multivariate time series is mathematically analyzed. The matrix based formulation is put in perspective to a recently published vector based approach. Moreover, a differential geometric point of view is taken as well. A numerical experiment with the Rössler attractor is added.	
14:00 – 15:40	**ThB2**	**Control in infrastructures** *Chairs: Rui Barros and Fares M'Zoughi*	Room B
14:00 – 14:20	85	Control action of a Tuned Mass Damper in mitigating earthquake-induced structural pounding between building floors Authors: Pedro Folhento, Rui Carneiro de Barros and Manuel Braz-César Abstract: Passive control devices are widely used in the reduction of lateral vibrations of building structures. Controlling these lateral vibrations is vital in preventing collisions between adjacent building structures during a seismic event. These collisions modify the dynamic behavior of the intervenient structures and cause substantial local damage. In this way, the implementation of a Tuned Mass Damper (TMD) can be a solution in mitigating the earthquake-induced building pounding. However, there are concerns regarding the effectiveness and practical applicability of solutions for this purpose. In fact non-linear inelastic behavior of building structures, expected during earthquakes, is one of these concerns that should be considered in the assessment of the TMD effectiveness. Hence, this study addresses the investigation of the control action effectiveness of a TMD in reducing the lateral displacements of one of two building structures modelled with elastic and inelastic behavior that are prone to earthquake-induced structural pounding. Results show that TMD is effective in reducing displacements, pounding forces, and number of impacts, under elastic building behavior. However, when inelastic behavior is considered the TMD becomes less effective.	