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EVALUATION OF THE INTER-STORY ISOLATION SYSTEM APPLIED TO AN EXISTING MASONRY SCHOOL BUILDING

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Abstract

Adding extra stories to an existing building is an interesting application of the Inter-story Isolation System (IIS) that allows to improve the seismic response of existing structures. Assuming that the isolated superstructure is much stiffer than both the isolation system and the existing structure, it behaves like a rigid body, acting as a Tuned Mass Damper for the substructure. Various methods are available in the literature for defining the optimal parameters of the isolation system, but only a few studies focused on this particular application with real case studies. In this paper, this application is shown for an existing masonry school building. The aim of the intervention is to raise the structure, increasing the space available for teaching activities, while also obtaining an improvement in the overall seismic behaviour of the substructure. First, an optimization for calculating the optimal IIS parameters is performed; then, based on the optimal results obtained, the effectiveness of the IIS application is evaluated by performing Time History Analysis, using a set of spectrum-compatible natural records and an equivalent-frame modelling (EFM) of the structure, and comparing the results obtained with and without the IIS.

Keywords: Inter-story isolation system, Tuned mass damper, Seismic isolation, Seismic retrofit, Structural optimization.

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1 INTRODUCTION

Inter-story isolation system (IIS) is an innovative technique that involves inserting seismic isolation layer between two stories rather than at the base of the structure. It allows greater architectural freedom as substructure and superstructure can have different shapes, materials and uses. A further benefit of the technique concerns the possibility of increasing the useful surface of the structure by limiting soil consumption. The latter aspect is especially beneficial for densely populated areas [1, 2].

IIS is also very effective to raise existing buildings with a sufficient margin of resistance to gravitational loads. Indeed, when it is correctly designed, the isolated superstructure acts as a non-conventional Tuned Mass Damper (TMD), allowing to control the dynamic response of the substructure [3, 4, 5, 6, 7, 8].

From an operational point of view, the IIS technique combines the function of seismic isolation and mass damping, and therefore has been analyzed in the literature through various methods [9]. Studies that address the problem from the seismic isolation perspective evaluate mainly the interaction between superstructure and substructure, typically using models with three or more degrees of freedom (e.g., [2, 9, 10, 11, 12, 13, 14]). In contrast, other studies investigate IIS following the TMD approach via frequency domain analyses of simplified 2DOF models [6, 8, 15, 16, 17, 18, 19].

Several studies on the IIS technique are available in the literature, demonstrating the effectiveness of the technique and its optimization. However, only a few studies investigate its application through case studies and 3D Finite Element modelling, especially regarding its use as a seismic retrofit technique [5, 7, 20, 21, 22].

This paper presents an application of the IIS to an existing structure, i.e., a real two-story masonry school building to which an isolated steel story is added. First, the structure and the associated 3D FE models, with and without IIS, are presented. Then, an optimization was performed, simplifying the structure with IIS to a 2DOF system (approach widely used in the literature for TMD system optimization). The effectiveness of the technique is discussed through results obtained from the Time History Analysis performed on the 3D Finite Element model. In addition to the performances obtained by considering the optimized solution, different isolation systems were considered, aiming to evaluate the performance of the technique even for solutions that differ from the optimal one, as well as assessing the reliability of the optimal solution considering structural non-linearity.

2 APPLICATION OF IIS ON A MASONRY BUILDING

2.1 Presentation of the case study building

The case study analyzed is a two-story masonry school building located in the municipality of Padua and built in the 1950s. This structure is representative of the category of masonry school buildings built in the mid-1900s in Italy and is characterized by large interior spaces, long bays and significant inter-story heights (Figure 1).

The structure has a rectangular floor plan (dimensions 25x10.7 m) and a total height of about 11 m. The masonry walls of the building are made of solid brick and mortar, as indicated by the original project. The thickness is 50 cm for external walls and 30 cm for internal walls.

The techniques used for numerical simulation of masonry structures is the equivalent frame method, which allows the generic façade to be represented by a frame composed of 3 types of macro-elements: piers, spandrels and rigid connections (Figure 2). In this study, the approach proposed by Dolce (1989) [23] was applied.

The masonry properties were assumed in accordance with the Italian code (Italian Ministry of Infrastructure and Transport, 2018 [24]), considering the average value of the range of parameters proposed by the standard.

The structure was modeled using the Finite Element software MidasGEN [25]. The non-linear behaviour of masonry was modeled with concentrated FEMA-type hinges [26] whose parameters used for calibration are set according to the Italian code [24].

Figure 1: Case study structure.

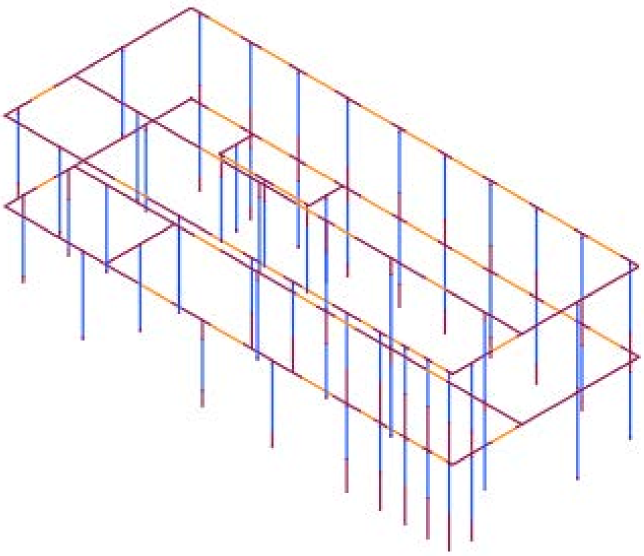

Figure 2: Equivalent frame model of the case study structure.

2.2 Presentation of the added story

The superstructure consists of a seismically isolated story above the existing structure. The structure is composed of steel elements, with the aim of maintaining a rather low mass (that results in 46% of the mass of the substructure).

A system of steel beams has been provided above the existing roof, which acts as a support for the isolation system and distributes the load on the underlying masonry elements, avoiding localized stress peaks. These beams are connected by secondary elements whose function is to stiffen the floor and prevent the buckling effects of the main beams.

HEA180 profiles were used for steel columns, while the beams differed according to the floor considered. Above the isolation system and in the grid of beams placed at the existing roof, IPE400 profiles were used for the main and IPE330 for the secondary beams; on the other hand, IPE300 profiles were used in the superstructure roof for the main and IPE240 for the secondary beams. The type of steel used is S235.

In order to obtain a rigid-body behaviour under dynamic loads, the superstructure was stiffer to horizontal actions through a bracing system.

The superstructure was modeled with linear beam elements, and the strength of the substructure was modified to consider the increase in compression in the masonry.

The isolation system is made with 10 isolators and 5 sliding supports, arranged as shown in Figure 3.

Figure 4 shows the Finite Element model of the structure, including the substructure, isolation system and superstructure.

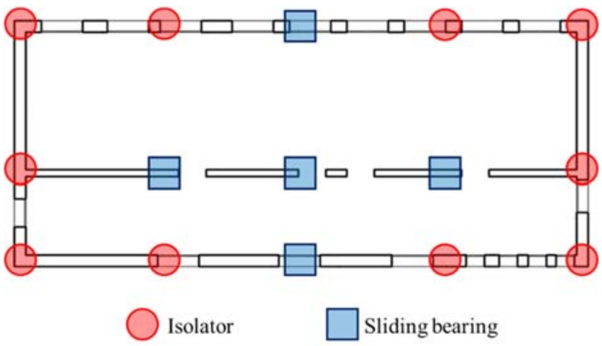

Figure 3: Layout of the isolation system.

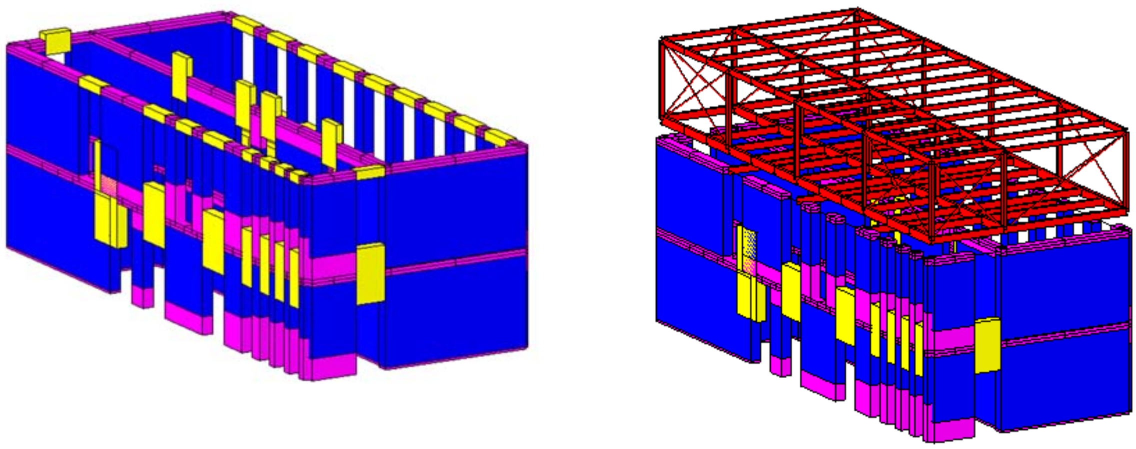

Figure 4: Finite Element model of the structure without (left) and with (right) IIS.

3 DESIGN OF OPTIMAL IIS AS TMD SYSTEM

Conceptually, the IIS subdivides the structure into three main parts, i.e., substructure, isolation system and superstructure.

The dynamic behaviour of IIS is strongly influenced by the isolated mass and, in particular, by the ratio between the isolated mass and the mass of the substructure (μ). As demonstrated in various studies (e.g. [9, 27]), when the mass ratio (μ) is less than 1, the dynamic behaviour can be classified as “mass damping”, and therefore it is possible to neglect the higher vibration modes of the superstructure, simplifying the system with an equivalent 2DOF model.

Using this approach, the substructure is represented by an SDOF system, defined by an angular frequency (ω_{ST}), a damping ratio (ξ_{ST}) and a modal mass (m_{ST}); on the other hand, the isolated superstructure is characterized by the parameters m_{IS} (sum of the masses of isolation system and superstructure), ω_{IS} and ξ_{IS} (angular frequency and damping ratio of the isolation system, respectively).

Normalizing the parameters of the superstructure to those of the substructure, the mass (μ) and frequency (ν) ratios can be defined (Equation 1). Together with ξ_{IS}, those parameters define the non-conventional TMD system to optimize.

Equation 2 shows the matrices of mass (**M**), damping (**C**) and stiffness (**K**) of the 2DOF reduced-order model, as a function of the aforementioned parameters.

$$\mu = \frac{m_{IS}}{m_{ST}} \quad ; \quad \nu = \frac{\omega_{IS}}{\omega_{ST}} \tag{1}$$

$$\mathbf{M} = \begin{bmatrix} 1 & 0 \\ 0 & \mu \end{bmatrix} ; \mathbf{C} = \begin{bmatrix} 2\xi_{ST}\omega_{ST} + 2\mu\xi_{IS}\nu\omega_{ST} & -2\mu\xi_{IS}\nu\omega_{ST} \\ -2\mu\xi_{IS}\nu\omega_{ST} & 2\mu\xi_{IS}\nu\omega_{ST} \end{bmatrix} ; \mathbf{K} = \begin{bmatrix} \omega_{ST}{}^2 + \mu\nu^2\omega_{ST}{}^2 & -\mu\nu^2\omega_{ST}{}^2 \\ -\mu\nu^2\omega_{ST}{}^2 & \mu\nu^2\omega_{ST}{}^2 \end{bmatrix} \tag{2}$$

3.1 Optimization approach and results

The objective of the optimization is improving the substructure performances, minimizing its seismic response in terms of displacements.

To model the stochastic nature of the seismic input, the Power Spectral Density (PSD) function of a zero-mean Gaussian stochastic process, $\mathbf{S}(\omega)$, is generally assumed. Considering this type of process, the random vibration theory produces the following covariance matrix:

$$Cov[\mathbf{x}\mathbf{x}^T] = \int_{-\infty}^{\infty} \mathbf{H}(\omega)\ \mathbf{S}(\omega)\ \mathbf{H}(\omega)^{*T}\, d\omega \tag{3}$$

where $\mathbf{H}(\omega)$ is the displacement frequency response function of simplified 2DOF model and ω is the input frequency of the system. Assuming the seismic excitation as a white noise process, its PSD function is no longer dependent on ω ($\mathbf{S}(\omega)=S_0$); therefore, in order to obtain the optimal isolation parameters (i.e., ν, ξ_{IS}), the minimization of the integrals in Equation 3 (representative of the dynamic response of the system, in terms of displacement) is equivalent to the minimization of the integrals in Equation 4, where $\boldsymbol{\sigma}^2$ is the displacement variance.

$$\sigma^2 / S_0 = \int_{-\infty}^{\infty} \left|\mathbf{H}(\omega)\right|^2\, d\omega \tag{4}$$

Therefore, on the basis of these hypothesis, the objective of the optimization (J_d) is find the optimal ν, ξ_{IS} values (from 0 to 1) that minimize the integral in Equation 4. The optimization can be defined as follow:

$$J_d : \begin{cases} \min\limits_{\xi_{IS},\nu} \int\limits_{-\infty}^{\infty} |\mathbf{H}(\omega)|^2 \, d\omega \\ subjected\ to\ \begin{cases} 0 < \xi_{IS} \le 1 \\ 0 < \nu \le 1 \end{cases} \end{cases} \tag{5}$$

To apply the aforementioned method to the specific case study, the first step consists of defining the mass ratio (μ). To calculate μ, it is necessary to perform modal analysis of the substructure and calculate the modal mass.

The modal analysis results are reported in Table 1. For this case study, the first mode was chosen for the x direction ($\omega_{ST,x}$=28.74 rad/s, participation mass of 78.5%) and the second mode for the y direction ($\omega_{ST,y}$=32.84 rad/s, participation mass of 62.9%, neglecting the torsional contribution). The mode shapes are shown in Figure 5 and the modal masses are 414.7 tons along the x direction and 432.59 tons along the y direction; consequently, the mass ratios in the two directions are 0.88 along x (μ_x) and 0.85 along y (μ_y).

Then the optimization J_d can be performed, aiming to find optimal ν and ξ_{IS} parameters. Results are shown in Table 2, where, based on the vibration mode considered, the optimal stiffness (k_{IS}) and damping constant (c_{IS}) of the isolation system were also reported.

Mode	ω_{ST} [rad/s]	m-Tx [%]	m-Ty [%]	m-Rz [%]
1	**28.74**	**78.53**	**0.01**	**0.38**
2	**32.84**	**0.04**	**62.89**	**18.21**
3	37.20	0.06	0.004	0.01
4	44.33	0.12	7.77	23.40
5	45.75	0.47	10.90	35.04

Table 1: FE modal analysis results: angular frequencies (ω_{ST}) and % of participating mass (model without IIS).

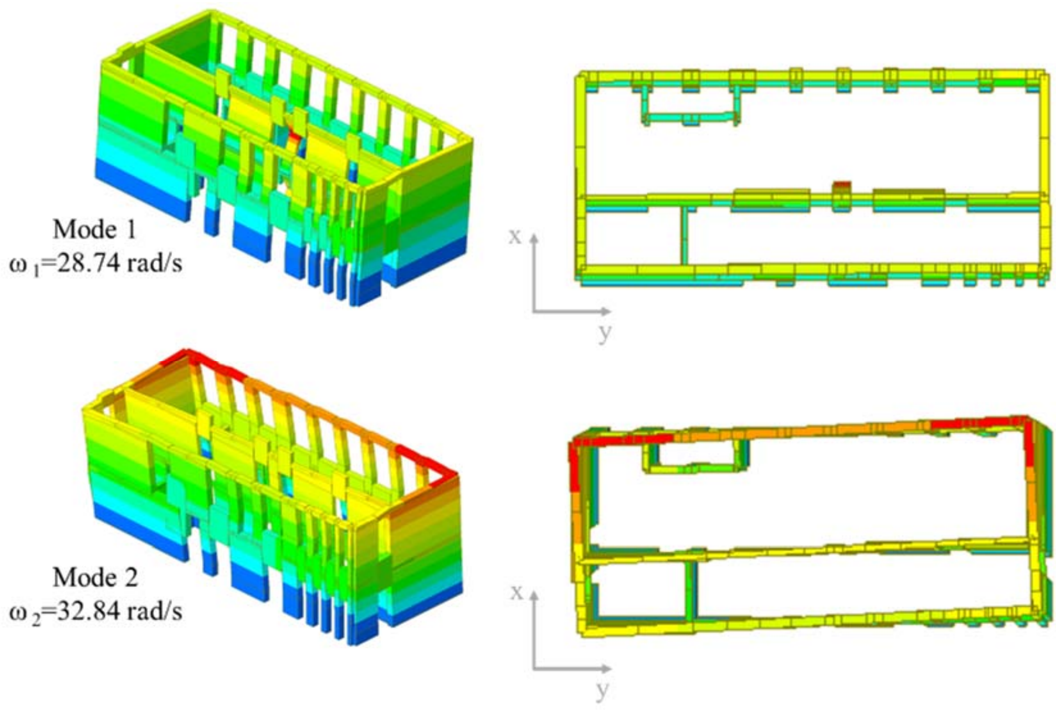

Figure 5: Main vibration modes of the existing structure.

Direction	μ [-]	ν	ξ_{IS} [-]	ω_{ST} [-]	k_{IS} [kN/m]	c_{IS} [kNs/m]
x	0.88	0.33	0.43	28.74	3337.11	298.10
y	0.85	0.34	0.42	32.84	4698.23	346.63

Table 2: Optimal IIS parameters.

4 ASSESSMENTS THROUGH TIME HISTORY ANALYSIS

4.1 Seismic inputs

For the Time History Analysis, 7 bidirectional natural records were chosen from the SIMBAD Database [28]. These records were scaled to be compatible (on average) with the Type 1 elastic response spectrum of EC8 [29], considering a bedrock acceleration (a_g) of 0.25 g and a soil type B, which results in a peak ground acceleration of 0.3 g. The main details of these accelerograms are reported in Table 3, and the associated acceleration spectra are shown in Figure 6 together with the EC8 reference spectrum.

ID	Earthquake	Date	Mw	Epicentral distance [km]	Scale factor X	Scale factor Y
Acc.1	Eastern Fukushima Prefecture	2011/04/11	6.6	26.24	1.30	1.38
Acc.2	Imperial Valley	1979/10/15	6.5	24.68	1.59	1.48
Acc.3	S. Suruga Bay	2009/08/10	6.2	25.38	0.60	0.97
Acc.4	Friuli 1st shock	1976/05/06	6.4	21.72	0.79	0.72
Acc.5	Eastern Fukushima Prefecture	2011/04/11	6.6	27.56	2.33	1.41
Acc.6	Irpinia	1980/11/23	6.9	21.79	1.34	1.93
Acc.7	Irpinia	1980/11/23	6.9	18.85	1.43	1.58

Table 3: Selected bidirectional natural records.

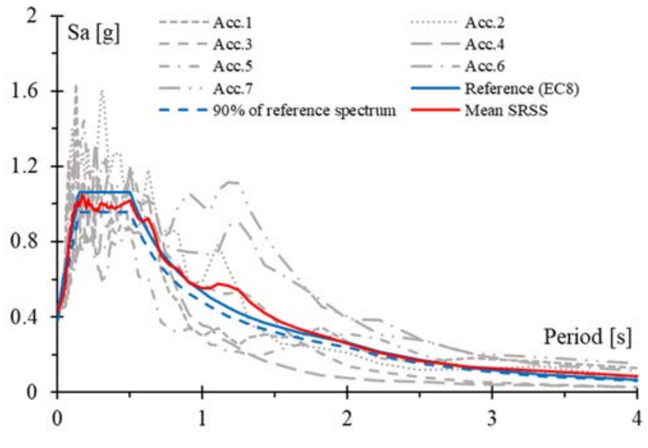

Figure 6: Acceleration response spectra of the selected records and EC8 reference spectrum.

4.2 Time History results

The results obtained from the analyses in terms of inter-story drift and story accelerations are shown below. Two points of the structure, located at the corners (as shown in Figure 7), were considered for assess the structural performance. In particular, first, the results obtained from the analysis of the existing structure (Existing) and those of the structure with optimized IIS (With IIS - OPT) are compared. Then, to evaluate the effect of the isolation period on the substructure response, three additional values of T_{IS} (=2, 3 and 4 s) were analyzed; in such cases, the isolation period was assumed equal in both directions. Table 4 shows the isolation parameters related to the T_{IS} values analyzed (the optimal damping value ξ_{IS} were assumed). The Time History Analysis was performed for both linear and non-linear structural models.

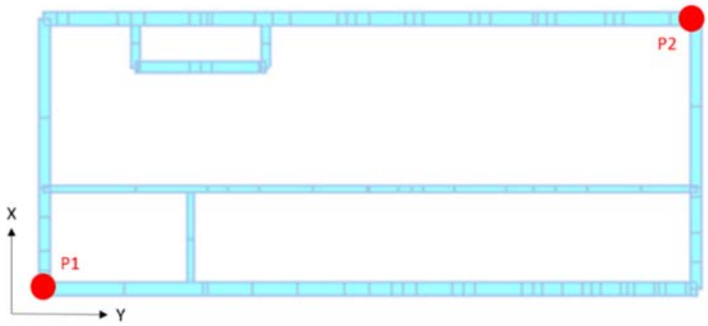

Figure 7: Positions referred to in the results below (P1 and P2).

T_{IS} [s]	ξ_{IS} [-]	ω_{IS} [rad/s]	k_{IS} [kN/m]	c_{IS} [kNs/m]
2	0.42	3.14	361.42	96.64
3	0.42	2.09	160.63	64.42
4	0.42	1.57	90.35	48.32

Table 4: IIS parameters referring to cases with T_{IS}=2, 3 and 4 s.

Figure 8 shows the inter-story drift, comparing the results obtained for the existing structure and with the IIS. Specifically, considering the case with optimal IIS parameters, in both directions, the maximum inter-story drift decreases. In the y direction, where the structure is more rigid, the drift decreases along the height (i.e., going from the first to the second floor). In contrast, in the x direction, where the structure is more flexible, the drift trend along the height reverses. This happens because the structural stiffness, that exhibits non-linear behaviour, collapses during the earthquake, and the isolation system, optimized for linear structural properties, becomes excessively rigid, dragging with it the story below.

Globally, the largest drift values occur at the control point P1. In the x direction, the drift value obtained in correspondence of the considered points are 1.16% (P1) and 0.87% (P2), while in the y direction are 0.37% (P1) and 0.30% (P2).

Analyzing the cases with variable isolation period, it can be stated that, overall, by reducing the stiffness of the isolators (i.e., increasing T_{IS}), a significant improvement in performance can be achieved, both with respect to the existing and the optimized case (also demonstrating that the IIS technique can be beneficial for a wide range of isolation parameters).

Among the cases analyzed, the best solution, i.e., the one that achieves the minimum value of drift, is obtained with T_{IS}=2 s in x direction (P1) and with T_{IS}=4 s in y direction (P2).

As for the floor accelerations, with the IIS are significatively reduced, as shown in Figure 9. Again, the largest peaks occur along the outer perimeter of the structure (P2 in the x direction, where the acceleration is reduced from 0.53 g to 0.37 g, and P1 in the y direction, where the reduction ranges from 0.58 g to 0.43 g).

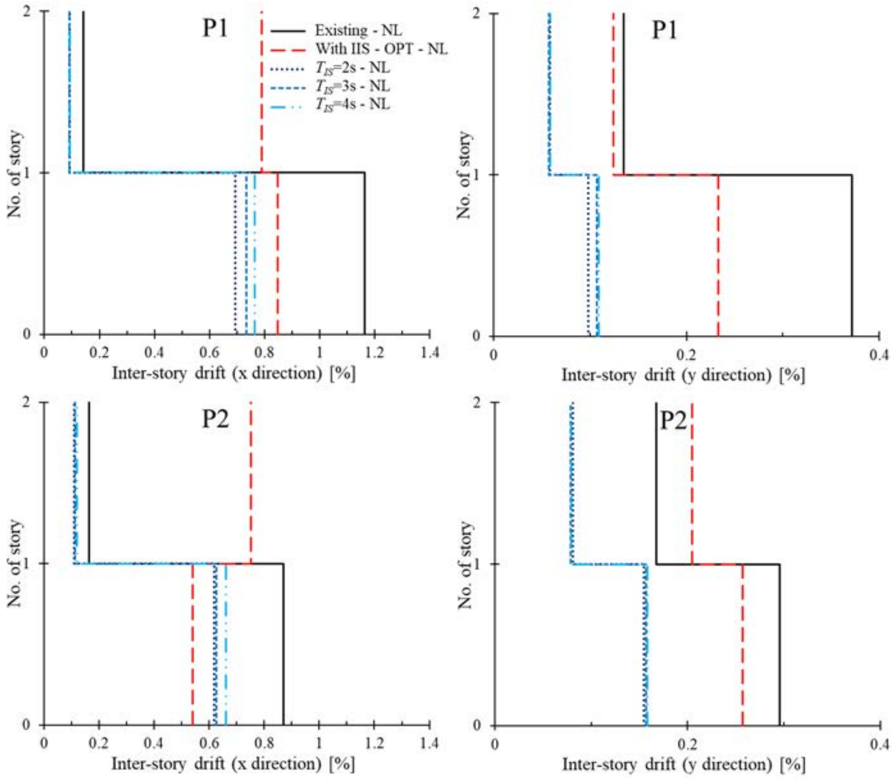

Figure 8: Inter-story drift for the structure with and without IIS, considering the solution obtained with optimal result and with T_{IS}=2, 3 and 4 s (for both x and y directions).

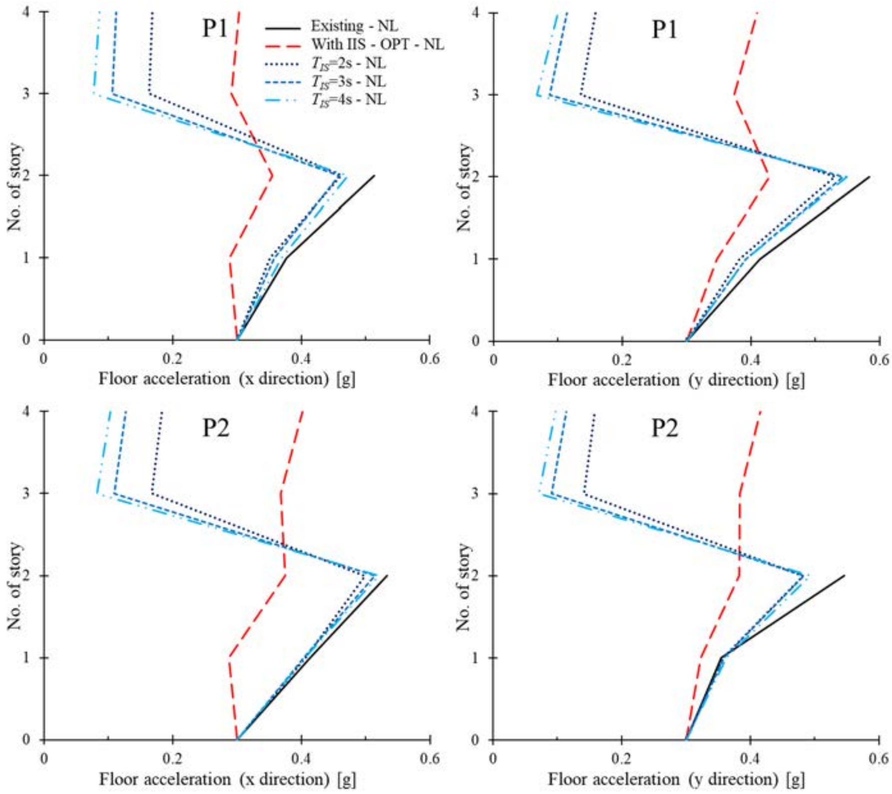

Figure 9: Peak floor acceleration for the structure with and without IIS, considering the solution obtained with the optimal result and with T_{IS}=2, 3 and 4 s (for both x and y directions).

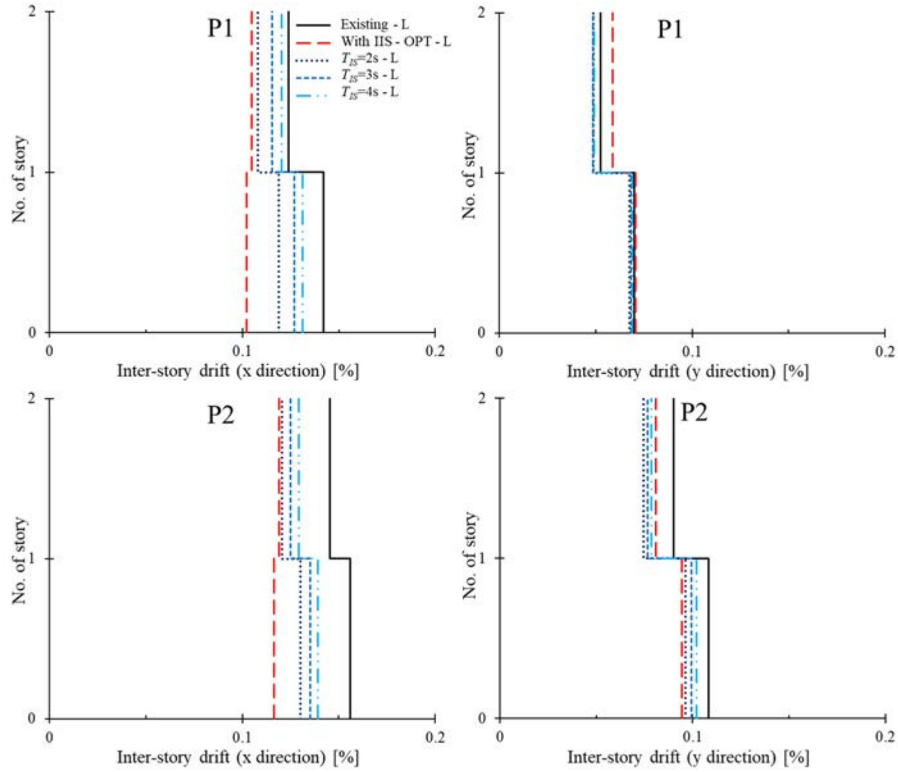

Figure 10: Inter-story drift for the structure with and without IIS obtained from linear analysis, considering the solution obtained with the optimal result and with T_{IS}=2, 3 and 4 s (for both x and y directions).

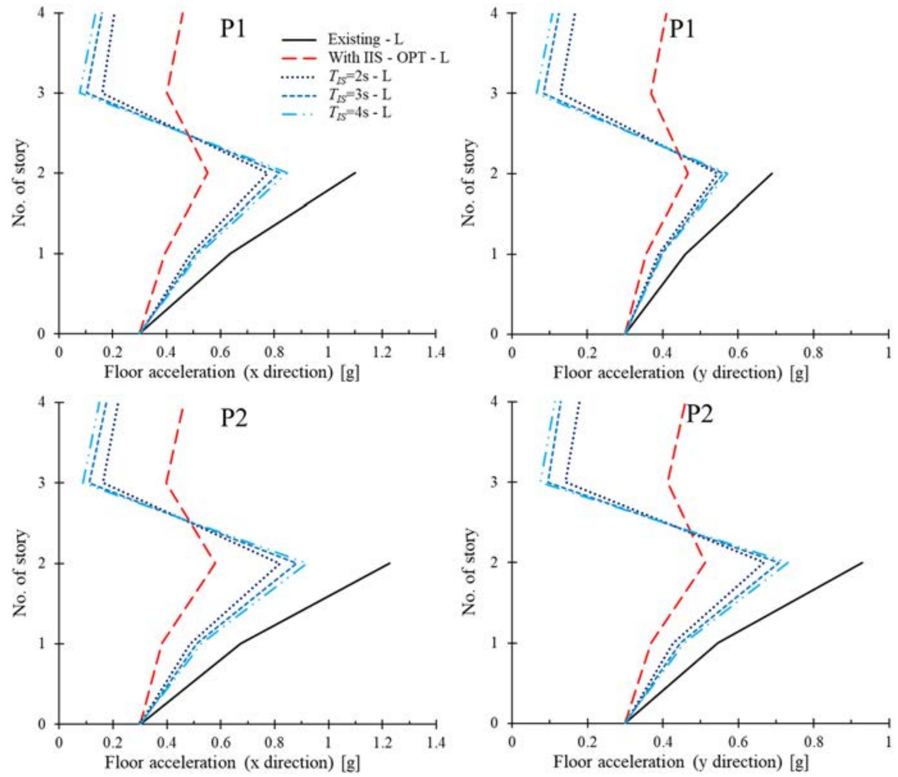

Figure 11: Peak floor acceleration for the structure with and without IIS obtained from linear analysis, considering the solution obtained with the optimal result and with T_{IS}=2, 3 and 4 s (for both x and y directions).

It is possible to state that by increasing the isolation period, also increases the floor accelerations in the substructure (compared the optimized case). The acceleration of the superstructure, on the other hand, is significantly reduced as T_{IS} increases (i.e., as the decoupling between the two structural parts increases).

In addition, with the optimal solution, a smoother acceleration profile is obtained than in all other cases, that is, the difference between the floor accelerations decreases.

4.3 Assessments on linear structural model

For the sake of completeness, a linear structural analysis was performed, to evaluate the validity of the proposed optimization approach.

Figures 10 and 11, demonstrate that when the structure remains in linear field, such as through a structural retrofit or for low-intensity earthquakes, the optimization approach is reliable.

Indeed, the graphs show that in terms of both inter-story drift and absolute acceleration, the best structural performance corresponds to the optimized solution. However, the solutions obtained with T_{IS}=2, 3 and 4 s are also effective in improving the response of the existing structure.

Finally, Figure 12 compare the drift of the isolation system (d_{IS}) obtained from the linear and non-linear structural analysis. Globally, increasing T_{IS} (from OPT to T_{IS}=4 s solution), d_{IS} increases from about 32 mm to 142 mm for the linear solution and from 27 mm to 150 mm for the non-linear one. It is observed also that when the IIS is design to act as a TMD (as in the case of OPT solution), d_{IS} is higher in the linear solution; indeed, in this case, the substructure preserves its stiffness, and the decoupling effect between substructure and superstructure is greater. When the T_{IS} increases, the effect observed for the OPT solution is reversed; in fact, in these situations, the IIS stiffness is significantly smaller than that of the substructure, which, in the non-linear situation, has higher top displacements.

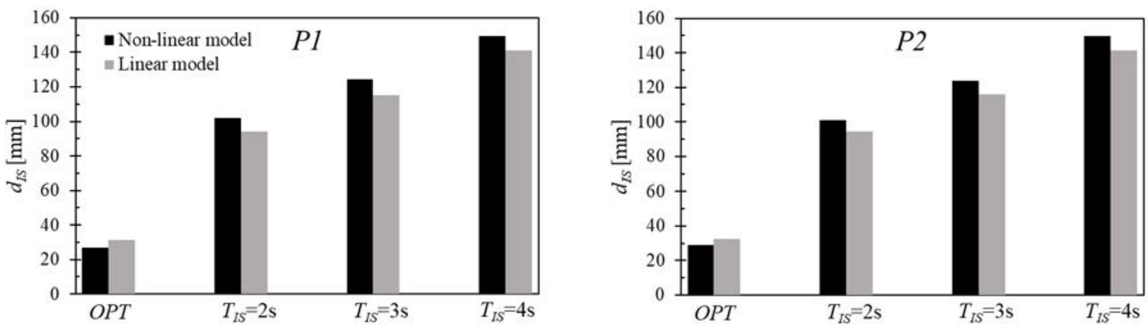

Figure 12: Drift of the isolation system d_{IS} considering the solution obtained with the optimal result and with T_{IS}=2, 3 and 4 s, for the linear and non-linear model.

5 CONCLUSIONS

The paper presents an innovative technique for improving the seismic performance of an existing masonry school building. It consists in raising the structure through a seismically isolated superstructure, which acts as a non-conventional TMD for the substructure. Consistently with the TMD theory, the seismic parameters of the isolators have been optimized with the aim of minimizing the response of the existing structure in terms of displacements.

The seismic response of the substructure was assessed through Time History Analysis (THA), performed on the Finite Element model of the structure.

From the results, it can be drawn the following considerations.

- In general, the IIS allow to reduce the seismic response of the substructure (both in terms of drift and acceleration).
- Considering the calculated optimal solution, the inter-story drift is significatively reduced in both directions; in particular, a reduction of 38% in x direction and of 37% in y direction is observed. Floor accelerations are also significatively reduced with the IIS; in particular, in x direction, the acceleration is reduced of 31%, while in y direction of 30%.
- By comparing different isolation periods, it was noted that the optimization is reliable only in the case where the structure exhibits linear behaviour. In fact, for this specific case study, increasing the isolation period improves the seismic response of the non-linear substructure.
- In general, the results shown the importance of considering structural non-linearity to identify critical issues that may characterize this type of structure. If the optimized intervention (with the OPT solution) does not allow to keep the substructure in the linear field, seems convenient to adopt a more flexible isolation system.

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