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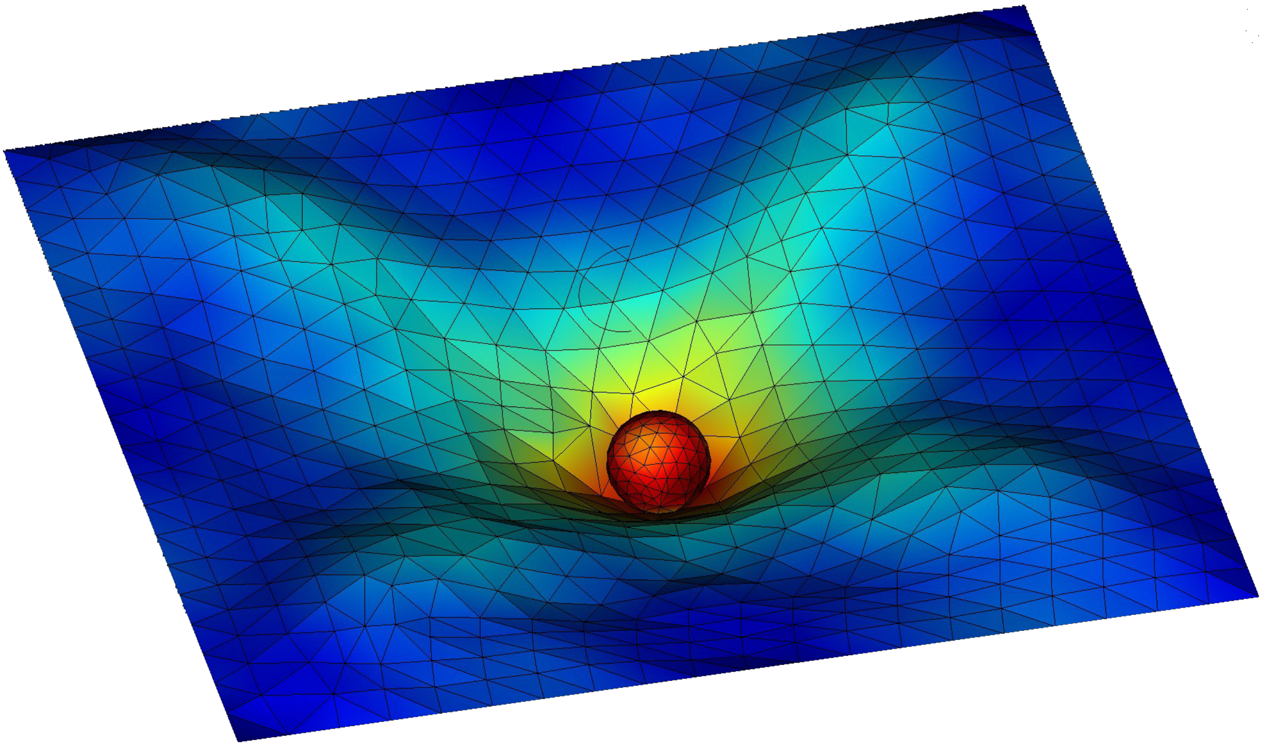

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PRELIMINARY ASSESSMENT OF SELF-CENTERING ROCKING STEEL SYSTEMS FOR LIGHTWEIGHT BUILDINGS

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Abstract

Building resilience is a growing priority in modern structural engineering, prompting the development of low-damage solutions like rocking systems. Self-centering rocking systems offer significant advantages by minimizing residual deformations and structural damage, ensuring rapid post-event functionality. This paper investigates re-centering rocking systems for lightweight steel structures, applicable to both new constructions and seismic retrofitting of existing buildings. The focus is on in-plane analysis of perimeter frames using controlled rocking mechanisms, enhanced with post-tensioned tendons or bars and energy dissipators such as friction or yielding devices. This preliminary study considers the possible applicability of this concept to a new lightweight 3 floors steel building by lumping the rocking behaviour at the first building level. Nonlinear time history analyses at various intensity levels are carried out to highlight the performance of the system. While initial findings highlight the potential of this structural solution to enhance building seismic performance, further research is required to optimize its implementation in practice.

Keywords: Rocking Systems, Light-Weight Steel Structures, Time-History Analyses.

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1 INTRODUCTION

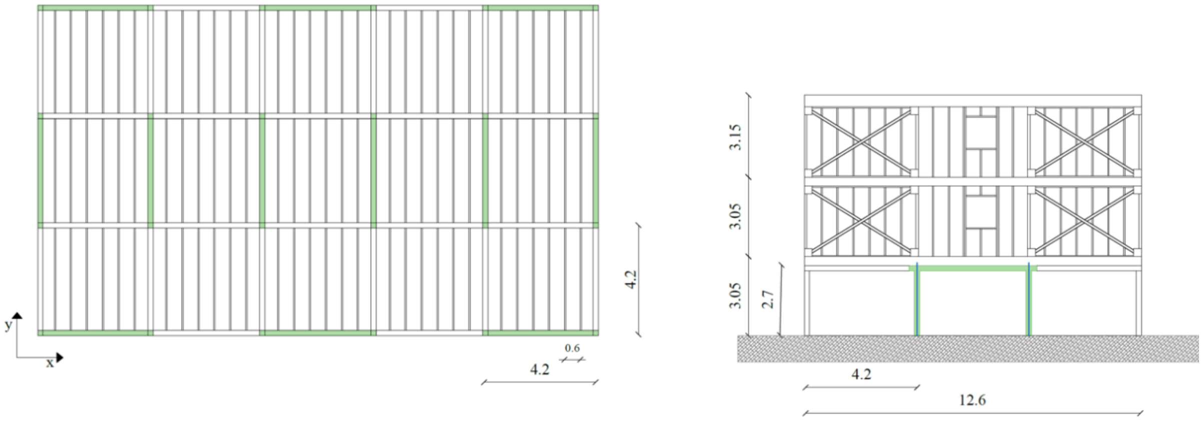
Current building codes require structures to perform across various limit states, balancing safety and damage control. Conventional design approaches aim to dissipate seismic energy through controlled damage in ductile elements, leading to costly repairs or demolition. This challenges post-earthquake recovery and increases losses. In recent years, building resilience to natural hazards has become essential. Resilient structures should withstand major earthquakes with minimal damage, no permanent deformation, and quick recovery, reducing economic losses and environmental impacts.

Self-centering rocking systems offer a promising solution by allowing rotations at specific sections while post-tensioned (PT) tendons restore the original position, minimizing residual drift. Additional hysteretic or viscous devices can improve energy dissipation, resulting in flag-shaped hysteretic behaviour. Over the last four decades, innovative self-centering solutions have been developed both for new buildings and retrofitting. For reinforced concrete (RC) buildings, rocking walls have been proposed [1, 2, 3, 4, 5, 6, 7], in which precast or cast-in-place wall panels are placed on the foundation without continuous longitudinal reinforcement at the interface between the wall and the foundation. Steel solutions include moment-resisting frames (MRFs) with PT strands [8, 9], controlled rocking steel braced frames (CRSBFs) with energy dissipation (ED) devices [10, 11], or self-centering steel plate shear walls (SPSWs) with unstiffened web plates [12, 13, 14]. Another solution for both steel and RC buildings is rocking podium structures, where the first-level columns rock and re-center during earthquakes, isolating the superstructure above. First implemented in Russia in the early 1980s [15], these structures demonstrated optimal performance in multiple earthquakes. Recently, the attention to these systems has grown, with several studies on their dynamic behaviour, design, and application [16, 17, 18].

In this context, this study investigates an innovative rocking platform for lightweight steel (LWS) buildings, suitable both for new construction and seismic retrofitting. LWS structures are gaining recognition for efficiency and sustainability, as reflected also in the upcoming Eurocode 8 [19]. A finite element model of the proposed system was developed. Nonlinear static, cyclic, and time-history analyses were performed assessing its seismic performance both considering and neglecting second order effects, to evaluate their influence on the global seismic response of the system.

2 CONSIDERED ROCKING SYSTEM

The analysis of the proposed rocking system begins with the definition of a three-storey residential building of lightweight steel construction as a reference case, in which all displacements caused by the earthquake are concentrated on the first level, where the rocking frames are located. The second and third levels are designed as a platform frame system in stud construction. The load-bearing floor system consists of C-shaped joists (250mmx50mmx20mmx2.5mm) spaced 0.6 m apart and 22 mm thick cross-laminated timber (CLT) panels, which also form the horizontal bracing system of the floor diaphragm. 2.7 m high load-bearing walls consist of vertical stud-connected columns (100mmx50mmx20mmx1mm), which are aligned with the floor joists. The columns are attached at each end to C-shaped horizontal wall rails (100mmx40mmx1mm). The system for absorbing the lateral forces consists of diagonal steel strips (100mmx1mm) arranged in an X-shape. The floor plan and elevations of the building are shown in Figure 1. The permanent loads are 0.86 kN/m^2 and 0.70 kN/m^2 for the intermediate and roof floors, respectively. The live loads were assumed to be 2 kN/m^2, as required by the Italian building code (NTC18) [20].



a) b)

Figure 1: Building configuration: a) plan, b) and elevation in y-direction (unit: m).

The first level is equipped with six self-centering rocking frames in each direction, depicted in Figure 2, consisting of S235 hot-rolled (HR) hollow-square columns and double-C beams (f_y=235 N/mm^2, f_u=360 N/mm^2, E=210000 N/mm^2), which provide the required displacement capacity. Re-centering is provided by 26.5 mm–diameter DYWIDAG threaded rods (f_y=950 N/mm^2, f_u=1050 N/mm^2) pre-stressed to 60% of the yielding resistance.

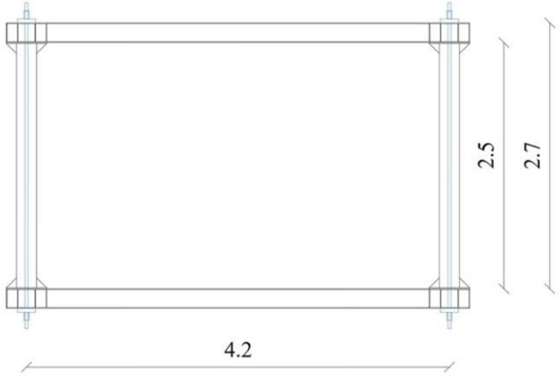
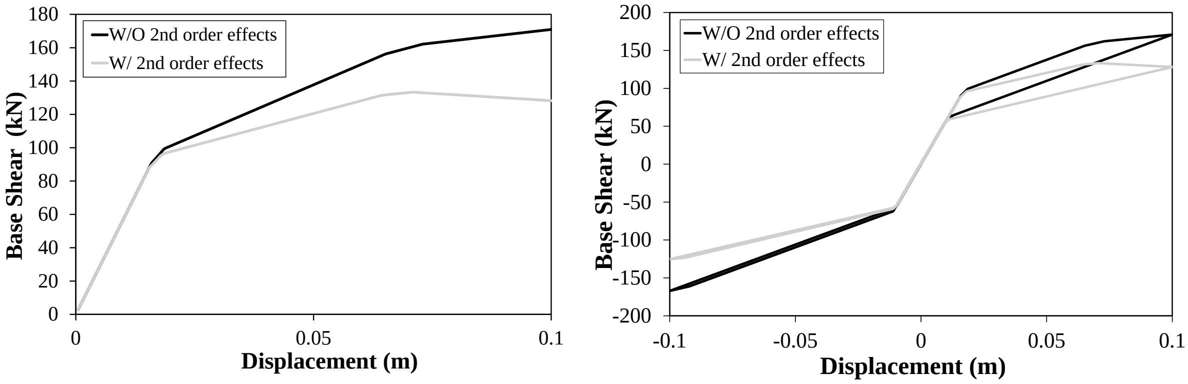


Figure 2: Proposed re-centering rocking system (unit: m).

The finite element model of the rocking frame was created using the Midas GEN software [21]. Beams and columns were implemented as elastic beam elements. Prestressed members were modelled as truss elements connected to the beams via rigid connections and hinged to the ground. The prestressing was simulated by forcing a displacement at its supports. The non-linear behaviour of the rods was implemented by axial, plastic hinges with trilinear behaviour. The interface between columns and beams was modelled by rigid elements connected at both ends with vertical axial compression-only springs with a stiffness of k= 1.68x10^{11} kN/m.

Nonlinear static and cyclic analyses were carried out to evaluate the performance of the system. Two consecutive load steps were created: in the first load step the gravity and post-tensioning are applied, in the second load step the horizontal force is applied to the structure. Two types of analyses were performed, either considering or neglecting the 2nd order effects, as shown in Figure 3. The 2nd order effects were analytically calculated and included in the model by accurately calibrating the inelastic properties of the tendons, which are characterised by a trilinear behaviour with a softening branch. As expected, the system exhibits an initial elastic branch up to the opening of the gap. Moreover, these preliminary analyses already show that the resistance of the system decreases when the 2nd order effects are applied.



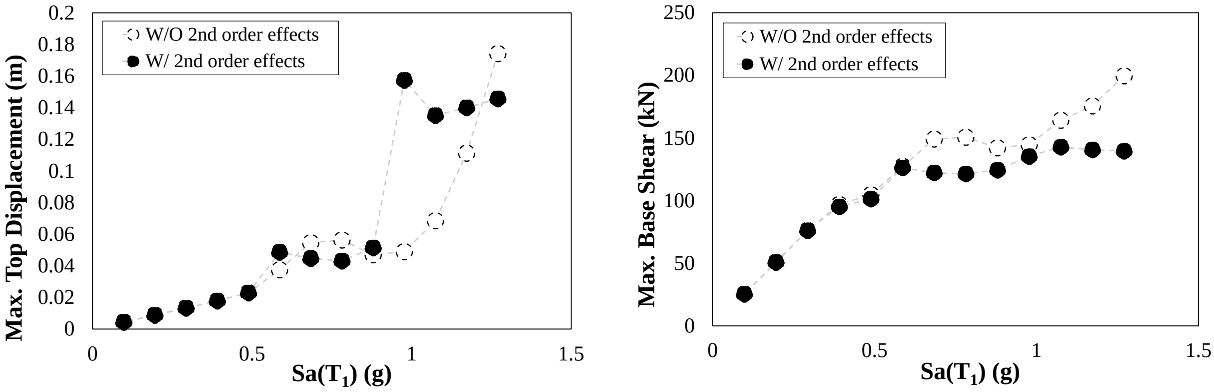
a) b)

Figure 3: a) Monotonic and b) cyclic behaviour of the system with and without 2nd order effects.

3 PRELIMINARY ASSESSMENT OF THE PROPOSED SYSTEM

To further evaluate the performance of the steel rocking system, nonlinear time history analyses were performed. A lumped mass was introduced at the centre of the upper beam and Rayleigh damping proportional to mass and stiffness was considered. A sequence of ground motions was applied to the structure to evaluate its behaviour under different seismic intensity levels. The building was assumed to be located in L'Aquila (AQ), an Italian city in a zone of high seismicity, on a site with soil type C and topographic amplification factor T_1. The input was derived from a spectrum-compatible accelerogram according to the provisions of NTC18. This data set was scaled while maintaining its shape to obtain different levels of spectral horizontal accelerations $Sa(T_1)$.

The results of the analyses have shown that at $Sa(T_1)=1.27g$, if the 2nd order effects are not taken into account, the post-tensioned rods break and thus the maximum load-bearing capacity of the structure is reached. In this case, the yield strength of the bars was reached at $Sa(T_1)=1.07g$. When 2nd order effects were considered, however, it was found that the PT bars broke at $Sa(T_1)=0.97g$. The plots in Figure 4 show that the behaviour of the structure with and without considering 2nd order effects is quite similar up to $Sa(T_1)=0.49g$ and that the capacity of the structure is lower when 2nd order effects are considered. Moreover, the acceleration in the structure would not increase once the rod failure is reached.



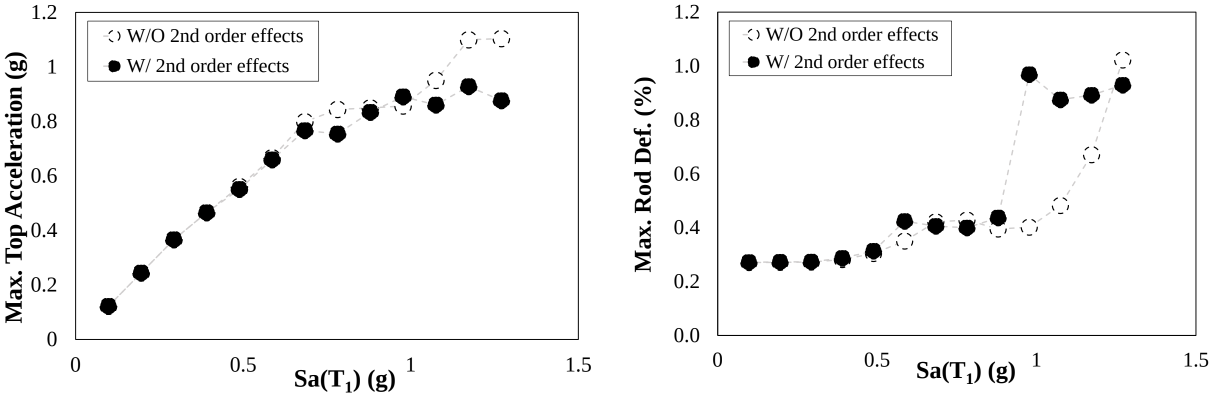


Figure 4: Time-history results (maximum) when second order effects are considered or neglected: displacement ate the top, base shear, acceleration at the top and deformation of the PT rods.

The base shear-displacement curves corresponding to the four horizontal acceleration levels are shown in Figure 5. These preliminary results confirm the reliable re-centering capability of the system, which effectively limits the residual displacements. As can also be seen in Figure 4, the system behaves quite similarly up to $Sa(T_1)$=0.49g, regardless of whether 2nd order effects are present. The upper displacement is generally higher when 2nd order effects are neglected, except in the range between $Sa(T_1)$ = 0.97g and 1.17g. In this interval, the PT rods have already failed when 2nd order effects are considered, while they remain intact when these effects are not considered.

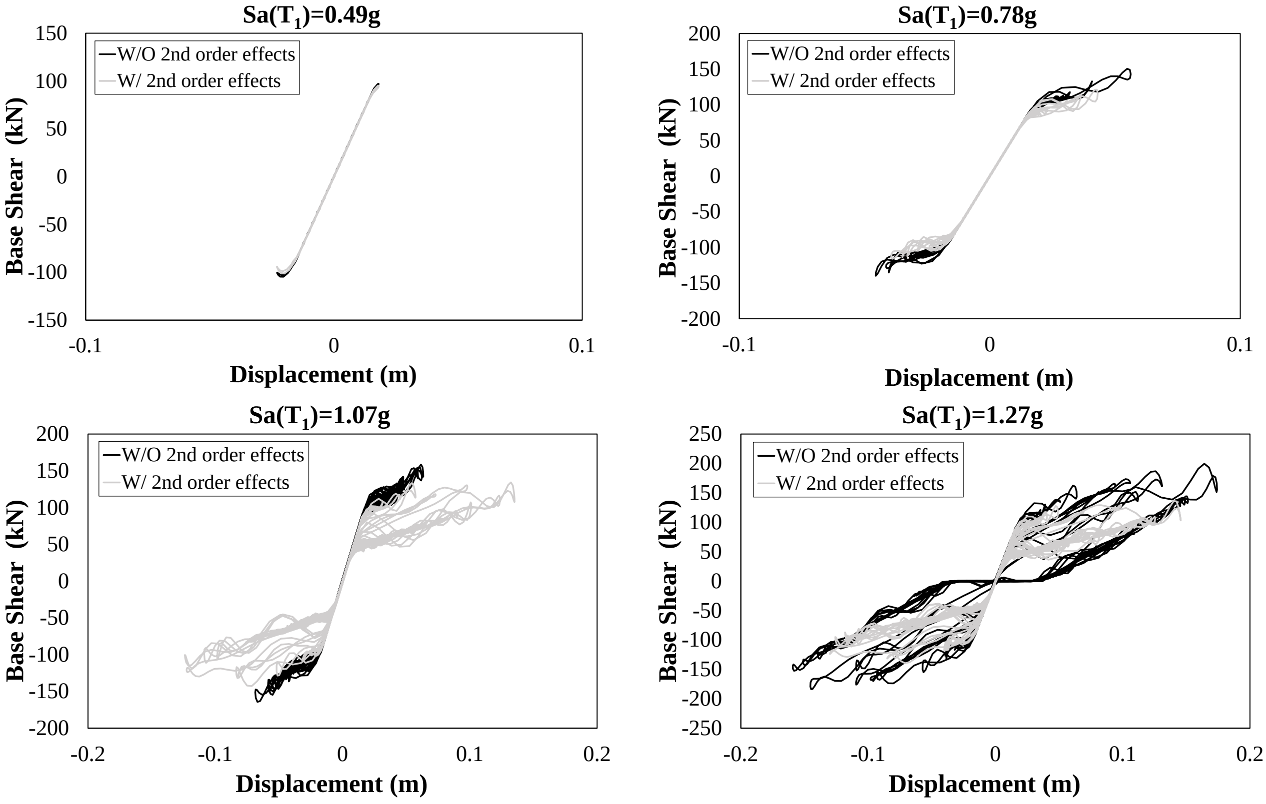
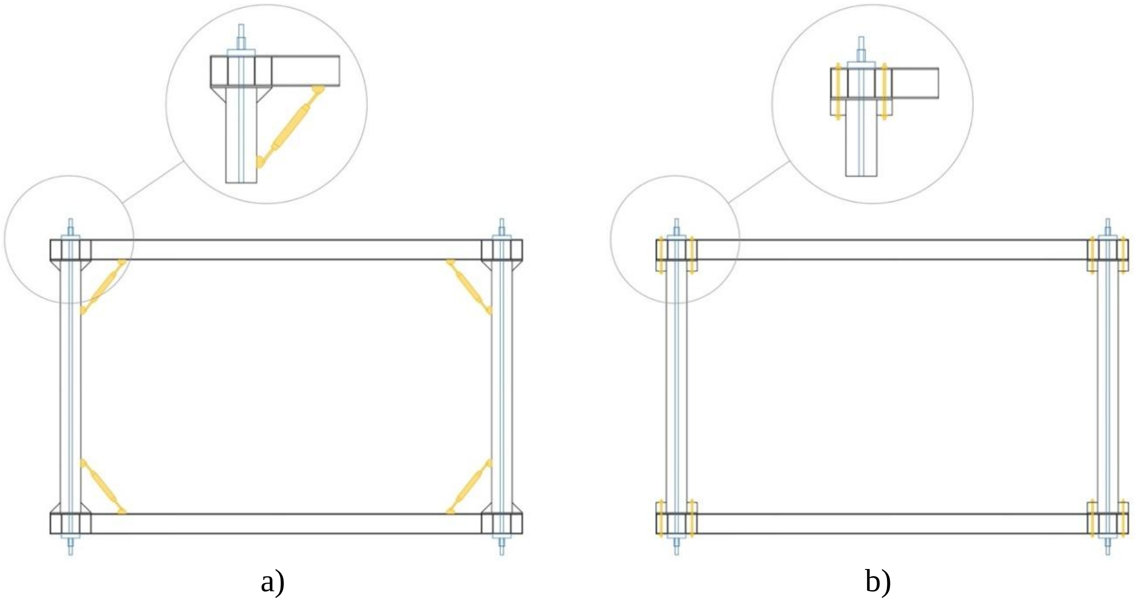


Figure 5: Time-history base shear-displacement results at different $Sa(T_1)$, comparing cases with and without 2nd order effects.

The diagrams in Figure 5 clearly show that the energy dissipation capacity of the system is quite limited as it appears solely on the yielding of the post-tensioned rods, thus jeopardizing the self-centering capability. Therefore, the introduction of energy dissipation devices such as hysteretic or viscous dampers is crucial to improve the performance of the system. These devices could be placed obliquely in the corners (Figure 6a) or at the rocking interfaces (Figure 6b).



a) b)

Figure 6: Rocking frame with energy dissipation devices in two different configurations: a) in the corners, b) at the rocking interfaces.

4 CONCLUSIONS

This study presents a preliminary proof of concept for an innovative self-centering rocking steel frame system for lightweight steel buildings, which can also be used for seismic retrofitting of existing reinforced concrete buildings. The proposed solution focuses the rocking mechanism on the first level and aims to isolate the upper structure and minimise the structural and non-structural damage during seismic events. A finite element model of the proposed rocking frame was developed and nonlinear static and time-history analyses were performed to evaluate the response of the system. The preliminary results have confirmed that the system is able to restore centering and thus improve functionality after an earthquake. However, it was found that the energy dissipation capacity is limited as it is associated with the yielding of the post-tensioned bars. The introduction of additional energy dissipation devices, such as hysteretic or viscous dampers, is considered essential to improve the overall performance of the system.

The analyses also revealed the significant impact of second order effects on the resilience of the system. When these effects were taken into account, a reduction in strength and earlier failure of the post-tensioned tendons were observed, emphasising the importance of considering these effects in the analyses. As this study is an initial proof of concept, further research and development work is required to optimise the design of the proposed system and deepen the understanding of its seismic performance. Future work will focus on the integration of suitable energy dissipation devices as well as the investigation of alternative structural configurations and the development of a design procedure.

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