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APPLICATION OF SEISMIC BASE ISOLATION IN A TIMBER MULTI-STORY FRAME BUILDING

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ABSTRACT: With the aim of continuing the progress being made in using wood as a structural material, this paper explores the application of seismic protection technologies to multi-story timber buildings, with a particular focus on base isolation. It examines the challenges and issues that can arise in practical design. The current literature does not provide in-depth coverage on this topic, which limits the confidence in using isolation techniques for wooden structures. This research looks into a practical case study of an eight-story residential condominium exploiting a hybrid timber-steel frame with specifically developed nodes connecting steel columns and glulam beams. After accurately modeling the structure, both linear and nonlinear analysis are performed. Four different structural conditions are analyzed to assess the effects of nodes and infill wall stiffness and to evaluate the effectiveness of the isolation system.

KEYWORDS: timber, multi-story, seismic-isolation.

1 - INTRODUCTION

In the past decade, there has been a significant increase in constructing multi-story buildings with wooden load-bearing structures. Given its sustainability (production process, disassembly and recycling), high prefabrication capacity (speed and simplicity of execution), and the flexibility (possibility of on-site modifications/cuts), timber represents an important material for the construction of structures. In particular, timber plays a key role in the design of structures in highly seismic areas. As a matter of fact, it is characterized by lightness (low mass), strength (good resistance characteristics, especially to instantaneous type loads) and stiffness. However, it is also a brittle material, especially for tensile stresses. This is amplified if the presence of defects is considered. Despite the low post-elastic capacity of wood, structures made of this material are able to achieve high ductility indices. The ductile behavior of a wooden structure is ensured by the system of joints that connect the elements to each other or to the foundation. The global deformability mechanisms of structures mainly involve the connections, which, since they are generally made of steel elements, allow high reserves of capacity. The plastic capacities of a construction are thus concentrated at the level of the connections, which play a key role also in its dynamic behavior, especially in multi-story structures.

These properties of the connections have been extensively studied and are now widely exploited for Cross Laminated Timber (CLT) wall multi-story structures. For timber frame structures, instead, these properties present more difficulties in their application. In fact, in frame systems, stability to lateral actions is ensured if beam-to-column connections are made capable of transferring bending moment from one element to the other. Since the timber elements are classified as brittle components, it seems necessary for the joints themselves to command the failure of the system. In addition, the beam-to-column node, generally made with cylindrical shank connectors, plays a key role in the lateral deformability of the structure. Indeed, the difficulty of making mechanical connections between wooden members to which infinite stiffness can be associated should be considered. There are several methods of making bending moment resisting joints. Two categories can be identified: bonded joints, which can generally be considered rigid, but are not suitable for predominantly seismic (dissipative) design because of their brittleness; mechanical connector joints, which can, instead, exploit the plastic capabilities of steel. The latter, however, due to steel-wood interaction phenomena such as slips and local deformations, are unable to offer infinite stiffness.

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Hence, the structure has such points of low stiffness that contribute to assigning the structure high lateral deformability. This is one of the reasons why multi-story frame technology is not widely used in seismic areas. Possible alternative design procedures applied to such structures are discussed in this study.

In his work, Symans [1] lists several events in which extensive damage to wooden structures led to great economic loss and loss of life. Over the years, the scientific community has researched new techniques for the construction of wood structures, especially for multi-story buildings in areas of high seismicity.

Besides connection systems that optimize the use of steel material (Trutalli et. al. [1]), innovative technologies also include anti-seismic systems known as SPT or 'Seismic Protection Technologies'. In fact, these systems allow to design by reducing seismic action instead of increasing the strength and stiffness of the structure. According to Ugalde [1], three typologies of SPT in timber structures can be identified: increasing of energy dissipation; applying seismic isolation systems; exploiting the "rocking" mechanism (in timber walls structures).

In particular, this study focuses on seismic isolation technology. The reduction of seismic action on the superstructure is associated with a number of positive aspects in terms of structural safety. Among these, the most important ones are: the reduction or elimination of damage to connections, which avoids temporary incapacitation of the structure and provides economic benefits for post-event seismic rehabilitation; the reduction in the number of connections (speed of construction) and wood sections (material savings); the lack of necessity to comply with the hierarchy of strengths, which for wood structures can be a major obstacle. Other advantages are the reduction of drift (i.e., less displacement between floors, which is especially problematic in frame structures with semi-rigid nodes), the reduction of damages to the contents of the building (people and things) and the maintenance of the operability of the building. Despite the many positive aspects, the use of these new technologies for wooden structures is still not widespread. Two reasons behind this are the high cost of the devices and their implementation in the structure. In addition, the scarcity of studies in the literature may affect the applicability of these technologies to timber structures.

The present study aims, therefore, to shed light on the application of isolation systems on timber structures, by analyzing the case study of a timber-steel frame structure seismically isolated at the base.

The conducted analyses provide insights into the behavior of the isolated structure under consideration, its problems and strengths. Linear static analysis, linear dynamic analysis with response spectrum, and nonlinear time-history analysis, which yielded mutually consistent results, were implemented on four versions of the building. Specifically, the analysis was initially conducted on the bare structure, neglecting the effect of the infills, and then extended to the infilled structure. The two configurations just mentioned were in turn analyzed under two different conditions: frame with rigid beam-column nodes and frame with semi-rigid beam-column nodes. The rotational stiffness values were calculated by applying the finite element method to the joint under consideration and comparing the related results with those obtained by the component method according to Eurocode 3 [4] (EC3).

2 - BACKGROUND

In the literature, there are only a few studies which investigated the applicability and efficiency of seismic devices for wooden structures. Ugalde [3] presented a review regarding the application of seismic protection systems on wooden buildings. Some case studies showed that it is possible to reduce the seismic impact on the superstructure by up to 90%, with good results in terms of inter-story displacement. However, there is still the need to find less expensive yet equally effective devices to ensure greater economic efficiency.

One example of the application of seismic isolation on timber structures is the post-earthquake reconstruction project developed in Italy in 2009, namely the 'C.A.S.E (Complessi Antisismici Sostenibili ed Ecocompatibili) project.' The need to deliver housing to the evacuees in a short time and remaining near the areas most affected by the earthquake, led to the development of a solution with base isolation at the urban level. Specifically, two plates (very large in plan) of reinforced concrete overlapping and separated by rigid pillars and an insulation system were conceived. The overlying plate serves as the support for the subsequent three-story housing structure built with technologies that in most cases involve wooden load-bearing structures. Of the two plates, the lower one is in contact with the ground, while the upper one rests on the pillars and insulation system and supports the loads arising from the structure. The plates were sized without knowing precisely the mass and loads involved. Having set a period of oscillation of the isolated structure equal to 4 s, thanks to the FPS (friction pendulum) isolation devices that allow a certain period of isolation regardless of the mass carried, it was possible to achieve the target period without knowing the masses exactly.

3 – PROJECT DESCRIPTION

Description of the structure under analysis

The project is based on the analysis of a practical case of an eight-story residential block building to be realized in Italy. Due to its current state of deterioration, the replacement of an existing concrete and masonry building with a new timber structure is proposed. The intervention is subject to several constraints, such as maintaining the pre-existing shape and dimensions and the number of housing units. A structure with a rapid construction process is preferable to allow for a fast re-entry of the owners of the housing units. For this purpose, the project includes solutions for fast assembling of members, precast timber facades and the implementation of a seismic isolation system. The latter allow to reduce design forces on nodes and expected damage for low-level earthquakes. The substructure is made of reinforced concrete, while the superstructure is a wood-steel frame with tubular steel columns and glulam beams. The floor consists of CLT panels resting on the beam grid. The stairwell and elevator shaft are built with continuous CLT walls from the ground to the top. CLT walls are also used for the prefabricated infill panels. The greater lightness of timber structures, given a certain objective period of oscillation, implies the use of devices with lower stiffness, with, however, a higher risk of falling into stability problems. Therefore, the FPS devices, previously utilized in the CASE project, are proposed in the present design. Hence, the isolation system includes 29 double curved surface sliding devices with a 5% friction coefficient and a curvature radius of 4 meters. The latter was designed taking into account the distance with obstacles in the vicinity of the building itself. In particular, the building borders, on the north side, another building at a minimum distance of 30 cm. Consequently, the maximum total displacement allowed to the structure in the dangerous direction, is about 20 cm.

The beam-column joints are made by means of HEA440 steel stub connected with bolts to the columns and to which web glulam beam heads (width 16+14 cm) are flanked and connected left and right. The connection with the wooden beams is made with full-thread screws inserted at the top and bottom of the HEA profile (Fig. 1). Epoxy adhesive is inserted at the interface between beam and log in order to limit sliding between wood and steel.

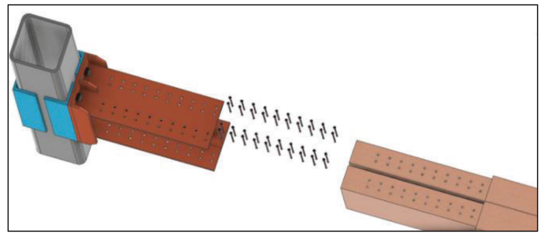

Figure 1. Steel column to timber beam joint technology

The seismic parameters of the site and structure under consideration, at the Life saving Limit State (SLV), are given in Table 1.

Table 1: Seismic design characteristics

Parameter	**Value**	
Nominal life of the structure V_N	50	years
Return period for SLV $T_{R,SLV}$	475	years
Peak ground acceleration (SLV) a_g	0.133	g
Max. spectral amplification factor F_0	2.528	-
Modified characteristic period T_c*	0.369	s
Site amplification factor S	1.800	-

Design criteria of the seismically isolated structure:

The primary objective of introducing the isolation system into the structure is to avoid irreversible damage to structural elements during seismic events. Therefore, the design criteria were:

a) Maintain an appropriately high isolation period while limiting the maximum building displacement, according to boundary constraints;
b) Non-dissipative behavior of the superstructure (elements and nodes respond by remaining in the elastic field);
c) Substructure as infinitely rigid and superstructure moving almost rigidly on the plane of isolation;
d) Achievement of an isolated oscillation period T_{is} at least twice the period of the same structure if it were without isolation T_{fb};
e) Avoid tensile stresses in the devices.

In order to meet these criteria, it is possible to act on the characteristics of the isolation devices, the stiffness (beam-column nodes and infill panels) and the mass of the superstructure. By carrying out an iterative analysis and design procedure, the best solution can be identified. Particular attention is paid to the distribution and stiffness of infill panels, as well as to the stiffness of beam-column joints.

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As said before, there is the need to build a new structure that is characterized by high speed of construction, in order to return housing to the displaced people in a short time. In particular, this structure is made of steel-wood, with construction details characterized by high prefabricability, which enhance simplicity and speed of assembly. Examples of such details are the choices on the beam-column joint system, CLT flooring (easily placed on the timber beam mesh) and infill panels, which should arrive at the site already equipped with joint systems to be lifted and placed on the façade.

The high lateral deformability of the superstructure is counteracted by the aforementioned infill system. These wooden wall elements are not structurally continuous from the ground to the top, as they are interrupted at each floor and connected to the beams using slot joints (vertical in direction). As a result, they do not transfer vertical loads and do not contribute to carrying gravitational or seismic forces to the ground.

Tensile loading problems occurred in the isolation devices at the elevator shaft opening of the base slab. It was observed that these CLT panels absorb large amounts of horizontal seismic forces and, through an oscillation mechanism, cause the slab and isolation devices to lift. The proposed solution is to locally lower the base-isolated slab to the zero level at the elevator shaft, while maintaining its continuity. A single isolation device of greater capacity is inserted below the compartment and at in its center of gravity, so that it would not be affected by any rotation of the system.

4 – DESIGN PROCESS

Numerical analyses were carried out using the FE software Midas Gen. The superstructure was modeled using beam elements for columns and floor beams, as well as for the walls of the stairwell, elevator shaft, and infill panels (following the equivalent frame modeling principle) (Fig. 2, Fig. 3). Connection systems were represented by an assembly of elastic and rigid links with stiffness values assigned based on the type of connector. The correct evaluation of the dynamic behavior of the superstructure (e.g., in terms of oscillation period or interstory displacement) requires that the model correctly includes all deformability components that may develop due to a horizontal action.

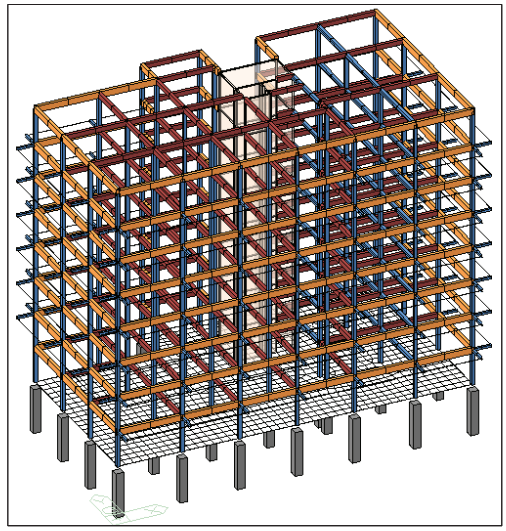

Figure 2. Numerical model of the eight-story frame building without infills (M1 and M2 models)

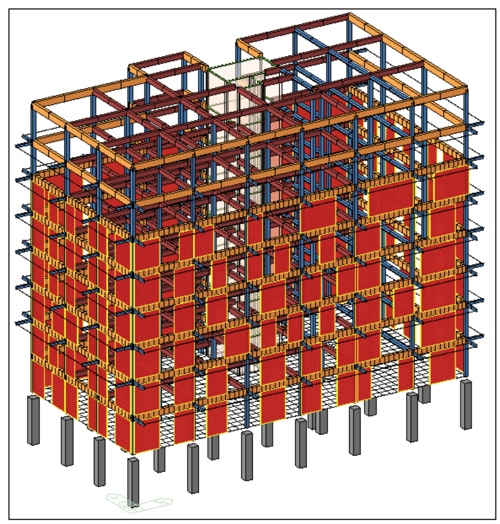

Figure 3. Numerical model of the eight-story frame building with infills (M3 and M4 models)

Starting with the joints of the CLT and infill panels, the stiffness value used was calculated using the simplified formulations given in Eurocode 5 [5] (EC5). An iterative procedure was required to properly design these elements and calculate their stiffness.

Whereas more accurate analyses regarding the beam-to-column connections that constitute the resisting frame system have been conducted. The presence of wood elements converging at the nodes represents a critical issue with regard to seismic design, both in terms of stiffness (whereby semi-rigid nodes lead to more deformable behavior overall) and in terms of strength (hierarchy of strength and ductility). Regarding the latter aspect, it should be pointed out that, in the present case,

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which involves the application of an isolation system, it is not necessary that a defined degree of ductility is achieved, as the structure is expected to remain in the elastic range for each level of seismic design stress. Joints between beams and columns are therefore designed with the aim of achieving high stiffness values, so as to counteract inter-story displacements and ensure greater efficiency of the isolation system.

As seen before, these connections feature an HEA440 stub connected to the column with pretensioned bolts. The connection between the stub and the beam's end was made with 4 rows of 10 screws inserted orthogonally to the shear surface. To prevent slipping between steel and wood, resin was applied at the interface between steel stub and timber. To calculate the rotational stiffness of the node to be inserted into the global model of the building, the two parts of the joint were considered separately: the steel-only portion (HEA440 stub bolted to the column); the wood portion, considering also the screws connecting the glulam beam and the steel stub.

Starting with the steel-only portion, the component method (in accordance with EC3) was first applied. Then, a plate/shell FE local model was used to conduct non-linear analyses, from which the cyclic bending moment-rotation diagram was derived point by point. The deformability components follow the scheme in Figure 4.

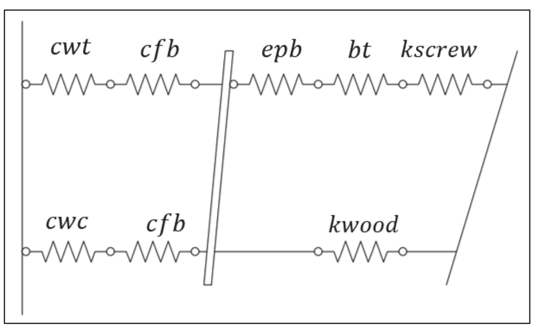

Figure 4. Scheme of the deformability components of the steel-timber joint

The steel-side deformability components considered in accordance with EC3 are: column web in compression cwc (2); column web in tension cwt (3); column flange in bending cfb (4); end plate in bending epb (5); bolts in tension bt (10). These components are calculated both analytically using the formulations given in EC3 and numerically using a local node model on FE analysis software. Whereas the wood-side components are: kscrew which considers the deformability of the screws in shear and kwood which, instead, considers the compression of the wood (this deformability is neglected).

The rotational stiffness of the joint is obtained by equation (1) below:

$$S_{j,ini} = E\, z^2 / (k_1+k_2+k_i+\ldots)^{-1} \qquad (1)$$

where z represents the lever arm of the internal forces, E is the elastic modulus of the material, and k_i is the stiffness of the deformability component as specified in EC3.

From the nonlinear analyses performed on the FE model of the connection (Fig. 5), the moment-rotation diagrams and thus the rotational stiffness value for the case of pretensioned bolts and non-pretensioned bolts were obtained (Fig. 6).

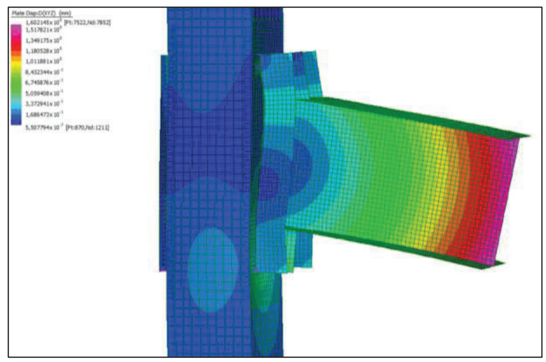

Figure 5. FE model of the steel part of the beam-to-column joint

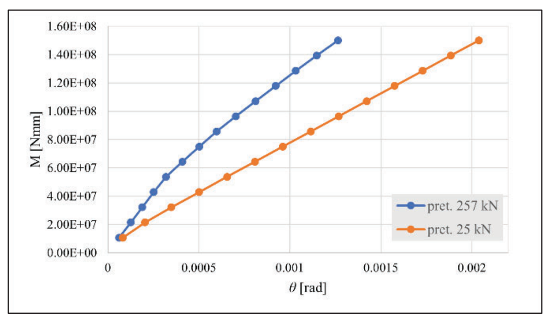

Figure 6. Moment-rotation curve of the steel part of the joint for pretensioned bolts (blue) and non-pretensioned bolts (orange)

Finally, the wood-steel deformability component was added. Since the stress level beyond which the resin becomes ineffective is unknown, two extreme scenarios are considered: resin active in which only the deformability of the steel portion is considered; resin not active with the deformability of the screws included.

The ratio between the rotational stiffness of the node and the flexural stiffness of the beam is then examined. In accordance with EC3, if this ratio is greater than 25 then the joint can be considered rigid, otherwise it should be considered semi-rigid. As can be seen in Table 2, for

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cases with active resin, the node can be considered rigid; whereas if the resin is inactive, the node behaves as semi-rigid.

Table 2: Stiffness ratio between joint and timber beam

Rotational joint to flexural beam stiffness ratio	Resin effective	Resin not effective
Component method	27	9
Numerical method	35	10

As for the global model of the structure, therefore, both cases of rigid and semi-rigid nodes were considered and then compared.

The numerical analyses on the global model included both linear dynamic analysis (spectral analysis) and nonlinear dynamic analysis (nonlinear time history analysis). For linear analyses, the isolation devices were represented as elastic springs (elastic links) between the substructure columns and the insulated concrete slab. These "elastic link" elements were assigned a vertical infinite stiffness and a horizontal stiffness equal to the linearized K_e equivalent stiffness (2) calculated by an iterative analysis procedure.

$$K_e = N_{Ed} (\mu/d + 1/R) \tag{2}$$

where N_{Ed} is the acting axial force on the device, μ is the nominal dynamic friction coefficient, d is the maximum horizontal displacement generated on the device and R is the equivalent curvature radius of the device.

As for nonlinear analyses, the nonlinear behavior of the isolation devices is introduced in the model, through the inclusion of "general links" elements that the software provides to represent the behavior of the isolation devices. In the present case, the "general links" have been assigned the properties of the friction pendulum isolators shown in Table 3.

Table 3: Characteristics of the seismic isolation devices

Parameter	Value	
Equivalent curvature radius R	4.0	m
Nominal dynamic friction coefficient μ	0.05	-
Horizontal displacement capacity (+/-) d_{Ed}	300	mm

The global analyses were conducted on four different structural models: a bare frame with semi-rigid joints (M1); a bare frame with rigid joints (M2); an infilled frame with semi-rigid joints (M3); and an infilled frame with rigid joints (M4). In this way, it was possible to analyze the contribution of both the stiffness of the joint and the infill system.

5 – RESULTS

Firstly, a comparison between the results of the analyses performed was made. A linear static analysis was also implemented. Clearly, the latter produced overestimated isolator displacements, since the analysis' procedure is based on a purely translational behavior of the structure that follows only the first mode of vibration. As can be seen from the modal analysis, the high deformability of the superstructure leads to modes of vibration above the first that are not negligible. All in all, the spectral analysis and the nonlinear time history analysis lead to rather comparable results for both displacements at the isolators and displacements at the top at the control points.

As expected, the superstructure exhibits significant lateral deformability, which negatively affects the period of oscillation associated with the activation of the isolation system. The ratio of the fundamental period of oscillation of the isolated structure to that of the fixed-base structure, did not exceed a value of 2 in any of the four cases, which is well below the recommended value of 3 (Tab. 4). Among the four configurations, the rigid node structure with infill system is the one that best provides isolated period displacement compared to that of the fixed-base structure.

Table 4: Numerical results for the four analyzed models

Results by model	T_{is}/T_{fb}	Max. disp. on the devices [cm]	Max. disp. on the building top [cm]
M1	1.35	9.10	24.10
M2	1.52	9.90	22.70
M3	1.56	9.70	22.65
M4	1.72	10.20	19.40

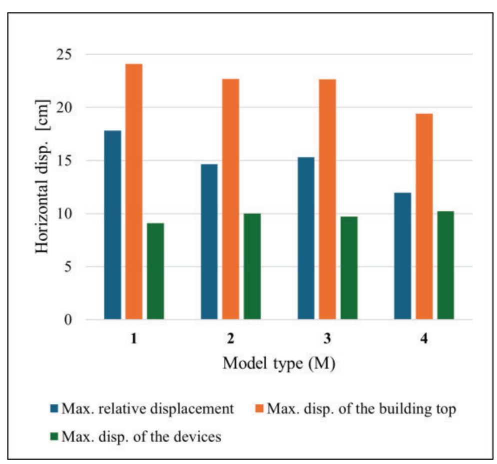

Figure 7. Comparison of horizontal displacements from numerical analysis between model configurations

Considering the maximum displacements of the seismic devices, the M1 model allowed slightly smaller displacements compared to the other three (Tab. 4 and Fig. 7). However, the most significant differences between the four configurations were evident in the superstructure's response. In particular, the time-displacements diagram of floors points along a vertical line of the building was analyzed. With respect to the others, the M4 model showed a higher degree of use of the isolators and smaller ratios between the displacements of the isolators and the relative displacements of the superstructure. Unlike model M4 (Fig. 9), model M1 (Fig. 8) exhibits effective activation of the isolation system for only a few instants of the seismic event. Moreover, when the system is activated the most (instants 5.0 and 8.7 s), the displacements of the superstructure are comparable, in magnitude, to the displacements of the isolators.

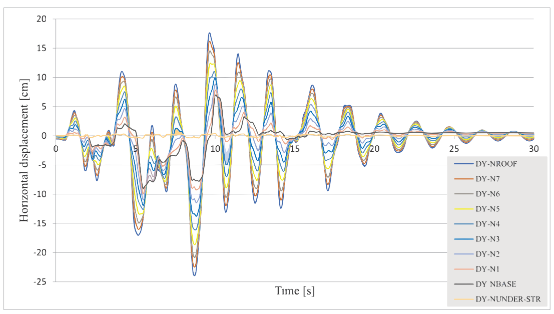

Figure 8. Horizontal displacements-time diagram for all the building levels (M1 model)

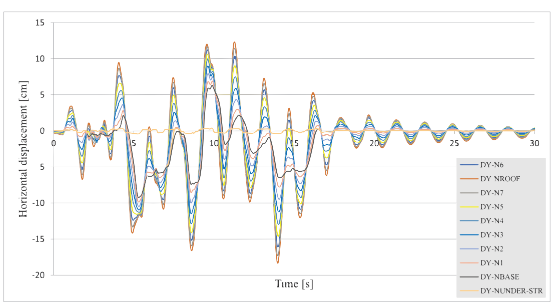

Figure 9. Horizontal displacements-time diagram for all the building levels (M4 model)

By comparing the floor displacements of M1 and M2 models at a specific time-step (Fig. 10), the impact of joint stiffness can be observed. The M2 model (bare frame with rigid joints) exhibited greater displacements at the isolators and smaller total top displacements compared to the M1 model (bare frame with semi-rigid joints). Similar results can be noted when comparing the total floor displacements of the M1 and M3 models (bare and infilled models). Thus, overall, joint stiffness and the presence of infills provide equal contribution.

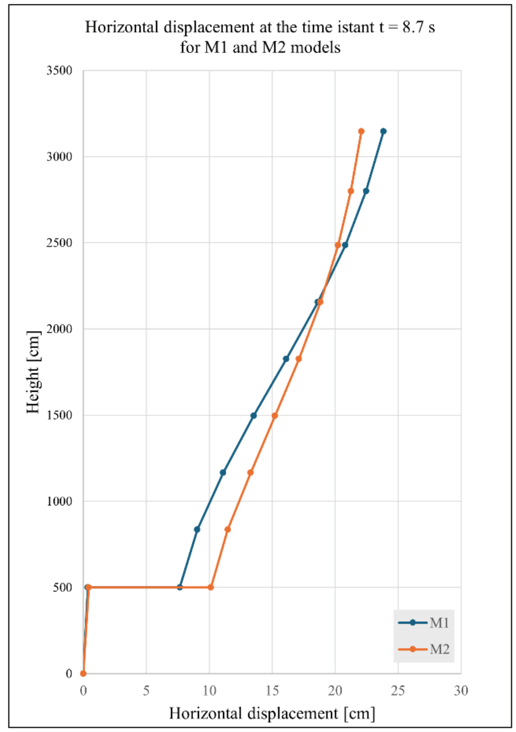

Figure 10. Horizontal displacements at the time instant t = 8.7 s for M1 (bare and semi-rigidi node model) and M2 (bare and rigid node model) models

To conclude, comparing the maximum displacements of the isolation devices, relative to the superstructure, and the maximum total displacements at the top, it was observed that model M4 achieves a lower total displacement, despite being the one that makes most use of the isolation devices.

6 – CONCLUSIONS

This study highlights the benefits and challenges of applying seismic base isolation to a multi-story timber frame structure. The high deformability of the superstructure limits the achievement of the minimum recommended value of 3 for the ratio of isolated system and base-fixed superstructure oscillation period. The stiffness of beam-column nodes and the presence of infills play a key role in increasing the effectiveness of the insulation system.

In addition, another issue worth mentioning is the increased risk of activation of the system by horizontal static actions, such as from wind. The lightness of the structure can ensure low values of horizontal frictional limit force that are not suitable to avoid activation of the

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isolation system by static forces. Moreover, it is relevant to underline that the risk of tensile stresses in the devices is likely to occur. Such tensile stresses can be caused locally by the rotations of rigid walls (CLT walls) or globally by the rotation of particularly tall superstructures.

Anyhow, it is worth exploring the application of the base isolation to timber construction since it could profitably reduce seismic actions and make timber construction more affordable, especially if applied to CLT construction where higher values of lateral stiffness can be easily obtained.

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