

NONLINEAR RESPONSE ANALYSES OF A 2-STOREY RC BUILDING WITH FIBRE-REINFORCED ELASTOMERIC ISOLATORS

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Abstract

Base isolation (BI) is one of the most promising techniques for seismic risk mitigation. However, the high cost of traditional isolators undermines the systematic implementation of BI systems in both new and existing buildings. In the last decade, fibre-reinforced elastomeric isolators (FREIs) have been developed as a promising low-cost alternative to steel reinforced elastomeric isolators. This study presents a numerical investigation on a three-dimensional, two-storey reinforced concrete (RC) frame designed for gravity loads only, which is representative of a recurrent situation in many existing European buildings even in earthquake-prone areas. The specimen was base isolated with innovative FREIs and was subjected to shaking table tests at LNEC, Lisbon, Portugal, within the ERIES-FREISUST research project. This paper presents the main outcomes of the numerical modelling of the building specimen. In a first stage, the building was analyzed through a pushover analysis in fixed-based configuration to identify the global seismic capacity, damage progression, and collapse mechanism. The influence of infill walls along one horizontal direction was also addressed by the model, showcasing its significant contribution to the lateral resistance. The FREI BI system was designed to achieve a target isolation period lower than 2 s. Assuming a Bouc-Wen hysteretic model of FREIs, which was calibrated based on experimental tests, the specimen response was investigated via nonlinear time history analysis (NLTHA). Based on the expected displacement capacity of the BI devices, both ordinary and pulse-like ground motion records were selected to be representative of medium-to-high seismicity regions and to finely tune the performance level of FREIs. Both horizontal components of ground motions were imposed to the model. NLTHA demonstrated adequate filtering effect of the isolation system with no damage to both structural and non-structural building components, confirming the experimental observations from the shaking table tests and the effectiveness of FREIs for applications to housing projects.

1 INTRODUCTION

Base isolation has been acknowledged as one of the most effective mitigation strategies against seismic action. During the last few years different isolation systems have been developed aiming at reducing costs to improve the number of applications in common housing projects. Fibre-reinforced elastomeric isolators (FREIs) represent one of the most promising solutions especially when adopting reclaimed compounds with plastic-type fabrics like polyester. In this regard the project FREISUST was recently funded within the ERIES call for 3D shaking table testing (STT) of a full-scale 2-storey reinforced concrete (RC) frame isolated with FREIs. The tests were performed on the 3D shake table (ST3D) facility on the National Laboratory for Civil Engineering (LNEC), Lisbon, Portugal.

This paper presents the nonlinear numerical simulation capable of faithfully reproducing the behavior of the building specimen subjected to testing. The numerical model was developed with the structural analysis software SeismoStruct v2024 [10] enabling both dynamic and static nonlinear analyses while combining mechanical and geometrical nonlinearities. The specimen was designed against gravity loads only, according to Eurocode 2 [1] and accounting for lifting operations in the laboratory. This approach aimed at providing the characteristics of common RC buildings designed without considering seismic provisions.

Once the numerical model was developed, nonlinear static analyses were performed to investigate the response and collapse mechanism of the fixed-base (FB) building, considering the contribution from infill walls along one of the two plan directions. In order to finely tune the ground motion selection for the STT, nonlinear time history analyses (NTHAs) were run for calibration of the scale factors (SF) to be adopted during the experimental testing. The calibration was performed in such a way as to attain the target displacement of the isolators, thereby limiting maximum acceleration, velocity and displacement values to payload of the ST3D facility.

2 DESCRIPTION OF THE BUILDING SPECIMEN

The prototype used in this project is a 2-storey reinforced concrete (RC) frame with a square plan of 4x4 meters and a total height of 6 meters. The interstorey height, measured from the centerline of the structural elements, is 2.70 m and 2.80 m on the second and first level respectively (Figure 1). These dimensions were chosen based on the constraints imposed by the LNEC, yet they still allow the specimen to be considered as full-scale.

An additional constraint set by the laboratory was related to the weight, which could not exceed 40 tons. This limitation was necessary to comply with the payload capacity of both the shaking table and the overhead crane used during the lifting and positioning of the specimen onto the ST3D platform. Both according to maximum weight of the specimen set equal to 40 tons and in order to test two infill systems, masonry infill walls were built in one direction only (X). In the other direction (Y), lighter plasterboard infills were installed with two openings allowing visual inspection inside the building during the experimental testing.

The RC elements were designed considering gravity loads only as suggested by Italian building code for base isolated structures NTC 2018 [5]. The dimensions of the structural elements are as follows whose reinforcement details are given in Figure 2:

- Elevation beams: 400x300 mm²;
- Columns: 350x350 mm²;
- Base beams: 600x350 mm²;

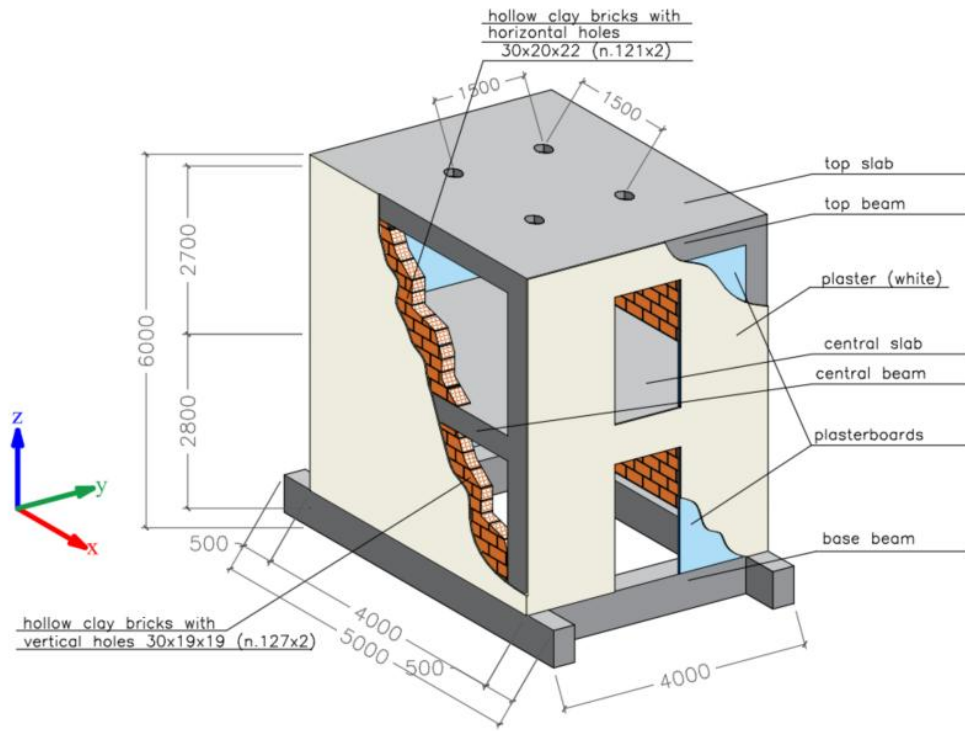


Figure 1: Drawing of the specimen to be tested on the ST3D facility at LNEC (dimensions in mm).

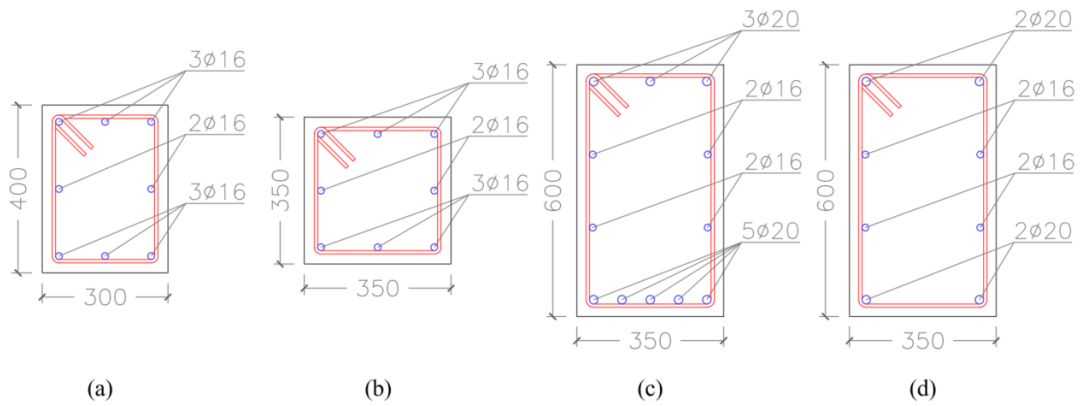


Figure 2: RC frame cross-sections: (a) elevation beams, (b) columns, (c) base beams, (d) extra-length base beams (dimensions in mm).

The design of the base beams was constrained by the need to prevent cracking during the lifting operations, since the building self weight was also increased by 50% to account for dynamic amplification factor. To enable lifting operations, the base beams along one direction were extended by an additional 500 mm beyond the building's footprint along X. Within such two beam extensions, four steel plates were embedded during casting. Each plate featured a hole at the top above the beam upper surface to allow connection with the overhead crane hooks.

The slabs consisted of 120 mm thick solid RC elements reinforced with a double layered welded wire mesh of 8 mm diameter at 150 mm spacing.

Masonry infill walls were built using hollow clay bricks with horizontal holes (30x19x19 cm³) used on the first storey and vertical holes (30x20x22 cm³) on the second storey. In the

other direction a plasterboard system was installed on both storeys to provide a more flexible infill system. On the external side the walls were then finished with a white plaster coat (Figure 3).



Figure 3: From left to right: construction of plasterboard infills, construction of brick infill walls; prototype completed in FB configuration.

Two different sets of FREIs were used: the first set included polyester fiber-reinforced isolators composed of 50% recycled rubber from used tires and leftovers (RP-FREIs); the second set consisted of virgin rubber isolators reinforced with carbon fibers (VC-FREIs). While RP-FREIs were adopted in unbonded configuration, VC-FREIs were installed with recess plates having two central holes of the bearing diameter fixed to the base of the shaking table and to the underside of the base beams, respectively (Figure 4 and Figure 5). Both VC-FREIs and RP-FREIs were installed without any mechanical connection.

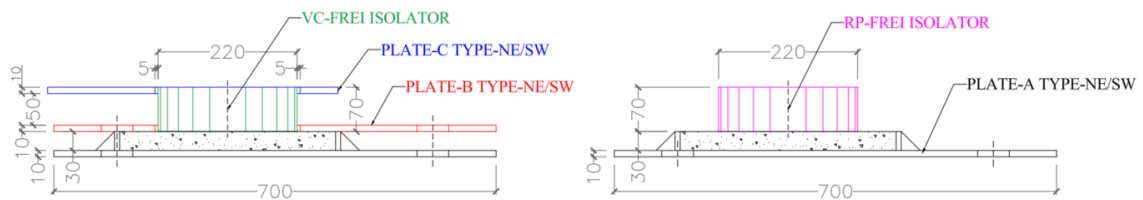


Figure 4: Drawings of the installation layout for VC-FREI (left) and RP-FREI (right).

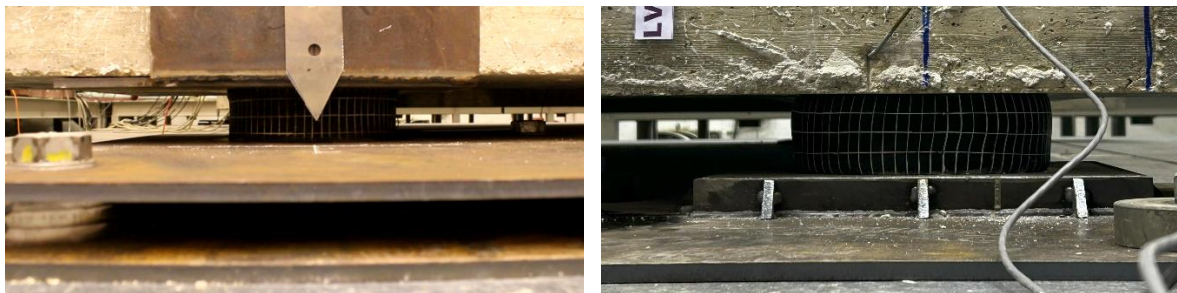


Figure 5: Photos of the installation layout for VC-FREI (left) and RP-FREI (right).

3 NUMERICAL MODELING

A rigorous approach was adopted by developing a nonlinear numerical model in SeismoStruct v2024 [10] accounting for nonlinear behavior of the RC, the masonry infills and the base isolators. For RC members the so-called “fiber” modeling technique was adopted, which allows for the discretization of the cross-section into a finite number of fibers as distributed plasticity approach. For masonry infill walls a macro model, using equivalent trusses and a central nonlinear link element to simulate the infill’s behavior, was considered, while a Bouc-Wen hysteretic relationship was implemented for FREIs.

3.1 Reinforced concrete

For concrete, the nonlinear model proposed by Mander et al. [6] (herein referred to as ‘con_ma’) was employed. This is a uniaxial nonlinear model with constant confinement: the effects of confinement provided by the transverse reinforcement are accounted for through a constant confining pressure throughout the entire stress–strain response.

The software requires the input of five calibration parameters to tailor the constitutive law of the material (Table 1):

- the cylindrical compressive strength f_c , obtained from uniaxial compression tests on 100x200 mm cylindrical specimens;
- the tensile strength f_t , which can be estimated from f_c using the following relation:

$$f_t = K_t \cdot \sqrt{f_c} \quad (1)$$

- the elastic modulus E_c , understood as the initial elastic stiffness of the material;
- the strain at peak compressive strength ϵ_c ;
- the specific weight γ of the material.

For the cylindrical compressive strength f_c , the characteristic value corresponding to the concrete class selected during the design phase (C30/37) was adopted. For the tensile strength f_t and the elastic modulus E_c , the characteristic and mean values, respectively, were used according to NTC 2018 [5]:

$$f_{ctk} = 0.7 \cdot f_{ctm} = 0.7 \cdot 0.3 \cdot \sqrt[2/3]{f_{ck}} \quad (2)$$

$$E_{cm} = 22000 \cdot \left(\frac{f_{cm}}{10}\right)^{0.3} = 22000 \cdot \left(\frac{f_{ck}+8}{10}\right)^{0.3} \quad (3)$$

The strain at peak compressive strength, ϵ_c , was assigned the default value suggested by the software, as was the case for the specific weight γ of the material.

f_c [MPa]	f_t [MPa]	E_c [MPa]	ϵ_c [-]	γ_c [kN/m ³]
30.00	2.03	32836.57	0.002	24.00

Table 1: Mechanical parameters adopted in the SeismoStruct material model “con_ma”.

For steel, a simple symmetric bilinear model (herein referred to as ‘stl_bl’) was adopted. This model consists of a linear elastic branch, followed by a strain-hardening plastic branch with constant slope. The software requires the input of five calibration parameters to tailor the constitutive law of the material (Table 2):

- the elastic modulus E_s , representing the initial elastic stiffness of the material;
- the yield stress f_y , marking the end of the elastic phase;
- the strain hardening parameter μ , defined as the ratio between the plastic stiffness and the initial elastic stiffness of the material;

- the ultimate strain ϵ_{ult} ;
- the specific weight γ of the material.

For the yield stress f_y , the characteristic value associated with the steel class selected during the design phase (B450C) was used. The hardening ratio μ was set equal to zero.

E_s [MPa]	f_y [MPa]	μ	ϵ_{ult} [-]	γ_s [kN/m ³]
$2.1 \cdot 10^5$	450.0	0.00	0.065	78.00

Table 2: Mechanical parameters adopted in the SeismoStruct material model “stl_bl”.

Stress-strain diagrams for both concrete and steel are reported in Figure 6.

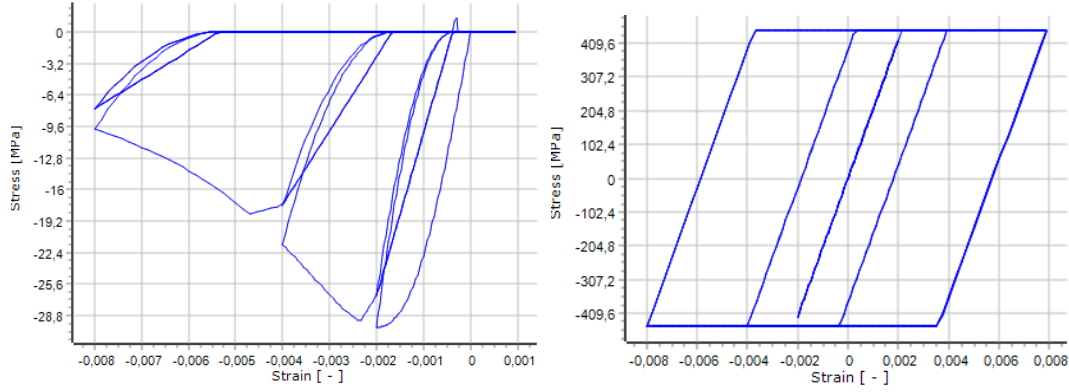


Figure 6: Stress-strain diagrams for concrete (left) and reinforcing steel (right).

3.2 Infill walls

Although infill walls are commonly classified as non-structural components, it is well known that they can significantly influence the structural response under lateral loads. Therefore, their contribution cannot be neglected in accurate modeling. A considerable number of studies have been conducted to characterize the behavior of infilled reinforced concrete frames under lateral loads, and numerous modeling approaches can be found in scientific literature. In the present study, the modeling approach proposed by Panagiotakos and Fardis [7] and later adapted by Varela et al. [11] was adopted.

This model belongs to the family of the so-called macro-models, which differ from the more refined micro-models, where individual bricks are modeled requiring a significantly higher number of parameters to be calibrated. Instead, this model requires a limited number of parameters that are relatively straightforward to determine. Macro-models employ a more practical approach by representing the masonry panel through equivalent diagonal struts, in accordance with the observation made by Polyakov [9], who noted that the stress flow within the infill panel tends to follow the diagonal direction.

Specifically, the model proposed by Panagiotakos and Fardis [7] and adapted by Varela et al. [11] considers the width of the equivalent strut, b_w , as the key parameter for estimating the stiffness and strength of the infill panel. This parameter is calculated using the following expression, also adopted in FEMA 306 [3]:

$$b_w = 0.175 \cdot d_w \cdot (\lambda \cdot h_c)^{-0.4} \quad (4)$$

where d_w represents the length of the panel's diagonal, h_c is the height of the column measured with respect to the centroidal axes of the beams, and the dimensionless parameter λ represents the relative stiffness between the infill panel and the reinforced concrete frame. The parameter λ considered for b_w is calculated as follows:

$$\lambda = \sqrt[4]{\frac{E_w \cdot t_w \cdot \sin(2 \cdot \theta)}{4 \cdot E_c \cdot I_c \cdot h_w}} \quad (5)$$

where E_w is the elastic modulus of the masonry infill panel, t_w is its thickness, $E_c I_c$ is the flexural stiffness of the column, θ is the angle of inclination of the diagonal with respect to the horizontal (Table 3).

Walls	h_w m	l_w m	t_w m	d_w m	θ rad	λ -	b_w m
Bricks w/ vertical holes	2.25	3.30	0.20	3.99	0.60	0.81	0.51
Bricks w/ horizontal holes	2.35	3.30	0.20	4.05	0.62	0.67	0.55

Table 3: Geometrical properties of the infill panel and of the equivalent strut.

To derive the mechanical parameters of this model, the assumptions proposed by De Risi et al. [4] have been considered:

- the elastic modulus of the masonry E_w is calculated in relation to the compressive strength f_w using the formula provided in FEMA 306 [3]:

$$E_w = 550 \cdot f_w \quad (6)$$

- the compressive strength f_w is determined from the compressive strengths of the mortar f_m and the masonry units f_b , using the Eurocode 6 [2] formula:

$$f_w = K \cdot f_b^{0,7} \cdot f_m^{0,3} \quad (7)$$

where K is a tabulated constant (EN 1996 § Table 3.3) [2] depending on the type of masonry.

- the shear modulus G_w is also derived from FEMA 306 [3] and is assumed to be 40% of E_w .
- the cracking shear strength τ_{cr} is calculated as:

$$\tau_{cr} = 0.275 \cdot \sqrt{f_w} \quad (8)$$

The parameters computed using the above formulas are summarized in Table 4.

Walls	K -	f_m N/mm ²	f_b N/mm ²	f_w N/mm ²	E_w N/mm ²	G_w N/mm ²	τ_{cr} N/mm ³
Bricks w/ vertical holes	0.45	5.00	3.00	1.57	865.5	346,2	0.34
Bricks w/ horizontal holes	0.35	5.00	1.50	0.75	414.4	165.7	0.24

Table 4: Mechanical properties for both horizontal and vertical holes bricks masonry.

The resulting force–displacement curve is trilinear (Figure 7) compared to the quadrilinear shape proposed by Panagiotakos and Fardis [7]. The adopted parameters are defined as follows:

- elastic stiffness of the uncracked panel:

$$K_{el} = 4.2 \cdot K_{peak} \quad (9)$$

- secant stiffness at peak resistance:

$$K_{peak} = \frac{E_w \cdot t_w \cdot b_w}{d_w} (\cos \theta)^2 \quad (10)$$

- post-peak stiffness:

$$K_{pp} = -0.08 \cdot K_{el} \quad (11)$$

- cracking shear strength:

$$F_{cr} = 0.75 \cdot \tau_{cr} \cdot A_w \quad (12)$$

where A_w is the horizontal cross-sectional area of the masonry.

- peak resistance:

$$F_{max} = 1.2 \cdot \tau_{cr} \cdot A_w \quad (13)$$

The parameters obtained from the above formulas are presented in Table 5.

Walls	F_{cr} kN	F_{max} kN	K_{el} kN/m	K_{peak} kN/m	K_{pp} kN/m
Bricks w/ vertical holes	170.8	273.2	62963.0	14991.2	-5037.0
Bricks w/ horizontal holes	118.2	189.0	31629.7	7530.9	-2530.4

Table 5: Backbone curve's parameters for both horizontal and vertical holes brick masonry.

Such modeling strategy requires using four equivalent diagonal struts converging at the center of the masonry panel. These struts are connected by a nodal element lumping all the nonlinearity derived from the previously obtained force-displacement curves. Steel is adopted as the material for the diagonal struts, as it is essential for them to remain within the elastic range.

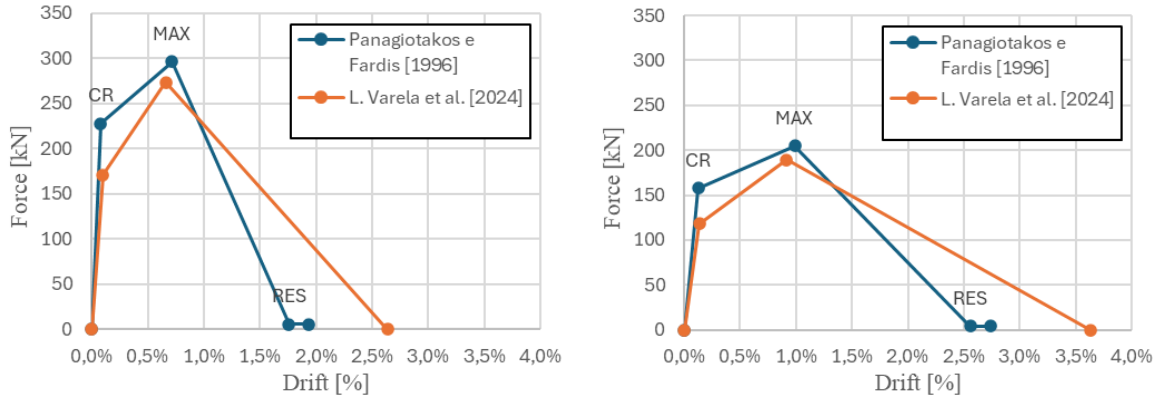


Figure 7: Backbone curve for infill wall made of hollow clay bricks masonry with vertical holes (left) and horizontal holes (right).

3.3 Seismic isolators

FREIs were modeled as point link elements with a nonlinear hysteretic Bouc-Wen constitutive relationship. Both VC-FREIs and RP-FREIs were manufactured by the Italian company Italgum S.r.l. and tested at the Mechanical Engineering Laboratory of the University of Naples “Federico II” to derive their nonlinear hysteretic behavior.

The “Isolator 1” element was employed in SeismoStruct: it is a zero-length 3D element featuring a nonlinear hysteretic constitutive law in the two shear directions and a linear elastic behavior in the remaining four deformation components. The force-deformation relationship in the shear directions is based on the model proposed by Wen [12] and Park et al. [8] and is defined by the following equations:

$$f_2 = \eta_{shear,2} \cdot K_{shear,2} \cdot u_2 (1 - \eta_{shear,2}) \cdot f_{y,shear,2} \cdot z_2 \quad (14)$$

$$f_3 = \eta_{shear,3} \cdot K_{shear,3} \cdot u_3 (1 - \eta_{shear,3}) \cdot f_{y,shear,3} \cdot z_3 \quad (15)$$

where $\eta_{shear,2}$ and $\eta_{shear,3}$ are the strain hardening ratios, $K_{shear,2}$ and $K_{shear,3}$ are the elastic stiffnesses, $f_{y,shear,2}$ and $f_{y,shear,3}$ are the yield strengths, and z_2 and z_3 are the internal hysteretic variables in the two shear directions of the isolator.

A total of thirteen parameters are required to define the behavior of the Isolator 1 element:

- elastic stiffness in the axial direction, K_{axial} ;
- elastic stiffnesses in the two shear directions, $K_{shear,2}$ and $K_{shear,3}$;
- elastic stiffnesses for the rotational and torsional degrees of freedom, $K_{torsional}$, $K_{rot,2}$, and $K_{rot,3}$;
- characteristic yield strengths of the isolator in the two shear directions, $f_{y,shear,2}$ and $f_{y,shear,3}$;
- strain hardening ratios in the two shear directions, $\eta_{shear,2}$ and $\eta_{shear,3}$;
- parameter A, which governs the amplitude of the hysteresis loop;
- hysteresis shape parameters β and γ , such that $A / (\beta + \gamma) = 1$.

The values adopted for such parameters are provided in Tables 6 and 7. In particular, the axial and shear stiffnesses, yield strengths, and strain hardening ratios were calibrated on experimental shear-compression tests, whereas the remaining parameters correspond to the software default values, as they were found not to play a significant role in sensitivity analysis. Figure 8 also presents the bilinear hysteretic relationship equivalent to the one included in the model. A different set of parameters was adopted to take into account higher stability limits of VC-FREI compared to RP-FREI, considering a target displacement of 140mm and 105 mm corresponding to 200% and 150% shear strain, respectively.

Type of FREI	K_{axial} kN/m	$K_{shear,2}$ kN/m	$K_{shear,3}$ kN/m	$K_{torsional}$ kN/m	$K_{rot,2}$ kN/m	$K_{rot,3}$ kN/m	K_{eff} kN/m
Virgin Carbon (VC)	25000	300	300	20000	20000	20000	100
Recicled Polyester (RP)	25000	460	460	20000	20000	20000	157

Table 6: Stiffness parameters for VC-FREIs and RP-FREIs.

Type of FREI	$f_{y,shear,2}$ N/mm ²	$f_{y,shear,3}$ N/mm ²	$\eta_{shear,2}$ -	$\eta_{shear,3}$ -	A -	β -	γ -
Virgin Carbon (VC)	4,50	4,50	0,22	0,22	1,00	0,50	0,50
Recicled Polyester (RP)	7,00	7,00	0,23	0,23	1,00	0,50	0,50

Table 7: Calibration parameters for VC-FREIs and RP-FREIs.

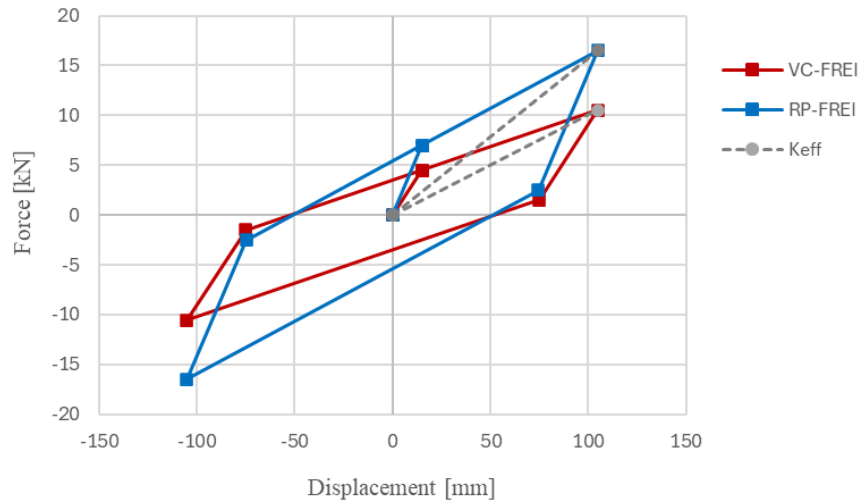


Figure 8: Backbone curves for shear behavior of VC-FREIs (left) and RP-FREIs (right).

4 MODAL ANALYSIS

Modal analysis was run for both FB and BI configuration. Fundamental vibration mode of the FB bare frame provides a period of 0,149 s, whose deformation occurs in the direction of the plasterboard walls (direction Y). The presence of infill walls (direction X) reduced the vibration period by approximately 40% to 0.090 s (mode 3 in the fixed-based model) thus playing a major role in terms of stiffness contribution. Both translational modes have a participating mass higher than 85% due to regularity of both geometry and mass distribution (Table 8).

When considering BI system, the fundamental period experienced a significant increase due to lateral flexibility of the FREIs, which controls the deformation of the structure. Two different models were adopted for RP-FREIs and VC-FREIs resulting in a target period of 1.6 s (Table 9) and 2.0 s (Table 10), respectively.

Mode	Period [s]	Ux	Uy	Uz	Rx	Ry	Rz
1	0,149	0%	88%	0%	43%	0%	0%
2	0,103	0%	0%	0%	0%	0%	87%
3	0,090	86%	0%	0%	0%	46%	0%

Table 8: Effective modal mass percentages for fixed-base configuration (first three vibration modes).

Mode	Period [s]	Ux	Uy	Uz	Rx	Ry	Rz
1	1,611	0%	100%	0%	0%	0%	0%
2	1,609	100%	0%	0%	0%	0%	0%
3	0,380	0%	0%	0%	0%	0%	100%

Table 9: Effective modal mass percentages for RP-FREIs isolated configuration (first three vibration modes).

Mode	Period [s]	Ux	Uy	Uz	Rx	Ry	Rz
1	2,014	0%	100%	0%	0%	0%	0%
2	2,012	100%	0%	0%	0%	0%	0%
3	0,383	0%	0%	0%	0%	0%	100%

Table 10: Effective modal mass percentages for VC-FREIs isolated configuration (first three vibration modes).

It should be noted that for the modal analysis the secant stiffness K_{eff} was used instead of the initial shear stiffness K_{shear} when defining the isolator properties (Table 6). Finally, it is worth noting that, for the isolated structure, the first two modes are translational in the horizontal plane and exhibit nearly identical vibration periods. This confirms that the superstructure stiffness has a relatively minor influence on the dynamic response of the system, including the differential stiffness introduced by the brick infill and plasterboard walls in the two horizontal directions. The third mode is torsional, while the fourth and fifth are associated with rocking motion.

5 PUSHOVER ANALYSIS OF THE FIXED-BASE BUILDING

Nonlinear static or pushover analysis was run for the FB configuration using displacement increments up to a predefined target equal to 5% of the total building height. By plotting the base shear versus top displacement at each increment step, the desired force-displacement relationship, i.e. the pushover curve, is obtained.

Two different pushover analyses were performed along the two main directions (X, Y) and corresponding results are shown in Figure 9. A significant difference in structural behavior can be noticed due to the presence of infill walls. Notably, the failure of the infills results in a sudden

drop in the post-peak strength. This indicates a brittle failure mechanism, occurring at very low displacement level of the order of 3.5 cm corresponding to an interstorey drift of approximately 1%.

Another key observation is that after the infill walls failure the green curve (X direction) closely follows the red curve (Y direction) corresponding to the bare frame model. This behavior is consistent with expectations thus contributing to model validation. The ultimate chord rotation was attained for a top displacement of around 12 cm that divided by total height corresponds to an average drift of about 2%.

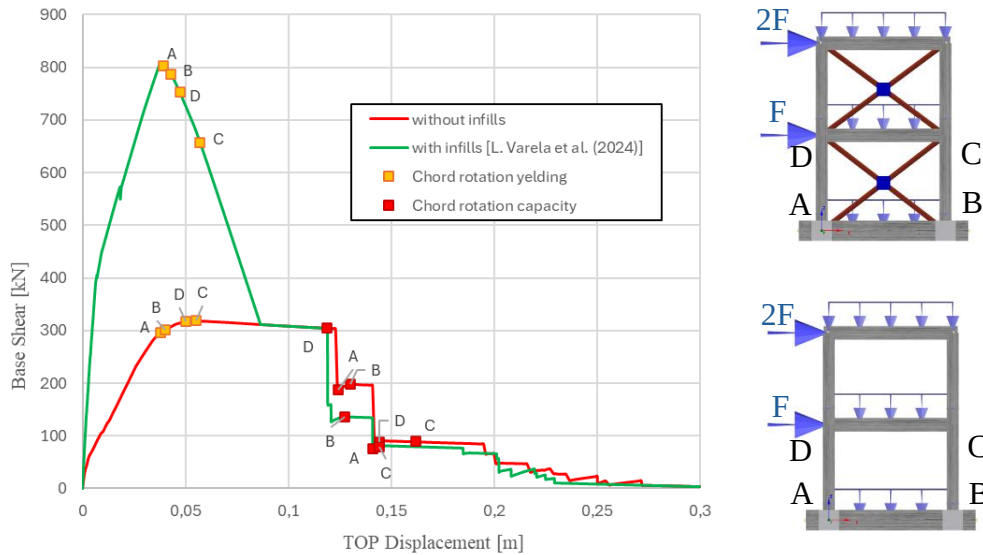


Figure 9: Pushover curves of the RC frame with infill walls (X) and without infill walls (Y).

In both cases, plastic hinges formed and reached collapse at the corners of the first-story columns, thus forming a so-called soft-storey collapse mechanism (Figure 10).

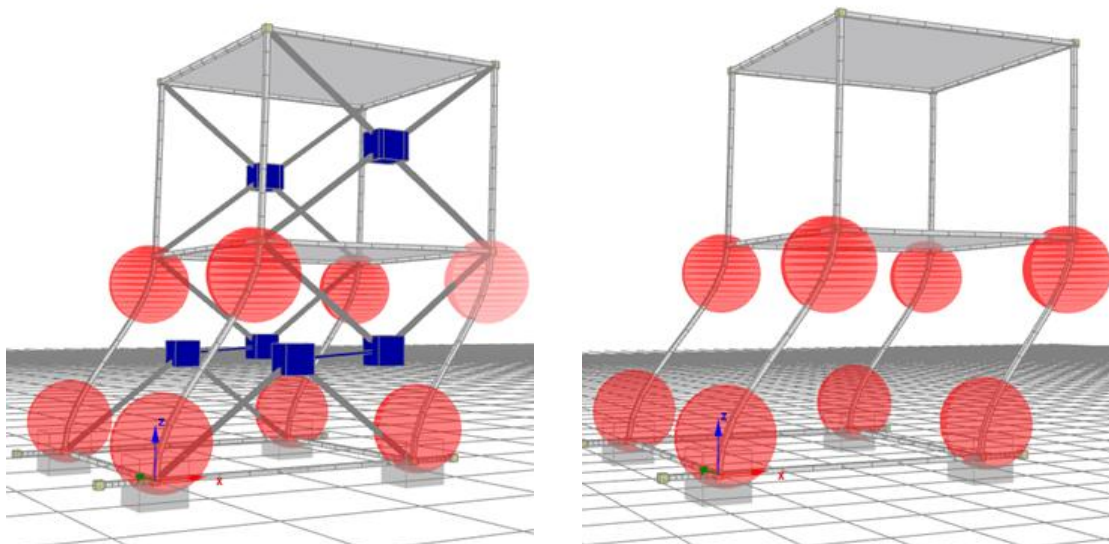


Figure 10: Collapse mechanism due to achievement of plastic hinge capacity, comparison between infilled frame (left) and bare frame (right).

By plotting the force-displacement relationship for the nonlinear link elements modeling the infill wall behavior (Figure 11), it can be seen that they fully developed their force-displacement relationship reaching complete failure. In contrast, the second-story infill walls cracked without attaining their peak strength.

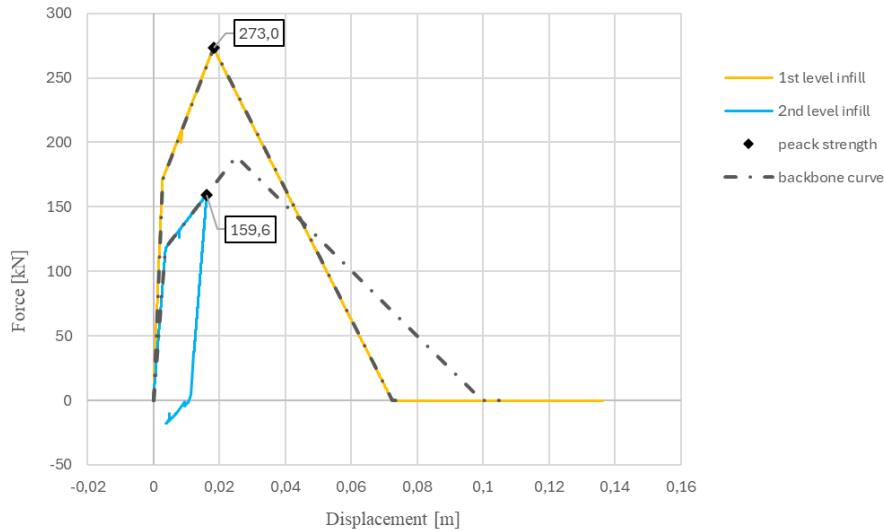


Figure 11: Force-displacement relationships of the infill walls.

6 TIME-HISTORY ANALYSIS OF THE BASE-ISOLATED BUILDING

Nonlinear time history analysis (NLTHA) was performed to calibrate the preliminary ground motion selection for the STT. The accelerograms were selected based on their spectrum compatibility from the NGA-West 2 database developed by the Pacific Earthquake Engineering Research Center (PEER) [13].

The following ground motions (GMs) were selected, including three non-pulse-like and one pulse-like events:

- Northridge Earthquake, Los Angeles, California, USA – January 17, 1994, $M_w = 6.69$;
- Loma Prieta Earthquake, California, USA – October 18, 1989, $M_w = 6.93$;
- Kobe Earthquake, Japan – January 16, 1995, $M_w = 6.90$;
- Jiashi Earthquake, China – April 11, 1997 (pulse-like), $M_w = 6.10$.

For the preliminary selection from the PEER database, the original accelerograms were scaled by scale factors (SFs) in such a way that the spectral displacement in the direction of maximum ground motion intensity would attain a peak of 105 mm considering a 15% damping ratio at a period of 1.6 seconds, which corresponds to the target period of the RP-FREIs specimen. To avoid excessive scaling of the seismic signal the SFs of the selected ground motions were desired to be close to 1.0. This was achieved by setting an extra check during the selection procedure. Moreover, it was necessary to verify that all the selected seismic records could have been applied by ST3D, considering its limitations in terms of displacement and velocity. The ST3D displacement limits are 200 mm for horizontal displacement and 180 mm for vertical one, while velocity limits are 60 cm/s, 50 cm/s, and 40 cm/s for the East-West (X), North-South (Y), and vertical (V) directions, respectively. From this point forward, the direction of the maximum seismic intensity will be conventionally referred to as the X direction, and during the analysis, it is aligned with the direction of the specimen's infills.

After a first set of NLTHA considering the four records with the corresponding SFs based on spectral matching it was found that in the case of non-impulsive earthquakes the isolator

displacement was well below the target value. Conversely, for the pulse-like ground motion the displacement values obtained from NLTHA were relatively close to the target. A correction factor (CF) was introduced and multiplied by the scale factor SF to obtain an appropriate seismic input towards the matching of isolator target displacement values. A trial-and-error approach was then adopted by applying $CF > 1$ to non-impulsive inputs and a $CF < 1$ to impulsive ones. The resulting SFs and CFs are reported in Table 11. As can be seen in Table 12, peak ground displacement and velocity for all the scaled records (scaled by $SF \times CF$) fit the ST3D limits.

Event Name	Applied SF	Applied CF	SF $\times$ CF
Northridge [1.45]	1.03	1.40	1.45
Loma Prieta [1.15]	1.05	1.10	1.15
Kobe [1.26]	0.90	1.40	1.26
Jiashi [0.96]	1.20	0.80	0.96

Table 11: Actual values for SF and CF.

Event Name	PGA _X g	PGA _Y g	PGA _V g	PGV _X cm/s	PGV _Y cm/s	PGV _V cm/s	PGD _X mm	PGD _Y mm	PGD _V mm
Northridge [1.45]	0.37	0.32	0.17	30.5	36.3	12.6	98	84	49
Loma Prieta [1.15]	0.56	0.17	0.09	45.8	18.7	11.0	174	85	44
Kobe [1.26]	0.63	0.64	0.47	46.0	46.9	21.7	142	120	71
Jiashi [0.96]	0.25	0.30	0.37	34.4	19.2	9.7	64	31	13

Table 12: Peak Ground Accelerations, Velocities and Displacement of selected GMs scaled by SF $\times$ CF.

The plots presented in Figure 12 show the peak top displacements in X direction for each selected GM scaled by SF $\times$ CF. The superstructure remains well within the elastic branch of the pushover curve exhibiting top displacements below 3 mm. Inter-storey drifts remained below 1.5 mm (drift $< 0.05\%$), thereby preventing the onset of cracking in the infill panels which would require drift levels higher than 0.1% according to Figure 7.

In terms of accelerations, it is worth mentioning that despite the high levels of imposed PGA, the peak storey accelerations (PSAs) were significantly mitigated due to the filtering effect of FREIs (Figure 13), especially with VC-FREIs.

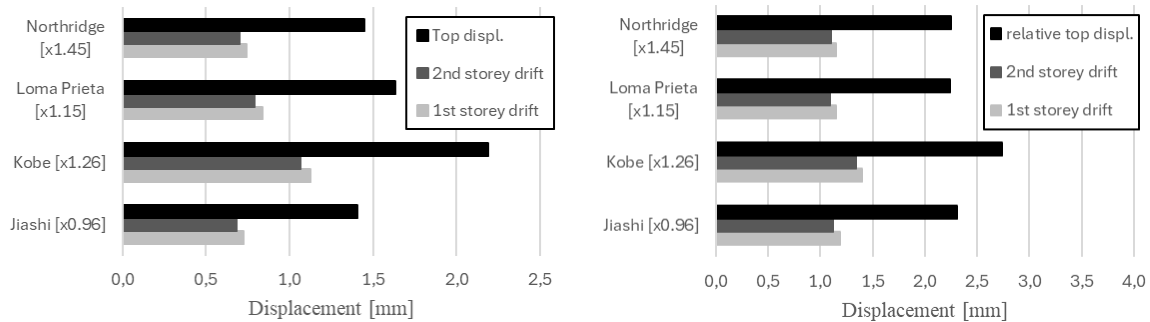


Figure 12: Relative displacement values of the superstructure (X direction) in both VC-FREIs (left) and RP-FREIs (right) configurations.

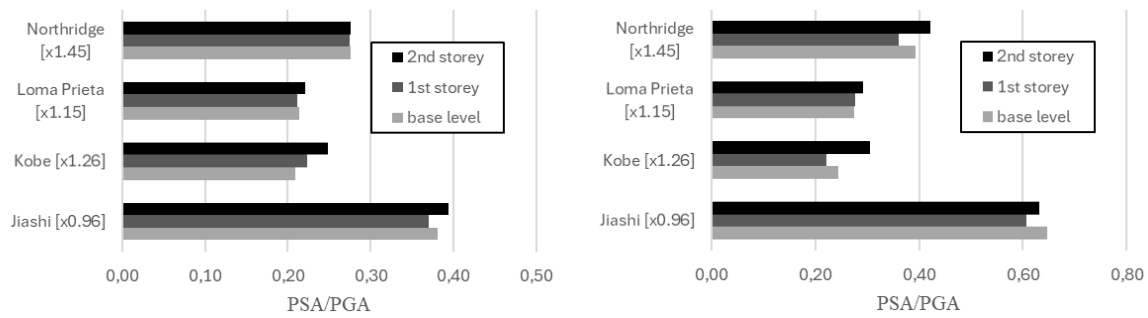


Figure 13: Ratio between the PSA and the PGA (X direction) in both VC-FREIs (left) and RP-FREIs (right) configurations.

7 CONCLUSIONS

This paper reported a numerical investigation on the seismic response of a full-scale 2-storey RC frame isolated at the base with innovative low cost FREIs. The building specimen was designed according to gravity loads only to be tested on a 3D shaking table at LNEC. Nonlinear numerical modeling followed a distributed plasticity approach for RC elements, equivalent strut models for infill walls and a Bouc-Wen relationship for FREIs. Through pushover analysis the fixed base building demonstrated a significant contribution from the infill walls both in terms of strength and stiffness and a limited displacement capacity due to a soft-storey mechanism at the ground level. For the base isolated configuration nonlinear time history analyses were run to define the target scaling of input records for the STT based on a preliminary spectral matching selection including both far-fault and near-fault ground motions. The target displacement of the isolation system was set equal to 105 mm considering an equivalent damping ratio of 15% according to preliminary characterization tests. The target period of the base isolated building was set in between 1,6 s and 2,0 s due to different stability limits of RP-FREI and VC-FREI, respectively. NLTHA confirmed no damage to RC elements and no cracking of the infill walls thus achieving satisfactory performance. The numerical modeling supported the design of the building specimen and the selection process of the ground motions predicting the seismic response with high accuracy. The experimental tests then confirmed the satisfactory behavior of the base isolated configuration both in terms of FREIs stability and damage prevention of the superstructure and nonstructural components.

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