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Experimental Investigation of Steel Frames Equipped with Dissipative Replaceable Links

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Abstract

In the last decades, high priority has been given to community disaster resilience owing to seismic events with a particular focus on the post-disaster restoration. Therefore, damage reduction of structural and non-structural elements after a disaster is fundamental for costs and for functionality aspects. In this context, the European RFCS project DISSIPABLE was funded with the aim to perform large demonstration tests of steel frames equipped with easily repairable seismic dissipative devices. In this paper, the experimental tests performed according to dynamic substructuring are described. The capacity of withstanding seismic actions as well as the energy dissipation relies on the Dissipative Replaceable Link Frame system, composed of two rigid columns connected by weakened beams. Two different configurations of frames equipped with DRLF systems were tested: i) frames made of only mild steel and ii) frames made of both mild and high-strength steel. Bidimensional frames were tested under different seismic intensity levels: Damage Limitation, Significant Damage and Near Collapse.

Keywords

Full-Scale Experimental Tests; Dynamic Substructuring; Pseudo-dynamic testing; Steel Structures; High-Strength Steel; Seismic Devices; Resilient Structures.

1 Introduction

Due to the potential high risk of damage associated with the earthquake, efforts have been done by the scientific community to increase the resilience of structures. By employing the capacity design philosophy, energy dissipation is achieved through the development of inelastic deformation in the structural members. Hence buildings designed according to this approach undergo significant damage whose repair work is most of the times not feasible or too expensive with consequences at the societal level. Therefore, reduction of damage of structural and non-structural elements after a disaster is fundamental for costs and for functionality aspects.

In this context, the European Research Fund of Coal and Steel (RFCS) DISSIPABLE project was funded to provide experimental evidence on both the high degree of energy dissipation and the easily replaceability after a major seismic event of full-scale steel structural specimens endowed with dissipative seismic components. In this paper, the results of the experimental campaign carried out at the University of Trento will be discussed.

The specimens under investigation were equipped with the Dissipative Replaceable Link Frame (DRLF) system, that is the lateral resisting system, and it is composed of two rigid columns connected by beams with reduced sections. Two different configurations of frames equipped with DRLF system were tested: i) frames made of mild steel only and ii) frames made of both mild and high-strength steel. Pseudo-dynamic hybrid tests were carried out with the use of the substructuring technique, i.e. only the ground floor of the structure was physically tested in the laboratory, whilst the remainder was numerically simulated. Bidimensional frames, representative of a 3D case study, were tested. Different seismic intensity levels were applied to the structures: Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC). In particular, the aim of the project was to demonstrate the effectiveness of the components to exhibit favourable hysteretic behaviour as well as to be easy to replace after a major seismic event.

The paper is organised as follows: a brief description of the dissipative component under investigation is given in Section 2; in Section 3 the numerical models of both the component and the building prototypes are reported; whilst the description of the experimental test results is presented in Section 4; in Section 5 a comparison of the behaviour of the two structure is illustrated and finally in Section 6 conclusive remarks are drawn.

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2 Dissipative Replaceable Link Frame (DRLF) system

The dissipative replaceable link frame system shown in Figure 1, is intended to be used only in the external frames of a steel building to not occupy the internal space. The component is made up of two closely spaced strong columns, 1.25m in the frame under examination, rigidly connected by beam links with reduced sections (RBS) at both ends [1]. The whole system behaves as a Vierendeel beam, in which the beam links work mainly in bending or in shear, depending on their length, and the columns are subjected to a strong axial force component [2]. In this research, the design of the DRLF aimed at dissipating according to a flexural mechanism. For this system, replaceability is fostered by means of bolted connections between the devices and the columns. Moreover, the beam links are not part of the gravity load carrying system.

Figure 1 DRLF system configuration

3 Numerical modelling

In this section a brief description of the numerical modelling of the dissipative component of the building and the frames is reported.

3.1 Component modelling

The beam links were modelled in the finite element software OpenSees [3] as a series of five elements, where the non-linearity is condensed on the reduced beam sections (RBSs). The remaining parts were elastic beam elements with the stiffness property of the gross section. The hysteretic behaviour was modelled by means of the Bouc-Wen model, whose parameters were determined by fitting the numerical curves obtained by the finite element model developed in the software ABAQUS [4]. Figure 2 shows the numerical hysteretic behaviour of one of the RBSs that has been used in the following numerical models.

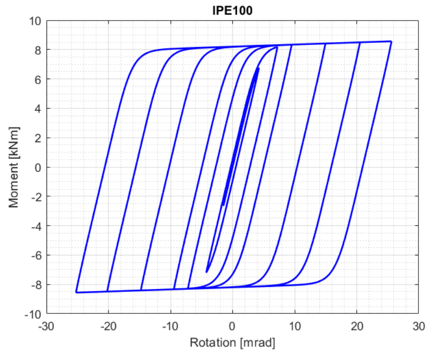
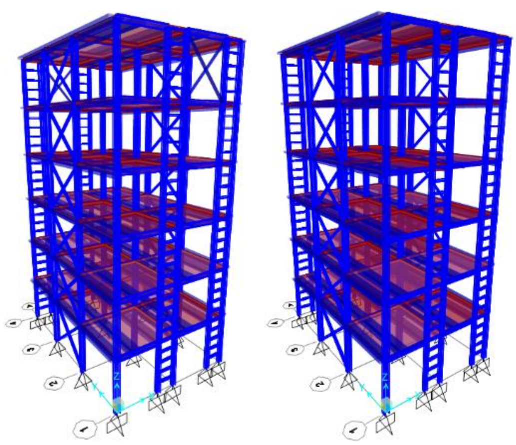


Figure 2 DRLF numerical hysteretic behaviour

3.2 Building prototypes

The prototype buildings under investigation were composed of two spans in the transversal X-direction, three spans in the longitudinal Y-direction and six-storeys. In the Y-direction the horizontal carrying load capacity relies on two external braced frames. In the X-direction instead, two parallel DRLF systems were employed for each external frame and coupled to reduce the building deformability. The stiffness increasing was achieved with two different approaches: in the first case, when the entire structure was made of mild steel (MS), DRLF – MS for brevity, by adding bracing elements at the top floor, see Figure 3a, whilst in the second one by high-strength steel (HSS) beams that were alternatively fixed to the column leaving the other end hinged, as shown in Figure 3b. In particular, the latter solution was extensively studied by Pinkawa et al [5].

In order to design the structure by means of linear dynamic analysis, the initial modelling of the 3D building was developed in SAP2000 [6]. The structure design was carried out according to EN 1998-1 [7] whilst the RBSs were detailed as reported in EN 1998-3 [8], as suggested by Pinkawa et al [9]. The nonlinear model of each building was developed in OpenSees. Modal, push-over and time-history analyses were employed to investigate the buildings response. Accelerograms at Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC) limit states were used to perform nonlinear time-history analyses, Table 1.



a) b)

Figure 3 Case-study buildings: a) DRLF – MS and b) DRLF – HSS

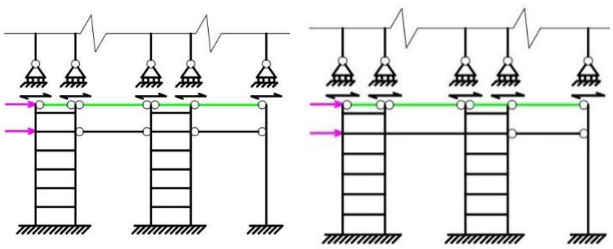
Table 1 Limit State characteristics.

Limit state	a_g [g]	T_R [year]
DL	0.200	60
SD	0.360	475
NC	0.504	1600

The procedure that was followed was divided into two steps: i) development of 2D nonlinear FE models in OpenSees that are as representative as possible of the 3D models; ii) development of meaningful 2D nonlinear FE models of substructures that are representative of the actual partition between physical and numerical substructures hybrid tests. The process was widely described by the authors in [10], where all the meaningful results are presented. For brevity, only the comparison in terms of periods is reported in Table 2 where the discrepancies between the three level of modelling is always lower than the 10%. In Figure 4 the substructured configurations are depicted: as the reader may notice, one actuator is located at the floor level, whilst the second one is placed at the point of contraflexure of the column of the second floor, which is approximately located at the column mid-height.

Table 2 First period along the X direction. Dimensions in sec.

Model	DRLF-MS	DRLF-HSS
3D	1.52	1.40
2D	1.55	1.38
2D Subst	1.39	1.37



a) DRLF Top Braces b) DRLF HSS

Figure 4 Substructuring configurations

4 Pseudo-dynamic hybrid tests

The pseudo-dynamic method was employed, which allows to run seismic records on a structure by expanding the simulation time by a time-scale factor λ, to avoid the effect of the structure inertia. As shown in Eq. (1) the time scaling factor is given by the ratio between the time integration step used to solve the equation of motion Δt_C and the wall clock time that marks the solution of one-time integration step Δt [12].

$$\lambda = \Delta t_c / \Delta t \tag{1}$$

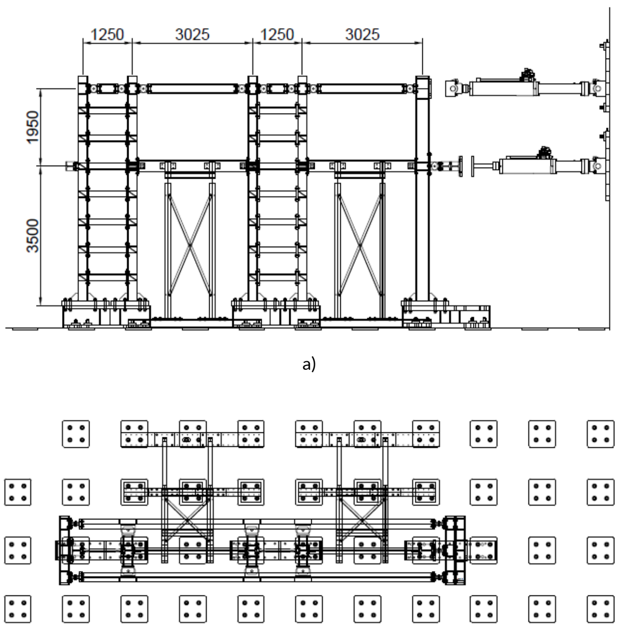
Moreover, in order to test full-scale structures, the substructuring technique was employed to divide the structure into a numerical subdomain and a physical subdomain. In this respect, the tests were conducted by means of the G-α algorithm described by Abbiati et al. [11].

4.1 Accelerogram selection

The tests were conducted at three limit states, namely at the Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC) limit states. For each of them, an accelerogram was selected with the criteria of spectral compatibility, by checking the structural performance and by minimizing the errors between the monolithic and the substructured frame. The spectral compatibility was checked according to the Eurocode 8 [7] provisions, whilst the structural performance of the frame was evaluated by checking if the rotations of RBSs were in agreement with the considered limit state, e.g. all the RBSs reach a rotation that is lower than the yielding rotation for the DL limit state. Moreover, a uniform dissipative behaviour of the sections in addition to a maximum rotation compatible with the considered limit state was reached for both the SD and the NC limit states. It should also be highlighted that, for the DRLF – MS frame, the increasing of stiffness given by the bracing system caused small rotations of the RBSs at the last floor.

4.2 Hybrid test configuration

With regard to the laboratory layout, the first floor and a half of the structure was physically tested by means of two actuators, as depicted in Figure 5a. The physical substructure of the frame is composed of five columns: the left one is not part of the DRLF systems whilst the others, coupled with the link beams, constitute the two shear walls that carry the horizontal loads. In order to impose the same displacement at the top of each of the columns, beams with high axial stiffness were placed at the level of the higher actuator. Furthermore, two rigid axial beams are laterally placed at the floor level to replicate a rigid diaphragm, as illustrated in Figure 5b.



a)

b)

Figure 5 Experimental test set-up for test on DRLF - MS frame: a) Front and b) Plan view

A schematic representation of the hybrid test configurations is depicted in Figure 6 and Figure 7.

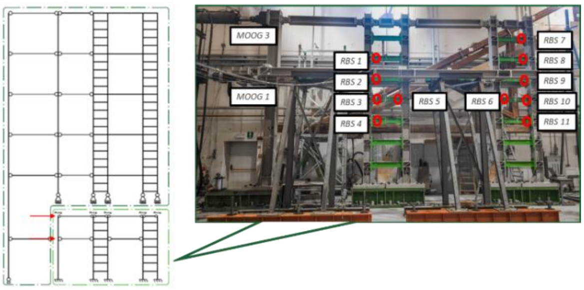


Figure 6 Hybrid test configuration on DRLF – MS frame

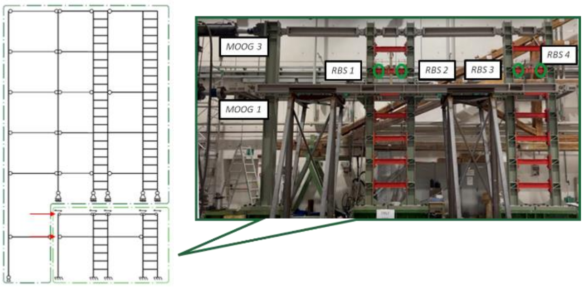


Figure 7 Hybrid test configuration on DRLF – HSS frame

In both tests several sections were fully instrumented by means of both strain gauges, installed in an elastic region near the RBSs, and displacement transducers. The former served to estimate the bending moment while the latter to compute the rotation of the reduced beam section. Moreover, for the DRLF – MS frame also the external column was instrumented so as to capture the yielding of the column base. For DRLF – HSS, the beams section closest to the rigid beam to column connection were instrumented to calculate the bending moment and to detect yielding.

As shortly illustrated before, several sections were instrumented to estimate both bending moments and rotations. In this respect, the upper and the lower edge of beam link sections were instrumented to measure the strain in an elastic region of the beam. The curvature was then calculated by assuming plane sections:

$$\chi = (\varepsilon_{\text{top}} - \varepsilon_{bottom})/H_{sec} \tag{2}$$

Where ε_{top}, and ε_{bot} are the strains measured at the top and at the bottom of the section and H_{sec} is the height of the cross section. An estimation of the bending moment on each instrumented section, located in the elastic range, could be then obtained by means of the following formula, in which I_{beam} is the modulus of inertia of the section and E_s is the Young's steel modulus.

$$M = E_s \cdot I_{beam} \cdot \chi \tag{3}$$

The rotation of the RBS was then calculated as:

$$\varphi = (\Delta_{\text{top}} - \Delta_{bottom})/H_{sec} \tag{4}$$

Where Δ_{top} and Δ_{bot} are the top and the bottom displacements.

4.3 Model reduction

Aiming at reducing the computational burden, both the DRLF models were reduced to a simplified one. For DRLF – MS, the condensation was performed under the assumption of shear-type deformation of the structure that led to consider only seven horizontal degrees of freedom. These DoFs represent the displacements at each floor level and the one at the substructuring level. Lumped masses, connected by means of nonlinear shear springs, were located on each DoF. On the other hand, for DRLF - HSS model the reduction was performed at two different levels. Firstly, the beam links were modelled as a single shear spring, whose nonlinear parameters were calibrated to reproduce the behaviour of the entire beam link. Secondly, a series of five shear links were condensed in a single one, located at the mid-height of the floor. In both cases, in order to calibrate the nonlinear springs, a displacement control analysis was performed by imposing a cyclic displacement at the top floor of the reference model.

4.4 Hybrid test results – MS Frame

Hereafter, the results of the hybrid tests are briefly reported and described. For each limit state, good agreement between the hybrid test results and the OpenSees reference model was found. This highlights the capability of the hybrid simulation technique to conduct tests on an experimental substructure yet allowing for the behaviour of the whole structure.

In particular, for the DL limit state the elastic behaviour of the frame was confirmed from the bending moment of the RBSs, which was always lower than the elastic one. Besides that, the relation between the base shear and the top floor displacement remained linear.

For the SD limit state, the structure showed an inelastic behaviour as is also highlighted by Figure 8, which shows the bending moment of one RBS. Indeed, during the earthquake the bending moment in the devices exceeded the elastic bending moment. As can be seen from Figure 9, a residual displacement was observed at the first floor after the significant damage test, which testifies that the structure entered the plastic region. Furthermore, Figure 9 also shows that the residual displacement was greater than 1.4 mm.

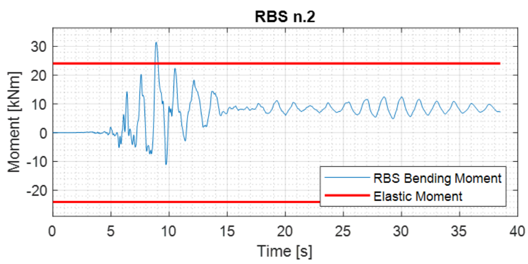


Figure 8 SD MS Frame – Bending moment of selected RBS

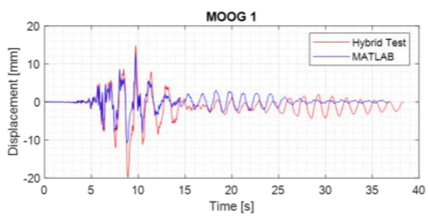


Figure 9 SD MS Frame – Displacement history at floor level

Figure 10 depicts the comparison in terms of base shear between the hybrid test and the OpenSees reference model. It is possible to observe that at the NC limit state a good agreement between the hybrid test results and the OpenSees reference model is highlighted.

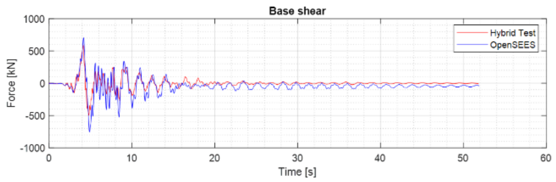


Figure 10 NC MS Frame – Comparison between the base shear of the test results and OpenSees reference model

Figure 11 shows the comparison in terms of Base Shear vs. Top Floor Displacement between the hybrid test and the OpenSees reference model. The graphs are superimposed with the pushover curve (black line) of the structure obtained from the OpenSees reference model and show that the frame exhibited significant inelastic behaviour.

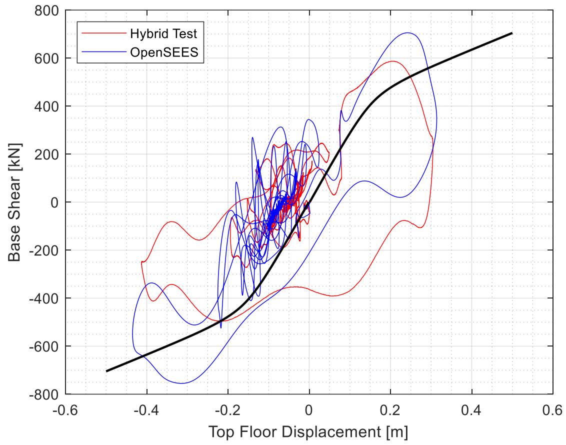


Figure 11 NC MS Frame – Base Shear vs. Top Floor Displacement

The inelastic behaviour is also highlighted in Figure 12 where the moment-rotation diagrams of one instrumented RBS is reported. Indeed, the bending moment measured experimentally exceeded the elastic resisting moment. Moreover, Figure 12 also shows the large amount of dissipated energy as well as wide cycles which confirm the excellent dissipative hysteretic behaviour of the DRLF. Moreover, the maximum rotation achieved is about 7mrad.

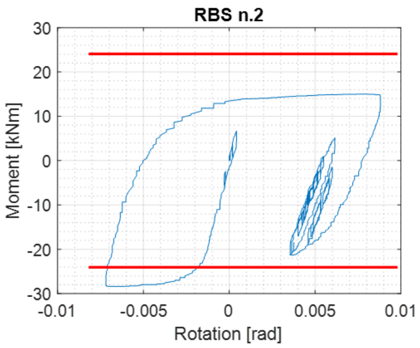


Figure 12 NC MS Frame – Moment-rotation diagrams of selected RBS

It is worth pointing out that for every limit state the left column base bending moment did not exceed the elastic bending moment confirming that columns remained elastic.

4.5 Hybrid test results – HSS Frame

With regard to the tests on the DRLF - HSS frame, it was possible to verify even in this case that both dissipative elements and the structure remained in the elastic field when subjected to an accelerogram at the DL limit state. Concerning the SD limit state, it can be noticed from Figure 13 that the RBS exceeded the plastic bending moment, as expected from the design.

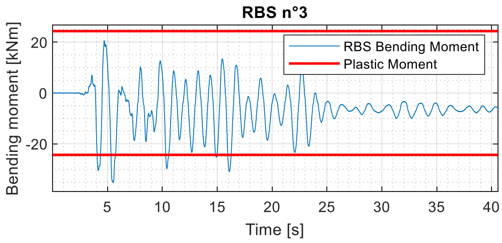


Figure 13 SD HSS Frame – Bending moment of selected RBS

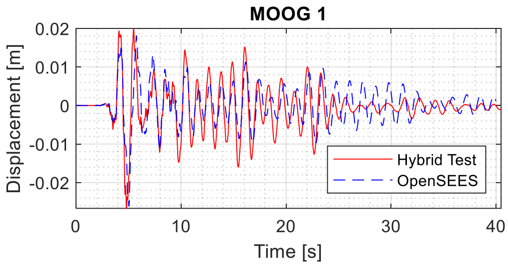


Figure 14 SD HSS Frame – Displacement history at floor level

Figure 14 depicts the displacement history over time at the floor level. The residual displacement at the floor level after the test was lower than 1 mm at SD limit state.

For the NC limit state, Figure 15 shows the comparison in terms of base shear vs. top floor displacement between the hybrid test and the OpenSees reference model. The graphs were superimposed with the pushover curve (black line) of the structure obtained from the OpenSees reference model; the structure exhibited significant inelastic behaviour, with a plastic residual drift of 1.1mm.

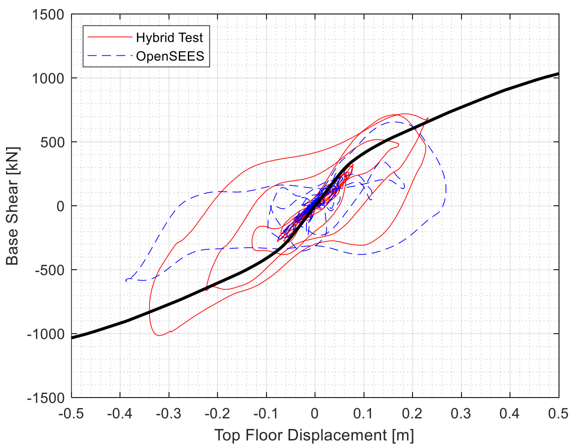


Figure 15 NC HSS Frame – Base Shear vs. Top Floor Displacement

The inelastic behaviour is also underlined in Figure 16 where moment-rotation diagram of a selected RBS is reported. Indeed, the bending moment measured experimentally exceeded the plastic resisting moment. Large and dissipative hysteretic behaviour is shown, and the RBS reached maximum rotations of more than 30mrad. The residual plastic deformation of the section can be observed.

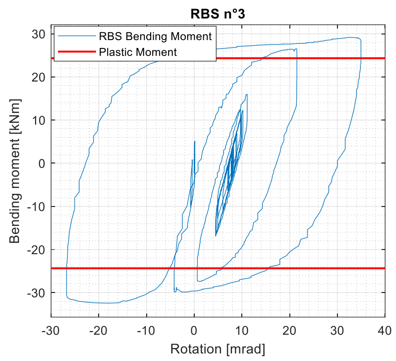


Figure 16 NC HSS Frame – Moment-rotation diagrams of selected RBS

5 Results comparison

Comparing the results of the two structures, equipped with the same dissipative component but coupled in different ways, it can be highlighted that they both behaved as expected at each limit state under investigation. In particular, no inelastic behaviour was detected at the damage limitation limit state whilst, at the significant damage, both structures went into the plastic region due to the plasticisation of the RBSs.

One difference between the two tests lies in the out of plane deformation of the RBSs. Indeed, for DRLF - MS a significant bending moment along the weak axis was detected which was not detected in the DRLF - HSS tests.

In both cases, even at the near collapse limit state test only the dissipative component yielded whilst the non-replaceable elements, e.g. beams and columns, remain into the elastic field. This confirmed the repairability of the structure after a strong earthquake event.

6 Conclusions

This article described the experimental campaign that took place at the Materials and Structures Testing Laboratory (MSTL) of the University of Trento. Initially, the procedure leading to the definition of the test frame was described, while the test results for the two frames under examination were subsequently shown. The capability of the hybrid simulation technique to conduct tests on an experimental substructure yet allowing for the behaviour of the whole structure was demonstrated by showing the results of the experimental campaign in comparison with the OpenSees reference models outputs. For both the specimens, the experimental behaviour satisfied the design assumptions, showing that the structures, including the devices, remained in the elastic field during the tests at the DL. For the SD limit state tests, only the devices underwent plastic deformation, demonstrating the reparability of the structure.

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