

# Hybrid simulation of existing steel frames with external BRBs for seismic retrofitting

## Project SERA-HITFRAMES - Part 2

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## 1 Introduction

Buckling-Restrained Braces (BRBs) have proven to be an effective solution to enhance the seismic performance of existing structures. When included within existing frames, BRBs create an additional load path and contribute to the overall strength, stiffness, ductility, and energy dissipation capacity of the structure [1, 2]. However, BRBs are typically placed aligned within the existing frames, thus requiring demolition and reconstruction of non-structural components. A possible way to address this limitation is to place BRBs externally to the frame minimizing the invasiveness of the intervention and, consequently, business interruptions and indirect losses. To investigate this solution, the HITFRAMES (*i.e.*, Hybrid Testing of an Existing Steel Frame with Infills under Multiple EarthquakeS) projects, funded by the EU-H2020 SERA Consortium in Europe, performed large-scale Pseudo-Dynamic Hybrid testing (referred to as PsD hereafter) on a non-seismically designed steel Moment-Resisting Frame (MRF) at the Structures Lab (STRULAB) of the University of Patras, Greece.

A two-story steel MRF designed primarily for gravity loads and retrofitted with BRBs placed externally to the frame was considered for case study purposes. The design of the case study building aimed to replicate the characteristics of an existing steel frame in Amatrice (Italy), which is considered to be representative of pre-code design scenarios and was extensively damaged after the 2016 Central Italy Earthquake. The first part of the experimental campaign aimed to evaluate the performance of the existing frame, including the influence of masonry infills, and the results are summarized in Di Sarno *et al.* [3]. The second part of the experimental campaign focused on the seismic response of the structure retrofitted with BRBs.

The test specimen of the retrofitted frame was a single-frame, two-story steel MRF. The rest of the structure was modeled in OpenSees (*i.e.*, the numerical substructure). The BRBs were installed considering a tailored connection, placing the BRBs eccentrically to the plane frame. Significant attention was placed on the connection details for BRBs and their influence on the response of the tested frame. PsD tests were performed for increasing intensities of the selected ground motion record.

## 2 Description of the specimen

### *The prototype structure*

Figure 1 shows the plan and elevation views of the prototype structure, which is a two-story, one-bay by three-bay non-seismically designed steel MRF. The building has a constant inter-story height of 3.4 m and bay widths of 4.65 m and 8.65 m, respectively, along the  $x$ - and  $y$ -directions. The total dimensions of the building are 13.95 m by 8.65 m in plan and 6.8 m in height. The building was designed for gravity loads only following the European design code for steel buildings, Eurocode 3, assuming a non-structural permanent load equal to 2.58 kN/m<sup>2</sup> and an imposed load equal to 3 kN/m<sup>2</sup>. Wind loads were also not considered for low-rise structures, as per Eurocode 3, leading to a complete lack of lateral load-resisting systems in the frame design. The steel profiles were HE 220 A, IPE 240, and IPE 160, respectively, for columns, primary and secondary beams, with the weak axis of columns in the  $x$ -direction. A steel grade S355 ( $f_y = 355$  MPa) was adopted for beams and columns. All primary beams were connected to columns through full penetration welds, and columns were fixed at the base. A 200-mm-deep concrete slab was also considered for each story. The mass of the prototype building was equal to 117.0 and 95.0 tons, respectively, for the first and second stories. The interested reader can refer to Di Sarno *et al.* [3] for additional details on the prototype structure.

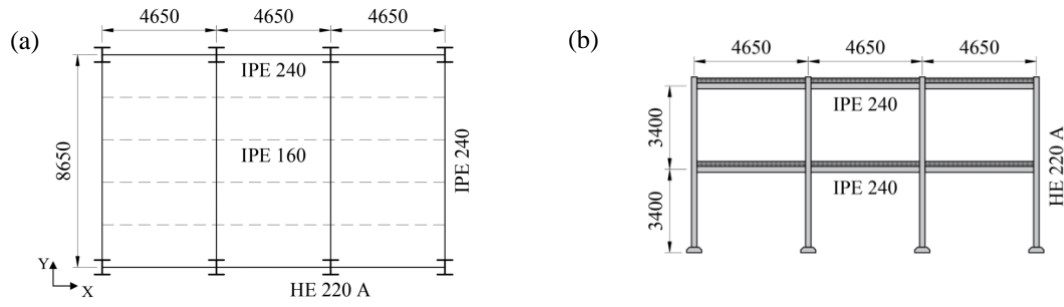


Figure 1. Prototype structure: (a) plan and (b) elevation views. [units in mm].

#### The scaled structure – scaling and similitude of the prototype building

The prototype structure was scaled down by a factor  $\lambda = 0.75$  for the experimental tests according to the lab capabilities and following the same strategy used in Di Sarno *et al.* [3]. The prototype structure after scaling is then referred to as the ‘scaled structure’ in the remaining part of the report. The model scaling was implemented assuming acceleration and material scaling identity, as summarized in Table 1. Such a scaling factor is typically considered adequate for investigating the frame’s seismic response at the global and local levels. As a result of the model scaling, the steel frame’s story height and bay width were reduced to 2.5 and 3.5 m, respectively, and the columns’ and beams’ profiles became HE180A and IPE200.

Table 1. Similitude scaling factors with  $\lambda = 0.75$ .

| Parameter   | Scaling factor          |
|---|-------------------------|
| Density   | $\lambda^{-1} = 1.33$   |
| Stress, strain, angular deformation, and acceleration | $\lambda^0 = 1.00$      |
| Period, time, and velocity                            | $\lambda^{-0.5} = 0.87$ |
| Length, linear deformation, and stiffness             | $\lambda^1 = 0.75$      |
| Force, weight, mass, and area                         | $\lambda^2 = 0.56$      |
| Volume, section moduli, and moment                    | $\lambda^3 = 0.42$      |
| Moment of inertia                                     | $\lambda^4 = 0.32$      |

#### BRBs design

According to Eurocode 8-Part 3, the retrofitted steel MRF should meet the requirements for a new structure in Eurocode 8-Part 1. BRBs were installed in the central bay of the steel frames only; hence, a total of four BRBs (*i.e.*, one per story of two parallel frames) were used for the retrofitting of the scaled structure. Although the BRBs were initially designed according to procedures described in the literature, the final choice for the BRBs used in the experimental campaign was dictated by the availability in the market. The selected BRB devices represent a slightly oversized but still realistic solution to the seismic retrofit of the steel frame. The properties of the BRB devices are provided in Table 2. The elastic stiffness ( $K_e$ ) is 88 kN/m, the force at first yielding ( $F_y$ ) is 125 kN, and the maximum allowable axial displacement ( $d_u$ ) is 20 mm.  $F_{1,T}$  and  $F_{1,C}$  are the yielding forces at stable hysteretic loops in tension and compression, which are 167 and 175 kN, respectively, and  $F_{u,T}$  and  $F_{u,C}$  are the maximum forces in tension and compression, which are 191 and 225 kN. The elastic braces had circular tube cross-sections with an external radius equal to 75 mm and a thickness of 10 mm.

Table 2. Property of the selected BRB device for retrofitting the scaled structure.

| $L$ (m) | $K_e$ (kN/m) | $F_y$ (kN) | $d_u$ (mm) | Tension        |                | Compression    |                |
|---------|--------------|------------|------------|----------------|----------------|----------------|----------------|
|         |              |            |            | $F_{1,T}$ (kN) | $F_{u,T}$ (kN) | $F_{1,C}$ (kN) | $F_{u,C}$ (kN) |
| 1.535   | 88           | 125        | 20         | 167            | 175            | 191            | 225            |

#### BRBs connection design

Figure 2 shows the geometry of the connection between the BRBs and the columns used for the specimen. The maximum effect due to the ultimate expected resistance of the BRBs (alternatively in tension and compression) was adopted to estimate the design demand on the bolted end-plate connections in terms of shear force, bending, and torsional moments.

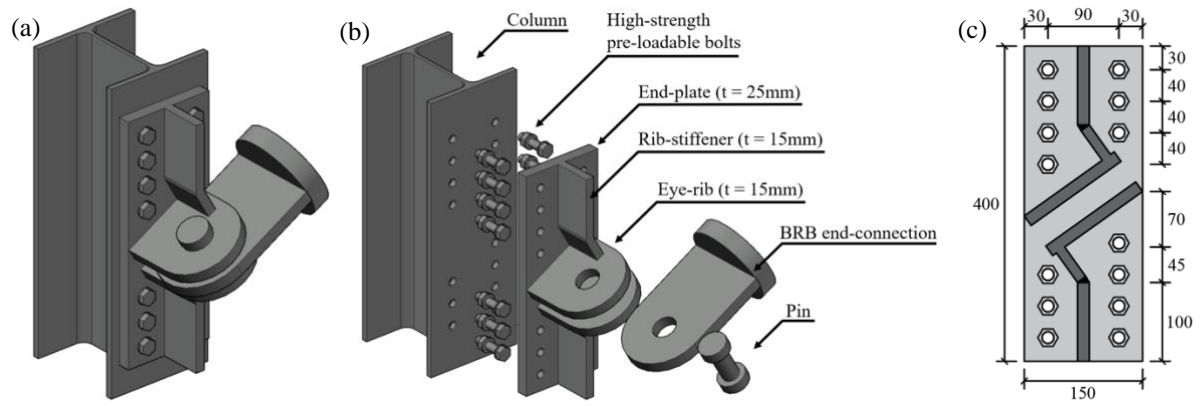


Figure 2. BRB-to-column connection details: (a) geometrical configuration; (b) disassembled configuration; (c) geometric dimension [units in mm].

### 3 Hybrid simulation

The specimen experimentally tested in the lab was the central bay plane frame of the *scaled structure*, as shown in Figure 3. Two configurations were considered, namely the *bare* and the *retrofitted test specimens*.

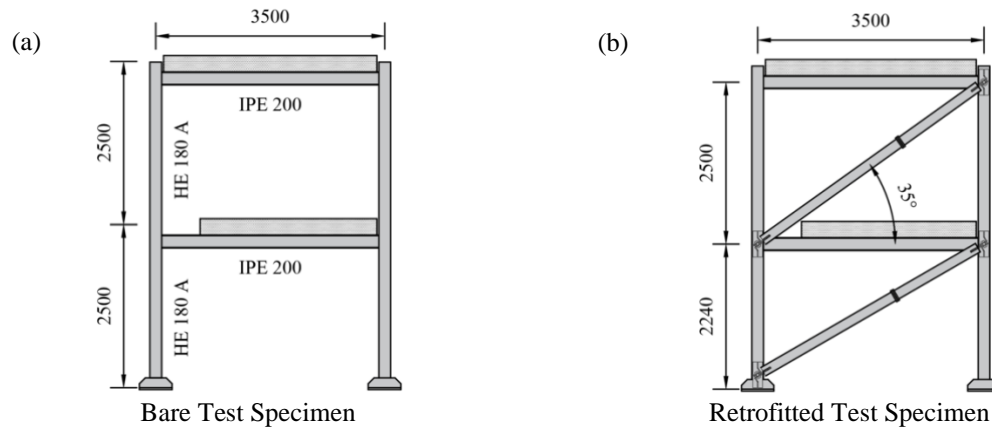


Figure 3. Schematic view of the (a) bare and (b) retrofitted test specimens [units in mm].

#### Test matrix

The test matrix is summarized in Table 3. A snap-back (free vibration) test was firstly performed on the test specimen in its bare configuration. A set of PsD tests was then conducted on both the bare and retrofitted configurations, considering incremental intensities of the ground motion record, with scaling factors (SF) ranging from 0.35 to 1.50. The SF = 1.5 was adopted to ensure the yielding of BRBs, such that the full behavior of BRBs could be observed during the test. The final test with SF = 1.5 was terminated due to large torsional effects in the columns.

Table 3. Test matrix for the bare and retrofitted steel frame.

| Test               | Description                                       |
|--------------------|---|
| BRB component test |   |
| Test specimen      | 1 Snap-back free-vibration test of the bare frame |
|                    | 2 PsD test of the bare frame (SF = 0.35)          |
|                    | 3 PsD test of the bare frame (SF = 0.75)          |
|                    | 4 PsD test of the bare frame (SF = 1.00)          |
|                    | 5 PsD test of the retrofitted frame (SF = 1.00)   |
|                    | 6 PsD test of the retrofitted frame (SF = 1.50)   |

### Test setup and instrumentation

Figure 4 shows the bare (a) and retrofitted (b) test specimen setup, including details on the installation of the external BRBs (c to g). The test specimen was built based on the common European construction practice. The steel frame was welded and prepared in the workshop and fully assembled in the laboratory. Four actuators were employed to conduct the PsD tests, as shown in Figure 4(a) and (b), with two connected to the slab at each story. Figure 4(i) shows the connection between actuators and slabs, which was designed to ensure a smooth transfer of stress from the actuators to the test specimen. Two parallel tubular beams were placed on top of the column base plates and anchored to the strong floor of the lab, as shown in Figure 4(f), in order to increase the rigidity of the base restraints of the test setup. The concrete slab was built following the construction of the steel frame. It is worth mentioning that stiffeners were also adopted to increase the rigidity of the beam-column connections, as shown in Figure 4(h). Figure 4(g) shows the connection between the bracing system and the BRB device. Moreover, Figure 4(c to f) show the details of the eccentric pin connections of the BRBs to the external flange of the columns.

Aside from the PsD tests, coupon tests were performed on steel pieces extracted from beams and columns of the test specimen and conducted according to the BS EN ISO 6892-1 standard. The experimental mean yield strength and ultimate strength of S355 steel were 424 and 575 MPa, respectively.



Figure 4. (a) Test setup of the bare test specimen; (b) test setup of the retrofitted test specimen; (c through g) BRB's connection details; (h) beam-to-column connection; (i) slab details.

The instrumentation for the PsD tests was designed to monitor both the global and local response of the test specimen. Figure 5 shows all the sensors used in the experimental tests. In the free vibration test, four accelerometers and four displacement-measuring optical devices (OPT1 to OPT4 in Figure 5) were used to monitor the story displacements and accelerations. Besides, two potentiometers (DTB1 and DTB2 in Figure 5) were placed to monitor any story transversal displacement. At the same time, a range of additional sensors were installed to monitor the joint rotations, the column deformations along the height of the expected plastic hinge zone, and the diagonal elongation of both stories, as illustrated in Figure 5. Furthermore, twelve strain gauges



were employed to measure strains induced in the selected column, as shown in Figure 5. The axial force and deformation of BRB devices were also monitored during the tests (DTA9 and DTA10 in Figure 5).

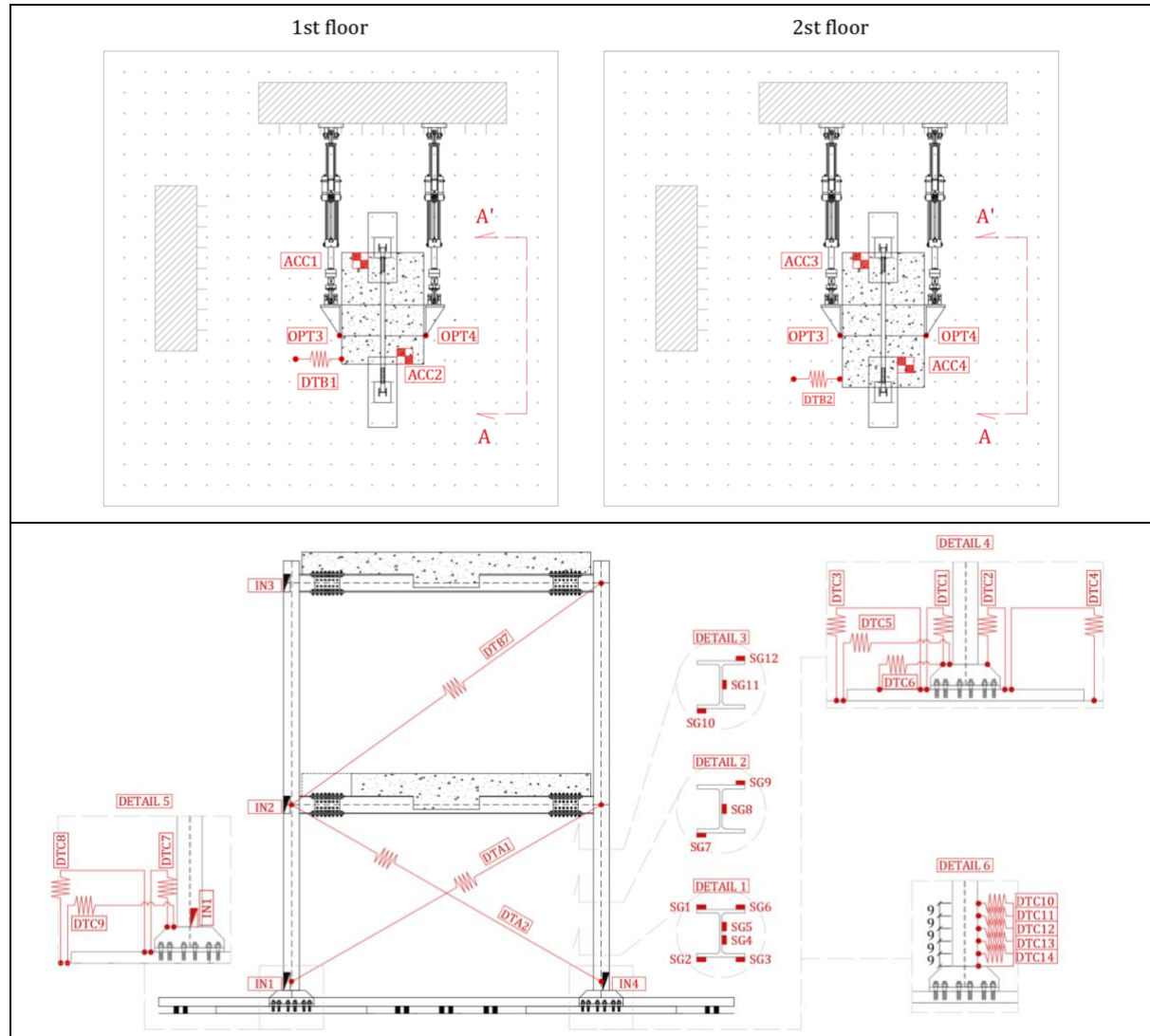


Figure 5. Location of the sensors for monitoring the response of the steel frame.

#### Ground motion record selection

The ground motions record utilized for the PsD tests was the East-West component of the 24<sup>th</sup> August 2016 Central Italy earthquake recorded at the Station in Norcia, Italy (NRC). Such strong motion was considered representative of areas with moderate- to high seismicity in Southern Europe. Table 4 reports the main information of the selected record, including the moment magnitude ( $M_w$ ), epicentral distance ( $R_{epi}$ ), and peak ground acceleration (PGA). The ground motion record was obtained from the Engineering Strong-Motion database (ESM) [4]. The earthquake time history was scaled in time by a factor of 0.87 according to the similitude scaling in Table 1. Figure 6(a) shows the accelerogram of the ground motion scaled in time, while Figure 6(b) and Figure 6(c) show its response spectra.

Table 4. Information of the selected ground motion record for the Pseudo-Dynamic Hybrid tests.

| Date                         | $M_w$ [ - ] | $R_{epi}$ [ km ] | PGA [ g ] | ID <sup>a</sup>         |
|------------------------------|-------------|------------------|-----------|-------------------------|
| 24 <sup>th</sup> August 2016 | 6.0         | 15.3             | 0.35      | EMSC - 20160824_0000006 |

<sup>a</sup>Station in Norcia, Italy (NRC) - East-West component of the ground motion. Source: <https://esm.mi.ingv.it/>

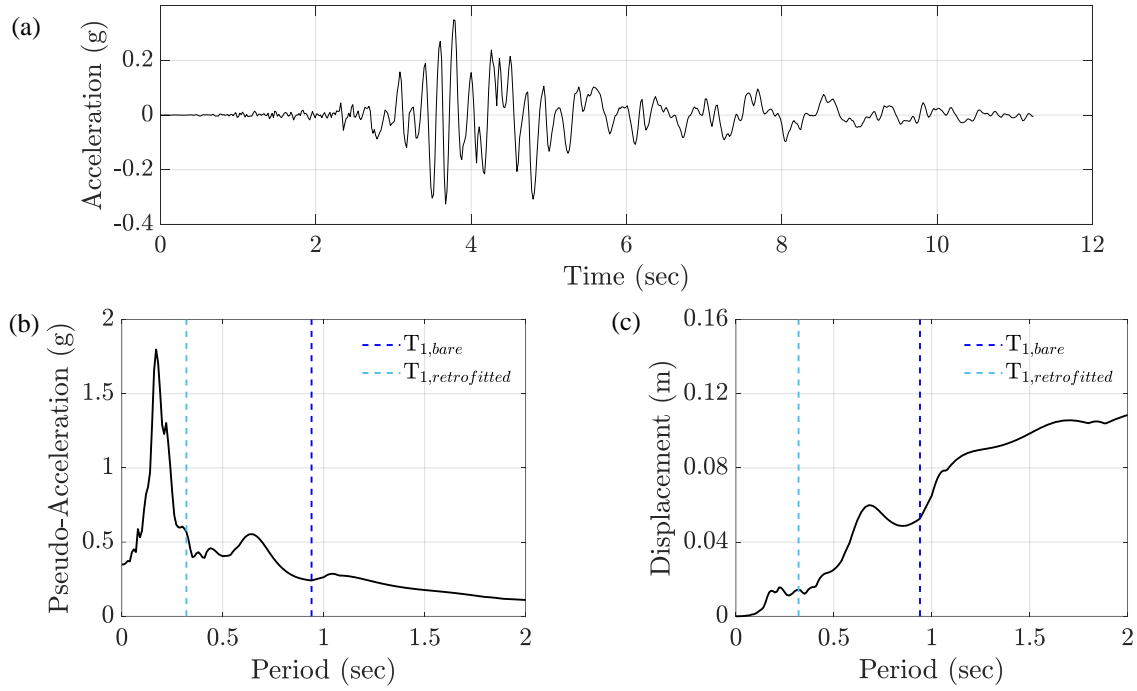


Figure 6. Selected ground motion record: (a) accelerogram; (b) acceleration response spectrum and natural periods of the scaled structure; (c) displacement response spectrum.

#### Implementation of the hybrid simulation

In the hybrid simulation study, the test specimen was the central bay of the prototype structure, which is retrofitted with BRBs. In contrast, the lateral bays, not retrofitted with BRBs, were considered as the numerical substructure. OpenSees was used for modeling the lateral bays. Figure 7 illustrates how the numerical and physical substructures were integrated in the hybrid simulation. At each time step of the test, the response of the target structure subject to the input ground motion was calculated through numerical integration, including the displacements at the interface between the numerical and physical substructures. The alpha-operator splitting method was used for the numerical integration. The interface between the numerical and physical substructures was assumed to be pin-connected, thereby transmitting no bending moment through the interface. Thus, the interface was modeled with hinge nodes in the OpenSees model. The calculated displacements at the interface were imposed on the test specimen by controlling the four actuators mounted on it. Subsequently, the restoring force of the test specimen was measured from the actuators, which was fed back to the OpenSees model to calculate displacements for the next time step. This process was repeated throughout the hybrid simulation.

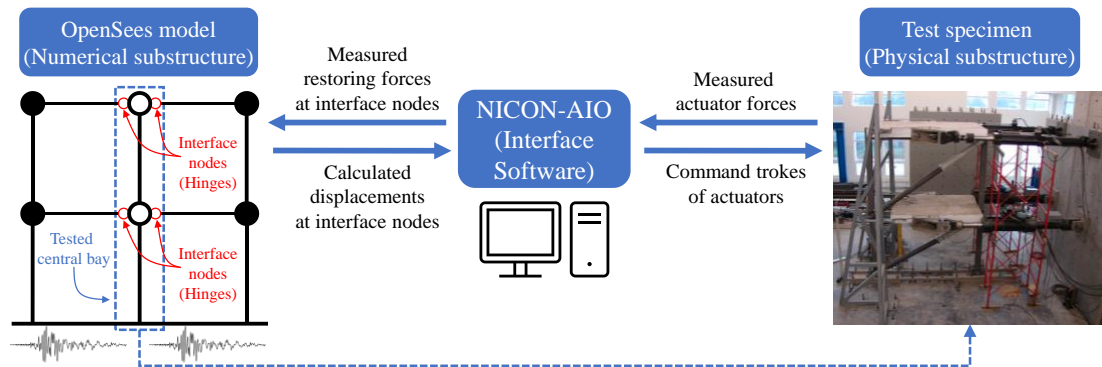


Figure 7. Integration of the numerical and physical substructures in the hybrid simulation using NICON-AIO.

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