17th World Conference on Earthquake Engineering, 17WCEE
Sendai, Japan - September 13th to 18th 2020

EXPERIMENTAL STUDY OF BRACED STEEL FRAMES SUBJECTED TO FIRE AFTER EARTHQUAKE

M. Lamperti Tornaghi[(1)], G. Tsionis[(2)], P. Pegon[(3)], J. Molina[(4)], M. Peroni[(5)], M. Korzen[(6)], N. Tondini[(7)], P. Covi[(8)], G. Abbiati[(9)], M. Antonelli[(10)], B. Gilardi[(11)]

[(1)] *Laboratory Technical agent, European Commission, Joint Research Centre, Ispra, Italy, marco.lamperti-tornaghi@ec.europa.eu*
[(2)] *Project officer, European Commission, Joint Research Centre, Ispra, Italy, georgios.tsionis@ec.europa.eu*
[(3)] *ELSA laboratory responsible, European Commission, Joint Research Centre, Ispra, Italy, pierre.pegon@ec.europa.eu*
[(4)] *Project officer, European Commission, Joint Research Centre, Ispra, Italy, francisco-javier.molina@ec.europa.eu*
[(5)] *Project officer, European Commission, Joint Research Centre, Ispra, Italy, marco.peroni@ec.europa.eu*
[(6)] *Deputy Head of division fire engineering, Bundesanstalt für Materialforschung und –prüfung (BAM), manfred.korzen@bam.de*
[(7)] *Assistant professor, Dept. of Civil, Environmental and Mech. Engineering, University of Trento, Italy, nicola.tondini@unitn.it*
[(8)] *PhD student, Dept. of Civil, Environmental and Mechanical Engineering, University of Trento, Italy, patrick.covi@unitn.it*
[(9)] *Assistant Professor, Aarhus University, Denmark, abbiati@eng.au.dk*
[(10)] *Paint & Sprays Business Manager, Etex Building Performance S.p.A., Belgium marco.antonelli@etexgroup.com*
[(11)] *Technical service manager, Xella Italia S.r.l., Italy, barbara.gilardi@xella.com*

Abstract

The EQUFIRE project aims to study the post-earthquake fire performance of steel frame structures and is part of the Transnational Access activities of the SERA project (www.sera-eu.org) at the ELSA Reaction Wall of the European Commission - Joint Research Centre. As it has happened in many historical occasions, after an earthquake, earthquake-induced rupture of gas piping, failure of electrical systems, etc. may trigger fire. The structural fire performance can deteriorate because the fire acts on a previously damaged structure. In addition, the earthquake may have damaged fire protection elements and the fire can spread more rapidly if compartmentation walls have failed. This is particularly relevant for steel structures as the high thermal conductivity of elements with small thickness entails quick temperature rise with consequent fast loss of strength and stiffness.

EQUFIRE studied a four-storey three-bay steel frame with concentric bracings in the central bay. The structure was designed for reference peak ground acceleration equal to 0.186g, soil type B and type 1 elastic response spectrum according to Eurocode 8. Tests were performed at the ELSA Reaction Wall and at the furnace of the Federal Institute for Materials Research and Testing (BAM).

The experimental activities at the ELSA Reaction Wall comprise pseudo-dynamic tests on a full-scale specimen of the first storey of the building, while the upper three storeys are numerically simulated. The aim is to study the response of the structure and fire protection elements, including their interaction, under the design earthquake and for different configurations: bare frame without fire protection, specimen with three fire protection solutions (conventional and seismic-resistant boards, and vermiculite sprayed coating) applied on the bracing and one column, and with conventional and seismic-resistant fire barrier walls built in the two external bays of the specimen. The testing programme at BAM included fire tests of five columns (two specimens without fire protection elements and three specimens with the types of fire protection mentioned above). Before the fire test, each column was subjected to a horizontal and vertical displacement history resulting from the seismic action. During the fire tests, the effect of the surrounding structure was simulated by limiting the axial thermal expansion.

The experimental results will serve to study the response of structural and non-structural components to fire following earthquake scenarios, with a view to improving existing design guidelines and future standards.

Keywords: fire following earthquake; concentrically braced steel frames; large-scale tests; pseudo-dynamic testing; sub-structuring

1. Introduction

Many historical events (e.g. the 1096 San Francisco, 1923 Tokyo, 1995 Kobe, 1999 Turkey, 2011 Tohoku and 2011 Christchurch earthquakes) have shown that, after an earthquake, fire may be triggered by earthquake-induced rupture of gas piping, failure of electrical systems, etc. The structural fire performance can then deteriorate because the fire acts on a previously damaged structure. In addition, the earthquake may have damaged fire protection elements and the fire can spread more rapidly if compartmentation walls have failed. This is particularly relevant for steel structures as the high thermal conductivity of elements with small thickness entails quick temperature rise with consequent fast loss of strength and stiffness.

The effects of seismic and fire actions have been traditionally studied separately because: i) the inherent issues related to each action are quite complex per se; ii) researchers and practitioners are typically specialised in one particular field; iii) experimental facilities have been conceived to reproduce one of the two actions; iv) full-scale tests are very expensive and feasible in very few facilities; v) there is lack of numerical codes capable of performing fire following earthquake (FFE) analysis at low computational cost.

Most of the works in literature involve numerical simulations on steel moment resisting frames [1][2][3][4] and only a few of them are dedicated to buckling-restrained and conventional brace systems, e.g. [5][6] that developed a framework for evaluating the post-earthquake performance of steel structures in a multi-hazard context that incorporates tools that are capable of probabilistic structural analyses under fire and seismic loads. Experimental studies have been performed on single elements [7], beam-concrete joints made of filled steel tubes [8], and full-scale reinforced concrete frames [9][9]. The study of literature reveals that several numerical studies on the post-earthquake fire behaviour of structural components have been carried out without being supported by comprehensive experimental research. Moreover, works on non-structural components are also very limited.

Therefore, the EQUFIRE project aimed to provide experimental data to study the post-earthquake fire performance of steel frame structures. It was part of the Transnational Access activities of the SERA project (www.sera-eu.org) at the ELSA Reaction Wall of the European Commission - Joint Research Centre. The project studied a steel frame building with concentric bracings by seismic pseudo-dynamic tests of a real-scale one-storey frame at the ELSA Reaction Wall and tests of single elements subjected to fire following earthquake at the furnace of the Federal Institute for Materials Research and Testing (BAM). The experimental results will serve to study the response of structural and non-structural components, and the interaction with different fire protection systems, to scenarios of fire following earthquake, with a view to providing sound experimental evidence and knowledge for improving existing design guidelines and future standards.

2. Design of the test specimen

A four-storey three-bay steel frame with concentric bracings in the central bay was selected for the experimental campaign. This frame is part of an office building with a square plan (12.5 m x 12.5 m) and it is located in Lisbon (Portugal) in an area of medium-high seismicity. The storey height is 3 m with the exception of the first floor, which is 3.6 m high, so that it can be tested at full scale in the laboratory. The lateral force resisting system consists of concentric braced frames (CBF) placed on the perimeter and at the centre of the building (see Figure 1). Figure 1 also shows the member sizes and the location of the test frame. Figure 2(a) shows the magnitude of the gravity loads acting simultaneously with the seismic load, and the fire load considered at one column adjacent to the diagonals of the bracing.

In detail, IPE sections with the weak axis in the plane of the frame were used for the bracing elements to force in-plane buckling. This choice was made for essentially two reasons: i) to avoid damage in the walls where the bracing is inserted in; ii) to keep a 2D modelling of the frame meaningful so that to maintain low computational demand for the hybrid tests.

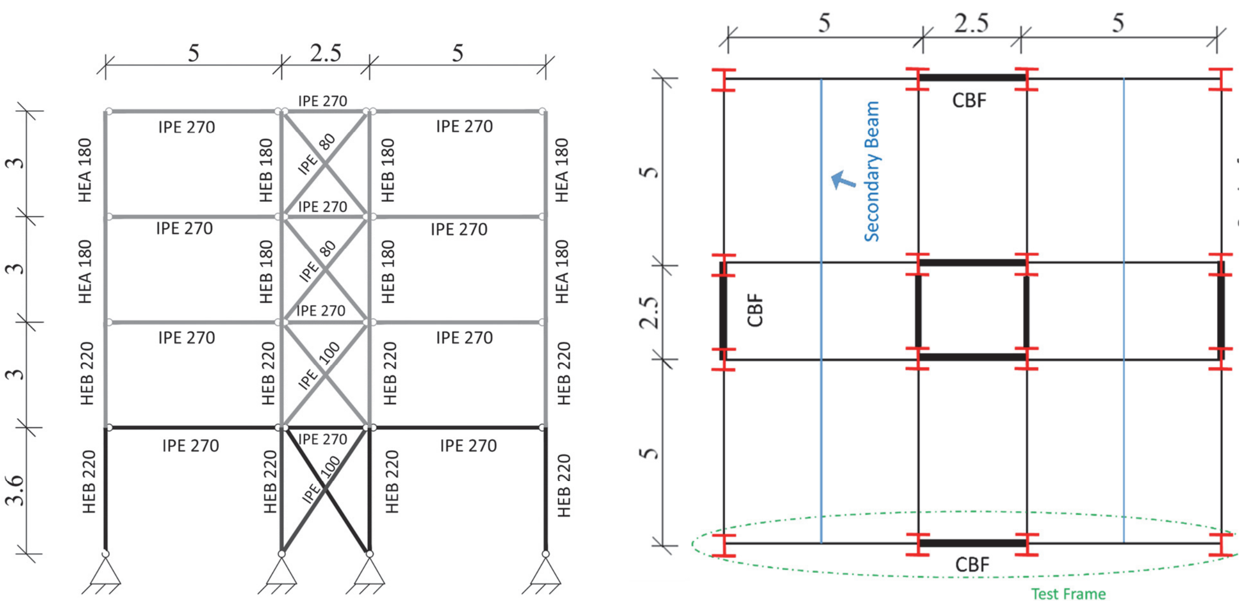
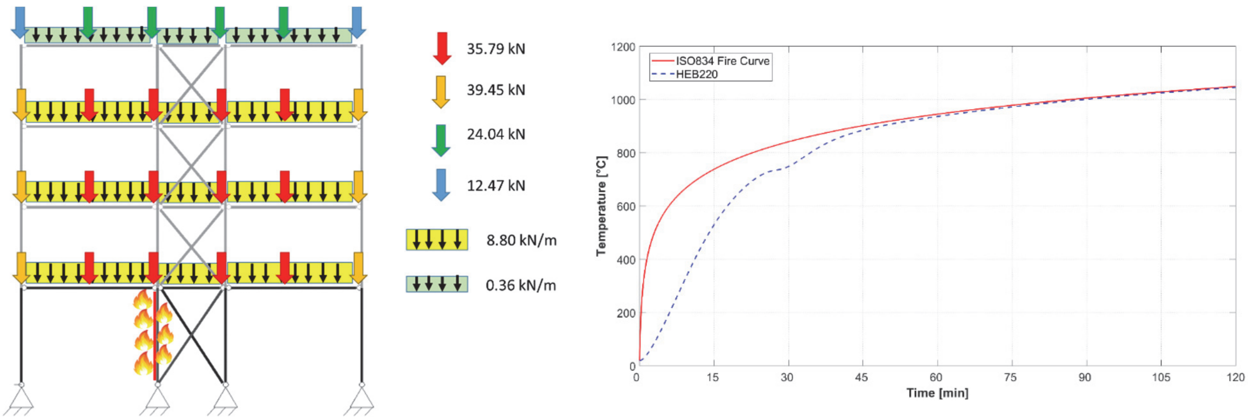


Fig. 1 – Elevation and plan of the building (dimensions in meters)



(a) (b)

Fig. 2 – Live and fire loads (a) and time-temperature ISO 834 heating curves and temperature of the HEB220 column (b)

According to the Eurocode 8[12], the frame was designed for seismic resistance following the capacity design criterion and a nonlinear dynamic 3D analysis was employed. In particular, a concept of "High Ductility Class (DCH)" was exploited with dissipation in the bracing members. Thus, the general modelling assumptions were the following:

- Only the bracing diagonal in tension was modelled at the ultimate (life safety) limit state.
- The columns were considered continuous along the height of the structure.
- All connections of the beams and diagonals were assumed pinned.
- Masses were considered lumped at the floors, following the assumption of rigid diaphragms.
- The building was regular in plan and in elevation.
- The building was located in Lisbon with a soil type B.

Seismic hazard maps of Portugal were used to identify the peak ground acceleration (PGA). Therefore, a reference PGA of 0.186 g, type B soil and a type 1 elastic spectrum were assumed for the design of the structure and then all the structural verifications described by Eurocode 8 [12] were carried out.

For the fire load, a prescriptive approach was chosen and in particular, the standard ISO 834 [13] heating curve was employed. This choice was made because it is easily reproducible in a furnace. In this respect, Figure 2 illustrates the temperature evolution with time of the gas and the steel temperature of an unprotected HEB 220 column at the base of the test specimen, calculated according to a lumped mass approach.

A set of fifteen accelerograms was selected considering the type of spectrum, type of earthquake scenario (magnitude range, distance range, style-of-faulting), local site conditions, period range, and ground motion components using the INGV/EPOS/ORFEUS European Strong motion Database [14]. Accelerograms were modified to match the target spectrum in the period range of 0.4÷0.9s that includes the fundamental period of the structure. The accelerograms were used to perform nonlinear time-history analyses and fire following earthquake analyses. A 2D and 3D model of the building was created using OpenSees [15], SAFIR [16] and ABAQUS [17] software to conduct seismic, fire and FFE numerical analyses of the braced steel frame.

The accelerogram shown in Figure 3 was selected among fifteen for the experimental hybrid tests and the numerical analyses, based on three main requirements:

- The selected accelerogram had to cause significant damage to the bracing elements.
- The horizontal displacement of the first floor had to be equal or lower than ± 30 mm to be compatible with the horizontal actuator stroke inside the BAM furnace.
- The axial force of the internal columns at the base of the second floor had to be below 1000 kN to be compatible with the actuators used to impose the vertical loads on the specimen at the ELSA Reaction Wall.

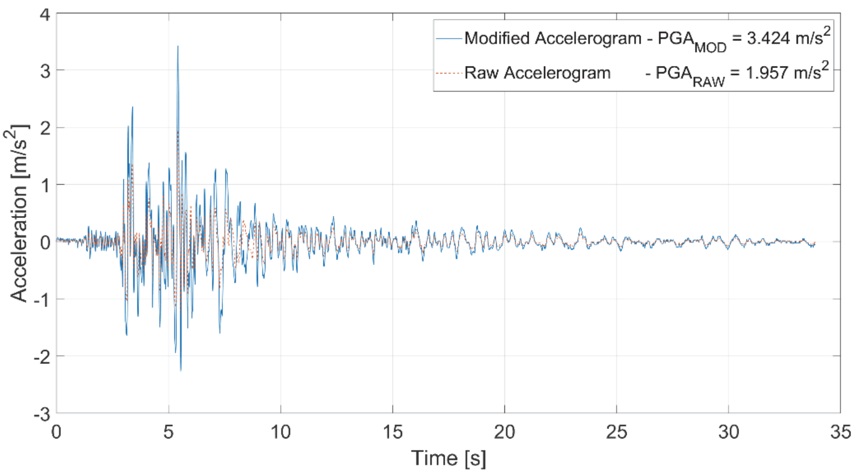


Fig. 3 – Earthquake acceleration time-history

A non-linear finite element model in OpenSees was used to evaluate the FFE response of the structure, which is expected to experience large displacements and plastic deformations in the bracing elements during the seismic action and non-linear behaviour of the column under fire condition. Fifteen non-linear thermomechanical beam elements, endowed with material and geometric nonlinearities, were used for the column subjected to fire action. The elastic-plastic constitutive law provided by Eurocode 3 [18] was adopted to model the mechanical properties of steel at elevated temperatures. Temperature dependency of elastic modulus, yield strength and strain proportional limit was accounted for according to Eurocode 3 [19]. Seven non-linear beam elements based on corotational formulation and the uniaxial Giuffre-Menegotto-Pinto steel material, with isotropic strain hardening (Steel02Material) [20] and geometric nonlinearities was used for the bracing diagonals. Non-linear beam elements were used for all elements to check that non-dissipative elements remain in the elastic field according to the design calculation.

As an example, Figure 4 illustrates the results of the numerical simulation of the FFE test on the bare structure (without fire protection) for the selected acceleration time-history followed by the ISO 834 [12] heating curve. As is possible to observe, the energy dissipation was concentrated in the braces and in particular at the ground floor. The internal columns and all the other elements remained in the elastic field during the seismic event. Collapse occurred 24 minutes after the start of the fire. Figure 4 also shows the final deformed configuration of the steel frame at the end of the simulation.

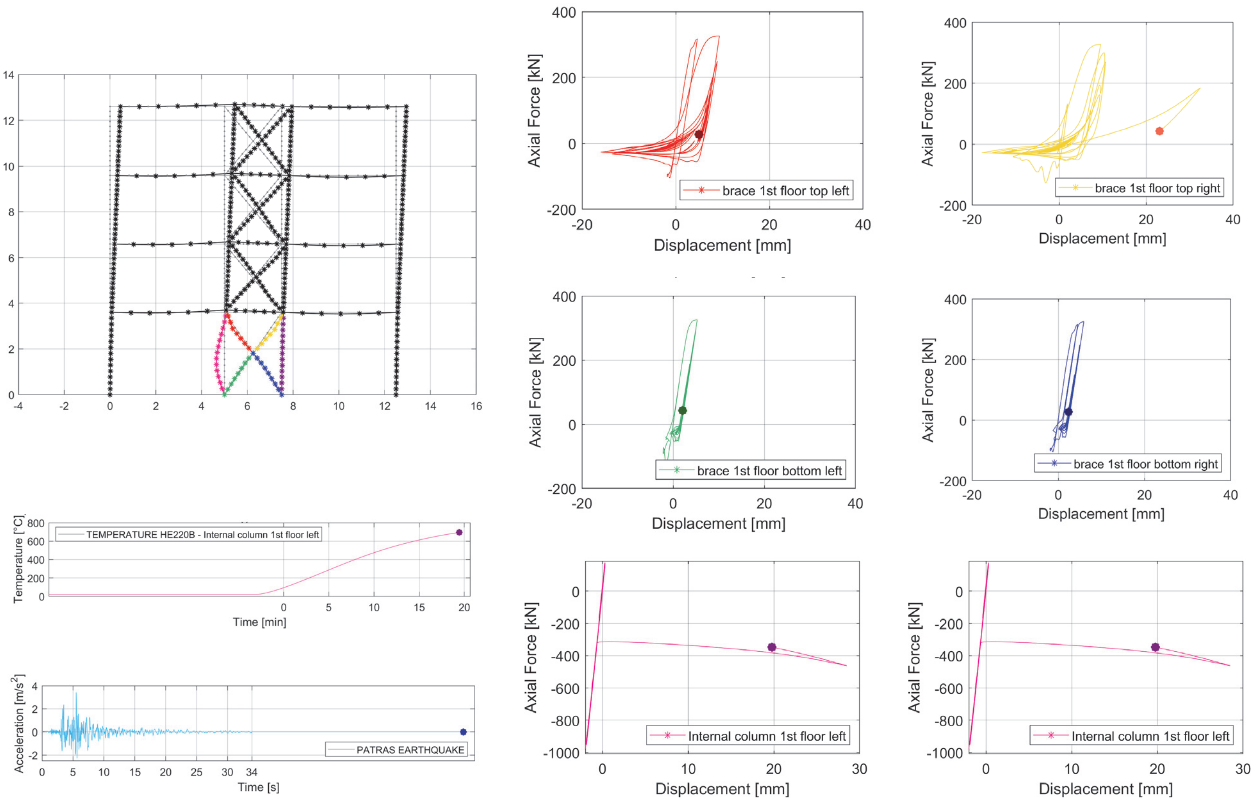


Fig. 4 – Numerical simulation of fire following earthquake with OpenSees

3. Experimental programme and setup

3.1 The experimental setup at BAM

The experimental tests at BAM were performed using a sub-structuring technique as shown in Figure 5, in which the physical column was heated by the standard ISO 834 curve and a constant numerical axial stiffness representative of the surrounding structure was applied as boundary condition at the top of the physical column. During the FFE tests, the axial force of the column was measured and then used to obtain the effect, in terms of displacement, of the rest of the structure. Those displacements were imposed on the column in order to keep the two substructures in mechanical equilibrium.

Five FFE tests were conducted at BAM:

- Test #0 Column E: without fire protection system;
- Test #1 Column A: without fire protection system;
- Test #2 Column B: fire protection system, PROMATECT-H, designed for seismic region;
- Test #3 Column C: fire protection system, PROMATECT-H, not designed for seismic region;

- Test #4 Column D: sprayed vermiculite spayed coating, designed for applications in seismic region. A mechanical reinforcing mesh retains the sprayed coating. It is located in the middle of the overall coating thickness.

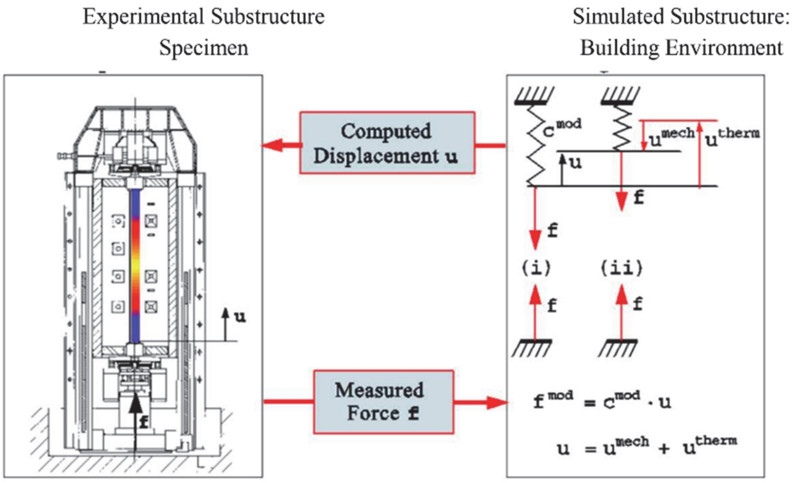


Fig. 5 – Sub-structuring method implemented for the tests at BAM [21]

The specimens were instrumented with thermocouples to measure temperatures at different positions of the cross-section and along the height. In detail, each specimen was equipped with 20 sheath thermocouples: six sensors in each section (2 on the web and 4 on the flanges) at three different heights along the column and additionally one sensor at top and base of the column. Six plate thermometers according to EN 1363-1 [22] measured the temperatures inside the furnace. Additionally, two other thermocouples measured the ambient temperature of the laboratory.

Each test was conducted as follows: the column was first subjected to horizontal and vertical displacement histories resulting from seismic non-linear time-history analysis and then the furnace was switched on and the ISO 834 curve was followed with constant axial stiffness, representative of the surrounding structure, as boundary condition.

3.2 Simulation algorithm

In order to enable hybrid simulation with mixed force- and displacement-controlled degrees of freedom (DOFs), a specific simulation algorithm was developed. A pair of Lagrange multiplier vectors are introduced to enforce both horizontal and vertical displacement compatibility between the physical substructure (PS) and the numerical one (NS). The NS is characterized by a dynamic balance equation that is solved with the Newmark-α method:

$$\begin{aligned}\mathbf{M}^N\ddot{\mathbf{u}}_{k+1}^N + \left[(1+\alpha)\mathbf{r}_{k+1}^N(\mathbf{u}_{k+1}^N) - \alpha\mathbf{r}_k^N(\mathbf{u}_k^N)\right] - \left[(1+\alpha)\mathbf{f}_{k+1}^N - \alpha\mathbf{f}_k^N\right] \\ - \left[(1+\alpha)\mathbf{L}^{N,v^T}\boldsymbol{\lambda}_{k+1}^v - \alpha\mathbf{L}^{N,v^T}\boldsymbol{\lambda}_k^v\right] - \left[(1+\alpha)\mathbf{L}^{N,h^T}\boldsymbol{\lambda}_{k+1}^h - \alpha\mathbf{L}^{N,h^T}\boldsymbol{\lambda}_k^h\right] = \mathbf{0}\end{aligned} \tag{1}$$

In Equation (1) $\mathbf{K}^N$ and $\mathbf{M}^N$ are the stiffness and mass matrices of the NS, whereas$\mathbf{u}^N$, $\mathbf{r}^N$ and $\mathbf{f}^N$ are the displacement, restoring force and external load vectors. Boolean matrices $\mathbf{L}^{N,v}$ and $\mathbf{L}^{N,h}$ locate interface Lagrange multipliers corresponding to vertical and horizontal interface DOFs respectively. Finally, parameter α modulates algorithmic damping. Accordingly, the PS is characterized by two static balance equations; one refers to the vertical force-controlled DOFs whereas the other refers to the horizontal displacement-controlled DOFs.

$$\begin{cases} \mathbf{r}_{k+1}^{P,h}\left(\mathbf{u}_{k+1}^{P,h}, \mathbf{r}_{k+1}^{P,v}\right) - \mathbf{L}^{P,h^T}\boldsymbol{\lambda}_{k+1}^{h} = \mathbf{0} \\ \mathbf{r}_{k+1}^{P,v} - \mathbf{L}^{P,v^T}\boldsymbol{\lambda}_{k+1}^{v} = \mathbf{0} \end{cases} \tag{2}$$

Superscripts h and v are used to distinguish between vertical and horizontal DOFs. $\mathbf{u}^{P,v}$ and $\mathbf{u}^{P,h}$ are vertical and horizontal displacement vectors of the PS, while $\mathbf{r}^{P,v}$ and $\mathbf{r}^{P,h}$ are the corresponding restoring force vectors. Boolean matrices $\mathbf{L}^{P,v}$ and $\mathbf{L}^{P,v}$ locate interface Lagrange multipliers corresponding to vertical and horizontal DOFs, respectively. The two following equations define the compatibility between NS and PS.

$$\begin{cases} \mathbf{L}^{N,h}\mathbf{u}_{k+1}^{N} + \mathbf{L}^{P,h}\mathbf{u}_{k+1}^{P,h} = \mathbf{0} \\ \mathbf{L}^{N,v}\mathbf{u}_{k+1}^{N} + \mathbf{L}^{P,v}\mathbf{u}_{k+1}^{P,v}\left(\mathbf{u}_{k+1}^{P,h}, \mathbf{r}_{k+1}^{P,v}\right) = \mathbf{0} \end{cases} \tag{3}$$

The solution to the system of equations is computed via operator splitting, which means with a single Newton iteration and a constant Jacobian. The procedure was verified considering the linear partitioned finite-element model of the frame reported in [23].

As can be observed in Figure 6, only 3 DOFs are coupled between PS and NS. In fact, a master-slave relation is imposed on all horizontal DOFs of the first story to follow DOF (10,1).

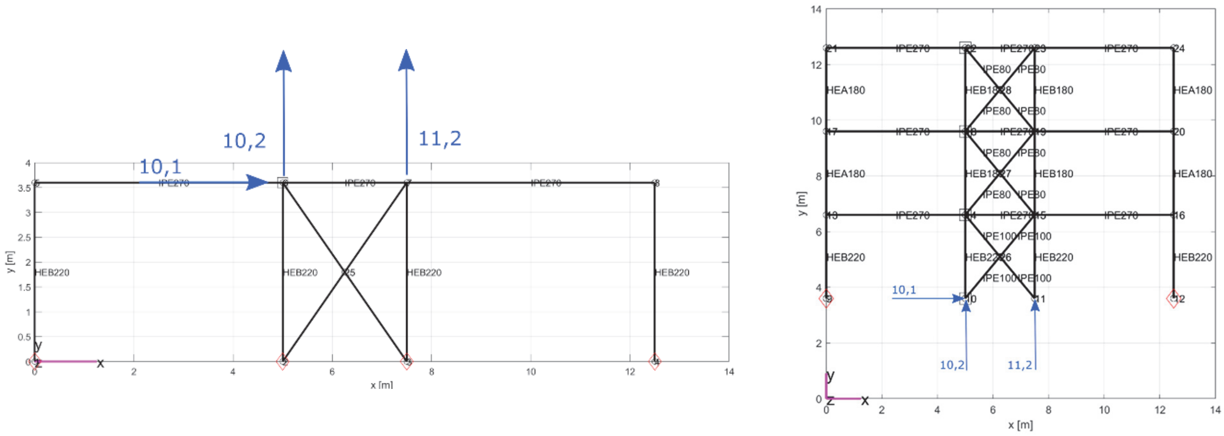


Fig. 6 – Partitioned model of the frame: a) PS and b) NS

Since external columns are connected to the braced frame by means of truss elements, their base vertical displacement at their base is blocked on the NS. Figure 7 provides a comparison between a reference solution computed with a monolithic FE model and the solution obtained with the proposed procedure. As can be appreciated, the dynamic response predicted by the proposed scheme reproduces the reference solution.

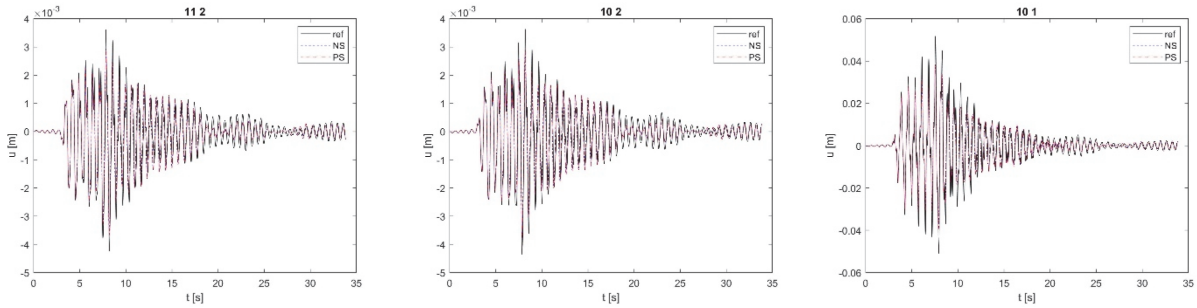


Fig. 7 – Comparison of displacement response histories of coupled DOFs: a) horizontal (10,1), b) vertical left (10,2), c) vertical right (11,2)

4. Experimental results

The results of the experimental tests at BAM are summarized in the following. The response history of the unprotected steel frame computed with OpenSees was verified against the results of the experimental tests at BAM. These results will be used for further calibration of the numerical model and for comparison with successive hybrid tests at JRC.

Figure 8 shows the input accelerogram, the deformed shape of the frame at the end of the simulation, and the comparison of the seismic test and numerical simulations in terms of horizontal displacement, axial displacement and axial force for all tested columns and the deformed shape of the frame. The comparison demonstrates good agreement. There is a little difference in negative vertical displacement, because the vertical actuator of the furnace is not designed to apply tension forces to the specimen.

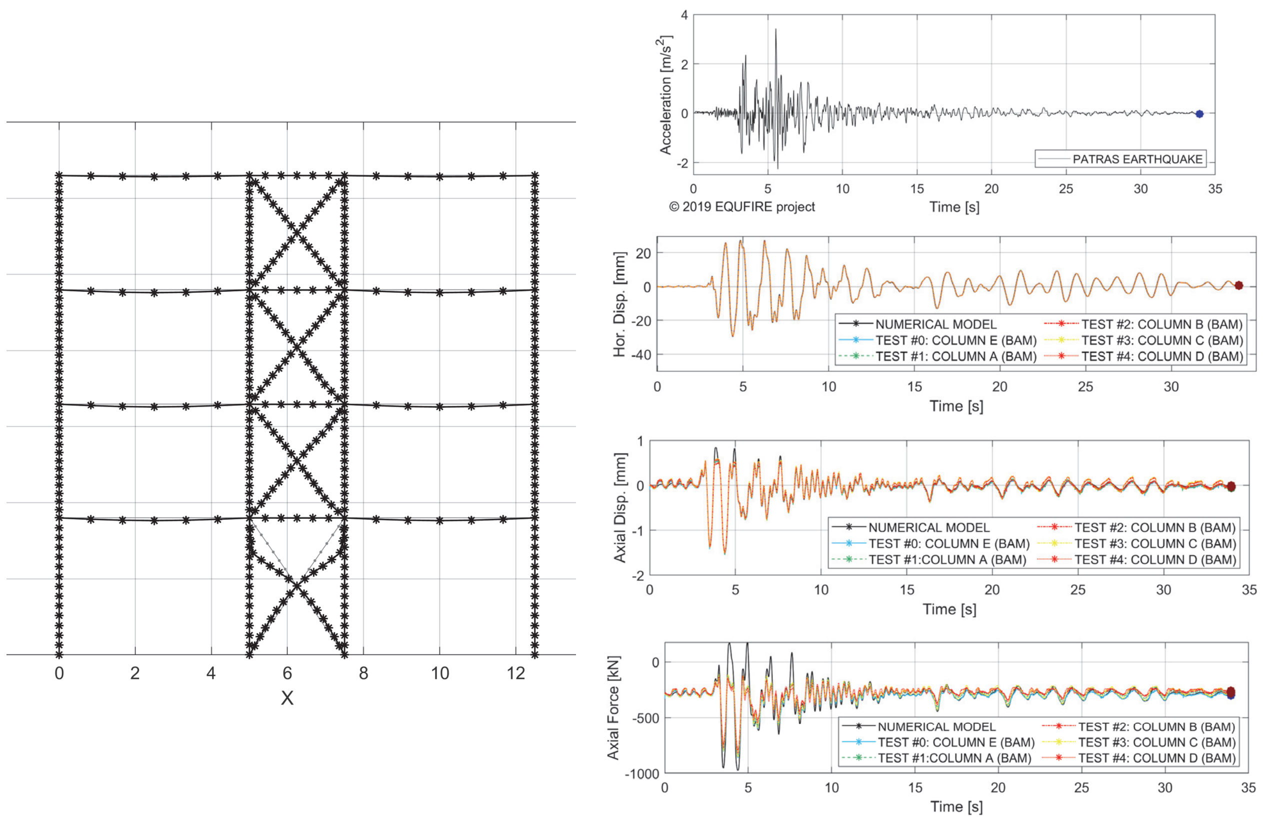


Fig. 8 – Comparison between the numerical model and the seismic tests

Figure 9 shows the comparison of the experimental and numerical (before and after the calibration) results for the FFE tests on the unprotected columns A and E, in terms of the evolution of the mean temperature, axial displacement and axial force. The calibration consisted in modelling the base of the columns with its actual initial stiffness and applying the recorded temperature evolution in the columns. The comparison demonstrated the good repeatability of the FFE test procedures. Figure 9 shows also a snapshot of the test.

Figure 10 shows the comparison of the evolution of the mean temperature, axial displacement and axial force of the FFE tests on the specimens with three fire protection solutions (conventional boards, seismic-resistant boards and vermiculite sprayed coating). Test #2 was interrupted due to a malfunctioning of a component and then was restarted. The specimens with the two different types of fire protection boards (columns B and C) showed similar performance. The sprayed vermiculite-type fire protection delayed the development of the temperature in column D with respect to columns B and C. Figure 10 shows also a snapshot of the test.

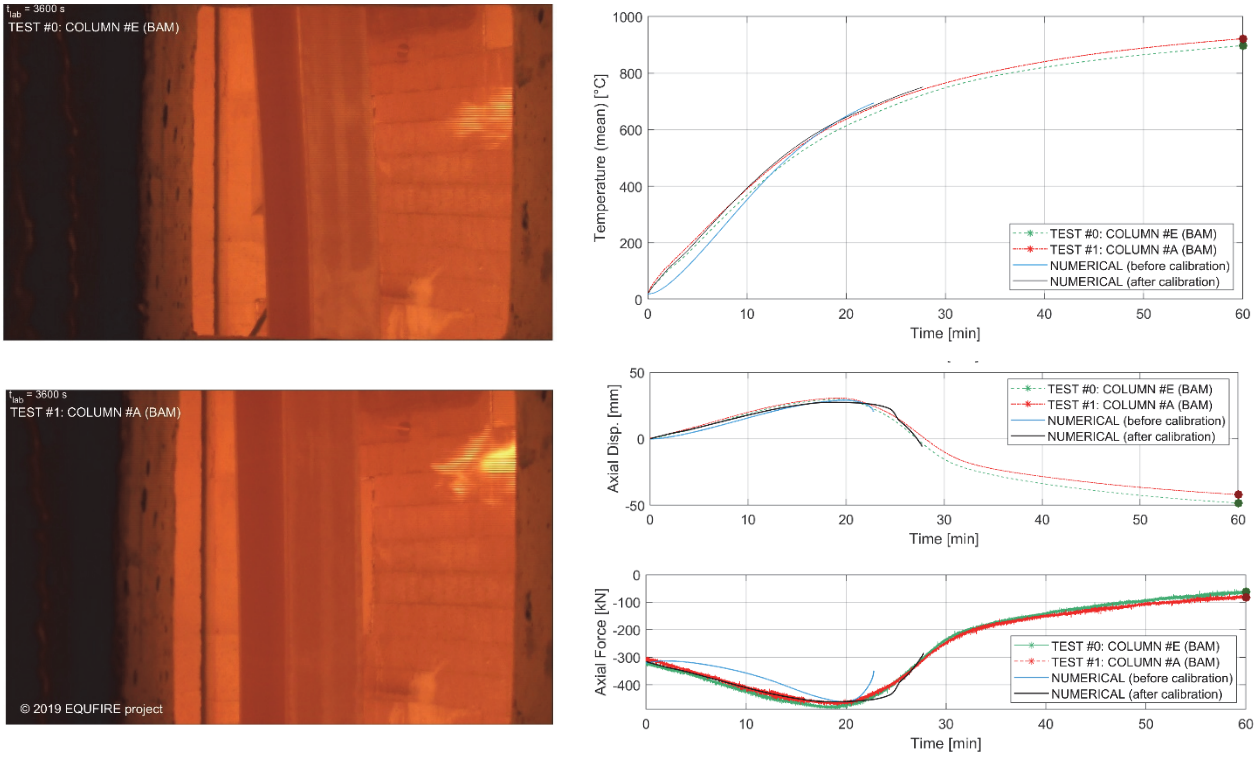


Fig. 9 – Comparison between the results of the numerical model and the FFE tests on the unprotected columns

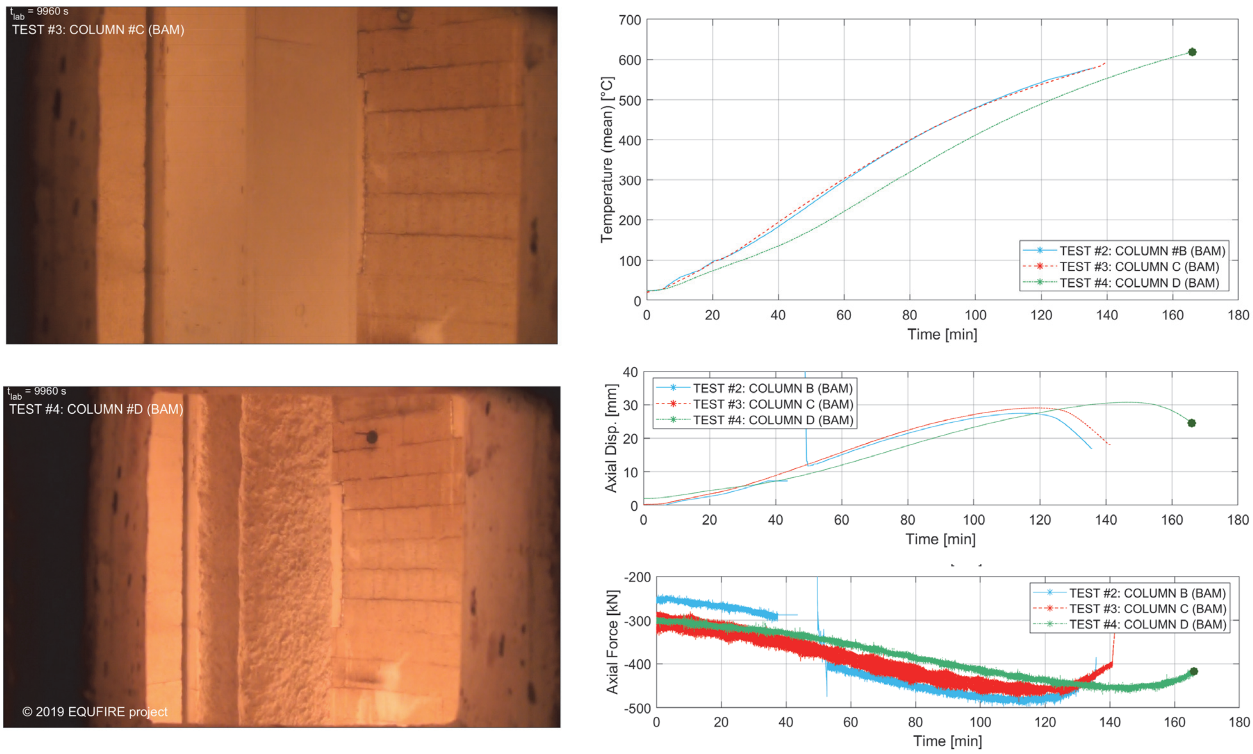


Fig. 10 – Mean specimen temperature, axial displacement and axial force of the protected for FFE tests of the protected columns

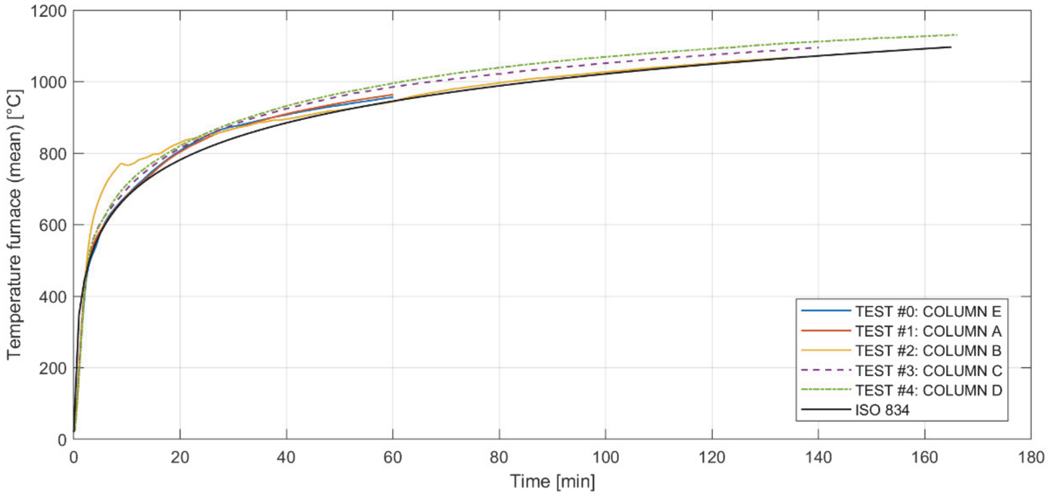
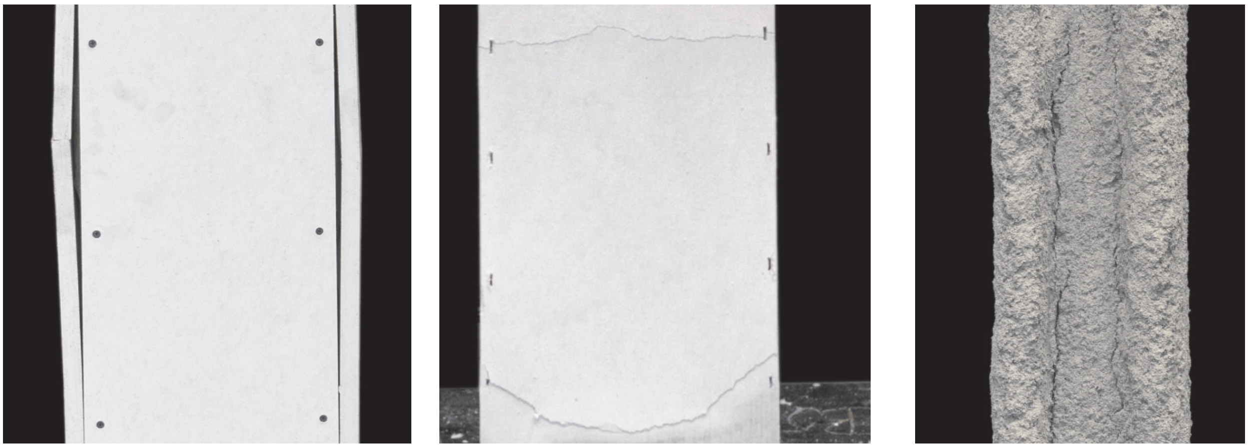


Fig. 11 – Comparison between the heating curves of the furnace and the ISO 834 curve

As shown in Figure 12, cracks on the fire protection elements developed due to the combination of seismic and fire actions. However, those cracks were not large enough to compromise the fire resistance of the columns. This was mainly due to the fact that the column is not a dissipative element and to the laboratory limitations in applying horizontal displacements.

(a) (b) (c)

Fig. 12 – Damage on the fire protection elements due to the combination of seismic and fire actions: a) test#2 column B; b) test#3 column C; c) test#4 column D



(a) (b) (c)

Fig. 13 – Damage on the fire protection elements due to the combination of seismic and fire actions: a) test#2 column B; b) test#3 column C; c) test#4 column D

5. Main outcomes and discussion

The paper presents the experimental study currently underway in the EQUFIRE project. It focuses on a three-bay, four-storey steel frame with concentric bracing in the central span, subjected to fire following an earthquake, with the aim to study the performance of structural and non-structural components. Five preliminary FFE substructure tests have already been carried out at the BAM laboratory in Berlin. The design of the entire experimental campaign and the preliminary results of the sub-structured column tests are presented in this document.

The comparison between the numerical analyses and the experimental results for the unprotected columns demonstrate a good agreement for both seismic and fire actions. The proposed simulation algorithm proved a good agreement in comparison with a reference solution calculated with a monolithic FE model. The following full-scale physical experiments will further confirm its reliability.

Tests for fire after earthquake were carried out on two unprotected steel columns and three columns with different fire protection solutions: conventional and earthquake-proof panels and vermiculite sprayed coating. In terms of fire protection, no serious damage was observed that would undermine the fire resistance of the columns. A more significant damage to the fire protection of the dissipative elements (bracing system) is expected during the FFE test series at the ELSA Reaction Wall of the Joint Research Centre. Indeed, testing a complete bracing system, including dissipative braces, should reproduce the actual earthquake conditions, where the compressed bracing is expected to cause more severe damage to the fire protection elements due to buckling under large horizontal displacement.

6. Acknowledgements

This project has received funding from the European Union's Horizon 2020 Research and Innovation Programme, under grant agreement No. 730900. Authors would also acknowledge Philippe Buchet (JRC) as well as Sven Riemer and Kai-Uwe Ziener (BAM) for their invaluable contribution to the experimental campaign.

7. References

[1] Della Corte G, Landolfo R, Mazzolani FM (2003): Post earthquake fire resistance of moment resisting steel frames. *Fire Safety Journal*, **38** (7), 593-612.

[2] Memari M, Mahmoud M, Ellingwood B (2014): Post-earthquake fire performance of moment resisting frames with reduced beam section connections. *Journal of Constructional Steel Research*, **103**, 215-229.

[3] Behnam B, Ronagh HR (2015): Post-earthquake fire performance-based behavior of unprotected moment resisting 2D steel frames. *KSCE Journal of Civil Engineering* **19** (1), 274-284.

[4] Keller WJ, Pessiki SP (2012): Effect of earthquake-induced damage to spray-applied fire-resistive insulation on the response of steel moment-frame beam-column connections during fire exposure. *Journal of Fire Protection Engineering* **22** (4), 271-299.

[5] Talebi E, Tahir M, Zahmatkesh F, Kueh A, Said A (2016): Fire resistance of a damaged building employing buckling restrained braced system. *International Journal of Advanced Steel Construction*, **14** (1), 1-21.

[6] Khorasani NE, Garlock MEM, Quiel SE (2015): Modeling steel structures in OpenSees: Enhancements for fire and multi-hazard probabilistic analyses. *Computers and Structures*, **157**, 218-231.

[7] Braxtan NL, Pessiki SP (2011): Postearthquake fire performance of sprayed fire-resistive material on steel moment frames. *Journal of Structural Engineering*, **137** (9), 946-953.

[8] Pucinotti R, Bursi OS, Demonceau JF (2011): Post-earthquake fire and seismic performance of welded steel–concrete composite beam-to-column joints. *Journal of Constructional Steel Research*, **67**, 1358-1375.

[9] Kamath P, Sharma UK, Kumar V, Bhargava P, Usmani A, Singh B, Singh Y, Torero J, Gillie M, Pankaj P (2015): Full-scale fire test on an earthquake-damaged reinforced concrete frame. *Fire Safety Journal*, **73**, 1-19.

[10] Meacham BJ (2106): Post-earthquake fire performance of buildings: Summary of a large-scale experiment and conceptual framework for integrated performance-based seismic and fire design. *Fire Technology*, **52** (4), 1133-1157.

[11] CEN (2019): *EN 10025-2, Hot rolled products of structural steels. Technical delivery conditions for non-alloy structural steels*. European Committee for Standardization.

[12] CEN (2004): *EN 1998-1 Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings*. European Committee for Standardization.

[13] ISO (1999): *ISO 834-1:1999, Fire-resistance tests -Elements of building construction- Part 1: General requirements*. International Organization for Standardization.

[14] Luzi L, Puglia R, Russo E, ORFEUS WG5 (2016): *Engineering Strong Motion Database, version 1.0*. Istituto Nazionale di Geofisica e Vulcanologia, Observatories & Research Facilities for European Seismology

[15] McKenna F (2011): OpenSees: A framework for earthquake engineering simulation. *Computing in Science and Engineering*, **13**.

[16] Franssen, J-M, Gernay, T. (2017): Modeling structures in fire with safirR©: Theoretical background and capabilities. *Journal of Structural Fire Engineering*, **8** (3), 300-323.

[17] ABAQUS (2011): *ABAQUS Documentation*, Dassault Systèmes.

[18] CEN (2005): *EN 1993-1-2 Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings*. European Committee for Standardization.

[19] CEN (2005): *EN 1993-1-2 Eurocode 3: Design of steel structures - Part 1-2: General rules - Structural fire design*. European Committee for Standardization.

[20] Silva V, Crowley H, Varum H, Pinho R (2015): Seismic risk assessment for mainland Portugal. *Bulletin of Earthquake Engineering*, **13**, 429-457.

[21] Korzen M, Magonette G, Buchet P. (1999): Mechanical loading of columns in fire tests by means of the substructuring method, *8th INTERFLAM '99: Fire science and engineering conference*.

[22] CEN (2012): *EN 1363-1:1999 Fire Resistance Tests-Part 1: General Requirements*. European Committee for Standardization.

[23] Filippou FC, Popov EP, Bertero VV (1983): *Effects of Bond Deterioration on Hysteretic Behavior of Reinforced Concrete Joints*, Report EERC 83-19. Earthquake Engineering Research Center, University of California, Berkeley.

