COMPORTAMENTO SISMICO DI UN TELAIO CONTROVENTATO IN ACCIAIO DOTATO DI PROTEZIONI PASSIVE AL FUOCO

SEISMIC BEHAVIOUR OF A STEEL BRACED FRAME EQUIPPED WITH PASSIVE FIRE PROTECTIONS

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ABSTRACT

The paper describes the experimental behaviour of a braced steel frame equipped with passive fire protections subjected to the seismic action as a part of the European EQUFIRE project. 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. Experimental tests were performed at the ELSA Reaction Wall of the Joint Research Centre (JRC) according to different configurations. In fact, bare elements without fire protection and specimens with three fire protection solutions, i.e. conventional and seismic-resistant boards as well as vermiculite spray, were considered. The experimental tests were performed by means of the pseudodynamic technique on a full-scale specimen physically representing the first storey of the building, while the upper three storeys were numerically simulated. The damage of passive fire protections applied on the bracing and one column are reported in order to give insight into a fire following earthquake scenario.

SOMMARIO

L'articolo descrive i risultati sperimentali di un telaio in acciaio controventato dotato di protezioni antincendio passive soggette all'azione sismica nell'ambito del progetto europeo EQUIFIRE. La struttura è stata progettata per un picco di accelerazione del suolo di riferimento pari a 0,186 g, tipologia di terreno B e spettro di risposta elastico di tipo 1 secondo l'Eurocodice 8. Sono state eseguite prove sperimentali sul telaio, equipaggiato con differenti configurazioni di protezione antincendio, presso il muro di reazione del laboratorio ELSA del Joint Research Centre (JRC). Sono stati infatti presi in considerazione elementi strutturali senza protezione antincendio e elementi strutturali con tre soluzioni di protezione antincendio, quali protezioni scatolari con lastre convenzionali e antisismiche oltre a spray di vermiculite. Le prove sperimentali sono state eseguite mediante la tecnica pseudodinamica su un telaio in scala reale corrispondente al primo piano dell'edificio, mentre i tre piani superiori sono stati simulati numericamente. Nell'articolo vengono riportati i danni delle protezioni antincendio applicati sui controventi e su una colonna a seguito di uno scenario di incendio post sisma.

1 INTRODUCTION

Earthquakes are destructive and unpredictable events with catastrophic consequences for both people and built environment. Moreover, secondary triggered effects can strike further an already weakened community, i.e. ground shaking, surface faults, landslides and tsunamis. In this respect, also fires following earthquake (FFE) have historically produced large post-earthquake damage and losses in terms of lives, buildings and economic costs, like the San Francisco earthquake (1906), the Kobe earthquake (1995), the Turkey earthquake (2011), the Tohoku earthquake (2011) and the Christ-church earthquakes (2011) [1][2][3]. In detail, FFE are a considerable threat as they can be wide-spread both at the building level and at the regional level within the seismic affected area owing to the rupture of gas lines, failure of electrical systems etc. and at the same time failure of the compart-mentation measures. Moreover, they are more difficult to tackle by the fire brigades because of their possible large number and extent as well as of possible disruptions within the infrastructural net-work that hinder their timely intervention and within the water supply system [1]. In this context, the development of a resilience plan is recommended to increase understanding of the impacts and the serious consequences of disruptions and failures in order to mitigate hazards, contain the effects of disasters and improve the strategic response to earthquakes/fires when they occur [4].

In a fire following earthquake scenario, the structural fire performance can worsen significantly because the fire acts on an already damaged structure. Furthermore, passive and active fire protections may have also been damaged by the seismic action and the fire can spread more rapidly if compartmentation measures have failed [4][5]. Thus, the seismic performance of the non-structural components may directly affect the fire performance of the structural members. As consequence, the minimization of the non-structural damage is paramount in mitigating the possible drop in structural fire performance. The loss of fire protection is particularly dangerous for steel structures because the high thermal conductivity associated with small profile thicknesses entails quick temperature rise in the profiles with consequent fast loss of strength and stiffness. Most of the literature involve numerical simulations on steel moment resisting frames [6][7][8][9] and only a few of them are dedicated to buckling-restrained and conventional brace systems [10]. Both developed a framework for evaluating the post-earthquake performance of steel structures in a multi-hazard context that in-corporates tools that are capable of probabilistic structural analyses under fire and seismic loads. Experimental studies have been performed on single elements [11], full-scale reinforced concrete frames [12] and full-scale hybrid FFE tests on steel braced frame (EQUFIRE project) [13][14]. The EQUFIRE project focuses on the analysis of the behaviour of a braced steel structure subjected to FFE through full-scale tests based on hybrid simulation at the ELSA Reaction Wall at Joint Research Centre (JRC) and through tests on single elements at the furnace of the Federal Institute for Materials Research and Testing (BAM).

2 CASE STUDY

A four-storey three-bay steel structure with concentric bracings in the central bay was selected as case study for the experimental campaign, as shown in Fig. **1**. This frame is part of an office building with a square plan (12.5 m x 12.5 m) and the location of the building was assumed to be in the city of Lisbon (Portugal); thus, in an area of medium-high seismicity. The interstorey height is 3.0 m except for the height of the first floor, which is equal to 3.6 m. The lateral force resisting system consists of concentrically braced frames (CBF). The frame is used in the experimental campaign of the EQUFIRE project as part of the Transnational Access activities of the SERA project (www.sera-eu.org) **Errore. L'origine riferimento non è stata trovata.Errore. L'origine riferimento non è stata trovata.**.

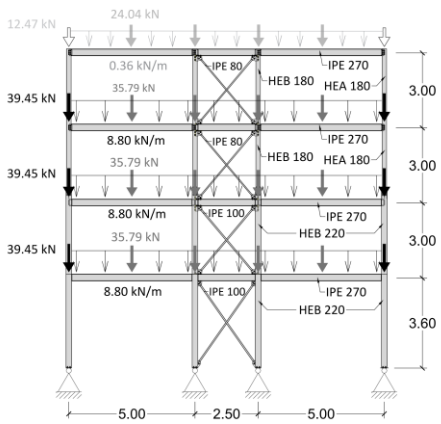


Fig. 1. Configuration of the frame (Dimensions are in meters).

The steel grade S275 (EN 10025-2, 2019) was adopted for the bracing system (dissipative elements), while steel grade S355 was selected for all the other non-dissipative elements (beams and columns). It is worth to point out that the yield strength for the bracings was taken as the mean value, i.e., 330 MPa, considering a coefficient of variation equal to 0.12 and using a lognormal distribution, whereas for the non-dissipative members was taken the experimental value obtained through material testing, i.e., 436 MPa **Errore. L'origine riferimento non è stata trovata.Errore. L'origine riferimento non è stata trovata.**.

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 so that to avoid possible damage to the walls where the bracing is inserted in.

The frame was designed according the Eurocode 8 **Errore. L'origine riferimento non è stata trovata.** using the capacity design criterion by employing a response-spectrum analysis (RSA). In particular, a "High Ductility Class (DCH)" was employed with dissipation in the bracing members. Then, nonlinear time-history analyses with natural accelerograms were employed to investigate the seismic response of the structure. The general assumptions were the following:

- All connections were assumed as pinned.

- The columns were considered continuous along the height of the structure.
- The building was regular in plan and in elevation.
- The building was located in Lisbon (Portugal) characterized by peak ground acceleration equal to 0.186 g and type B soil.

In order to perform non-linear time-history analyses, it was fundamental to model the seismic hazard through adequate selection and scaling of ground motion records. In this respect, a set of fifteen accelerograms for the significant damage limit state (SD) was selected considering the type of spectrum, 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 [21]. The 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 equal to 0.67 s. Among the fifteen accelerograms, the one shown in Fig., known as the Patras earthquake was used in this section as an example for the FFE test. The earthquake occurred on July 14, 1993 with a magnitude of 5.4. This accelerogram was chosen in order to be the same accelerogram used during the all EQUFIRE experimental tests, based on three main requirements:

- The selected accelerogram had to cause significant damage to the bracings.
- The horizontal displacement of the first floor had to be equal or lower than ± 30 mm to be compatible with the horizontal actuator stroke of the BAM furnace.
- The axial force of the internal columns at the beginning 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.

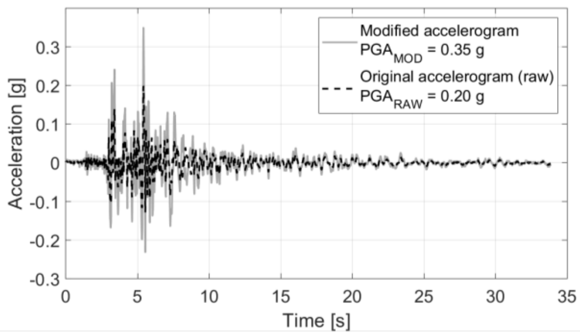
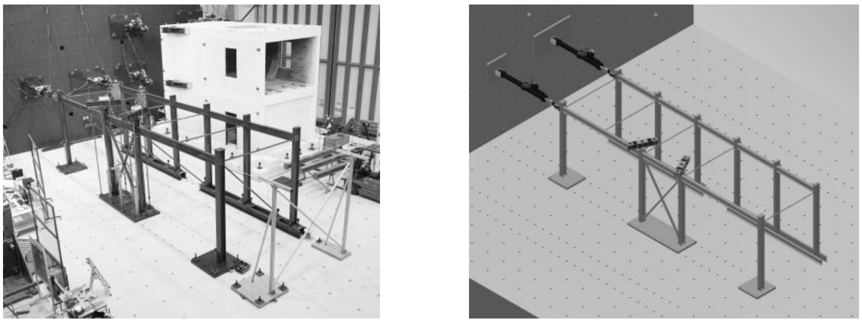


Fig. 2. Comparison between the original and modified accelerogram for the FFE simulation

3 EXPERIMENTAL PROGRAMME AND SETUP

The experimental mock-up at ELSA Reaction Wall (Fig. 3) represents the ground floor of the four-storey frame. A secondary frame, parallel to the main one is used to prevent out-of-plane deformation during the test. The two frames are fixed to the strong floor and tare-connected together by steel rods, which do not alter the seismic response of the mock-up.

Two 600 kN hydraulic jacks apply the vertical load on each internal column, whereas lateral loads are applied through 500 kN actuators connected to the ELSA Reaction Wall, one for the main frame and another for the secondary frame. To further reduce any possible interference from the secondary frame and eliminate any relative displacement during testing, the actuator of the safety frame applies the same displacement as the actuator of the main one. Load cells measure the loads applied by the vertical and horizontal actuators. Since the frame is statically indeterminate, the two central columns and the three beams are equipped with strain gauges that measure their internal axial loads. Displacement transducers measure the vertical deformation of the central columns, the axial deformation of the braces, as well as the lateral displacement of the whole frame.



(a) (b)

Fig. 3. General view of the specimen and experimental setup at the ELSA Reaction Wall

Four different configurations were tested:

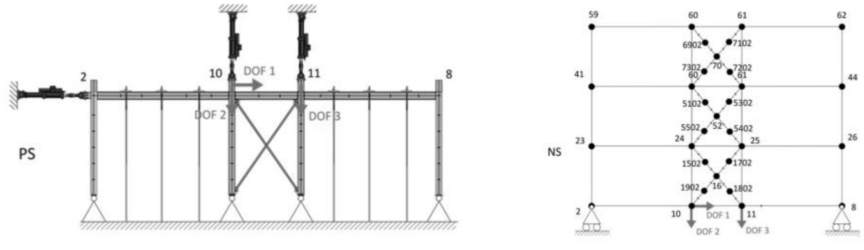
- Frame A: bare frame without fire protection;
- Frame B: conventional PROMATECT-H fire protection boards (not designed for seismic regions) on the brace;
- Frame C: PROMATECT-H fire protection boards designed for seismic regions on the brace;
- Frame D: PROMASPRAY P300 series, vermiculite wet mix spray-applied fire resistive material, designed for applications in seismic regions, on the brace and fire walls with and without seismic design.

The testing programme comprised three pseudodynamic tests (using the acceleration time history shown in Figure 3 and pga = 0.35g) and two cyclic tests:

- Test 1: pseudodynamic test (pga = 0.35g) on Frame A;
- Test 2: pseudodynamic test (pga = 0.35g) on Frame B;
- Test 3: pseudodynamic test (pga = 0.35g) on Frame C;
- Test 4: cyclic test (three cycles at ± 3.0 cm horizontal displacement) on Frame C;
- Test 5: pseudodynamic test (pga = 0.35g) on Frame D;
- Test 6: cyclic test (one cycle at ± 2.5 cm horizontal displacement, two cycles at ± 3.0 cm and two cycles at ± 3.5 cm) on Frame D.

Each pseudodynamic test was repeated twice. The scope was to damage the bracing without damaging further the structure (remaining more or less in the ± 3cm range of horizontal displacement), in a kind of large aftershock test. Since the horizontal displacement was below this range for Test 3, the cyclic Test 4 was performed on configuration 3. For the same reason, a final cyclic “funeral test” (test 6) was performed on Frame D comprising walls. The maximum horizontal displacement for test 6 was ± 3.5 cm, because it was hard to see damage in the walls at ± 3.0 cm. The results after the second run of the pseudodynamic tests and cyclic tests are presented in this paper.
The bracing system and the column on which the fire protection has been applied were replaced at the end of each of the Tests 2, 3, 4 and 5.

As can be observed in Fig. 4, only three DOFs are coupled between the physical and numerical substructures. In fact, a master-slave relation is imposed on all horizontal DOFs of the first storey to follow DOF (DOF 1). Since external columns are connected to the braced frame by means of truss elements, the vertical displacement at their base is blocked on the numerical substructure.

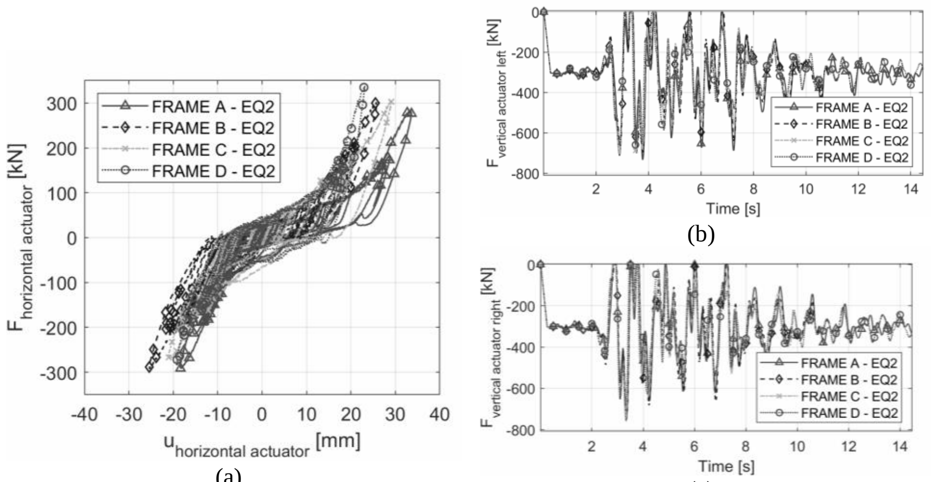


(a) (b)

Fig. 4. Substructuring scheme adopted for the pseudodynamic tests: (a) Physical substructure; (b) Numerical substructure.

4 EXPERIMENTAL RESULTS

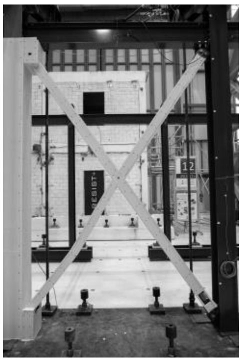
The results of the experimental tests at JRC are summarized in the following. Fig. 5 presents the force-displacement curves and vertical forces for the different frame tests performed at ELSA. The bare frame and the specimens with the fire protection boards with and without seismic design showed similar response for the design earthquake. The specimen of Frame C dissipated more energy than the ones of Frame A and B. This could be attributed to the metallic supports of the fire protection boards with seismic design. As expected, the specimen with sprayed fire protection and walls showed smaller displacement, i.e. higher stiffness, and higher resistance than the other configurations.



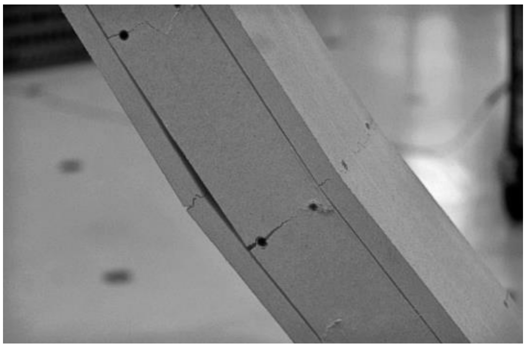
(a) (b) (c)

Fig. 5. (a) Force displacement curves of the 2nd earthquake Tests; (b) Vertical displacement of left actuator (DOF 2); (c) Vertical displacement of right actuator (DOF 3).

No significant damage was observed after Test 1 on the bare frame. Small cracks were observed on the fire protection boards during Tests 2 and 3 (Fig. 6a and Fig. 6b), which are not expected to affect their fire performance. Buckling of the braces was particularly observed during the repeated cycles at ± 3.5 cm horizontal displacement. This was observed in Test 4, that caused also some damage on the fire protection elements. Also, the specimen with sprayed fire protection and walls did not exhibit significant damage during the seismic Test 5. During the cyclic Test 6 on Frame D and as shown in Figure 19, the braces buckled, a diagonal crack passing through the masonry blocks and the mortar joints developed at the lower corner of the wall without seismic design, and the walls were detached from the columns and the floor. The most visible damage after tests 4 and 6 (Fig. 6c and Fig. 6d) was on the braces.



(a)



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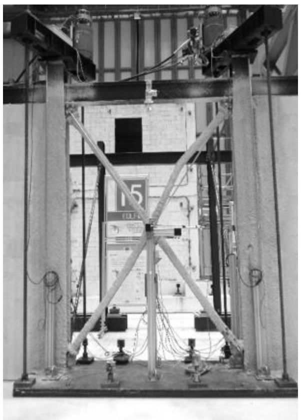

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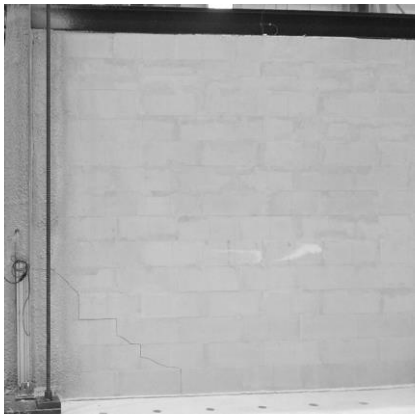

(d)

Fig. 6. Damages of the fire protection: (a) Frame B; (b) Frame C; (c) Frame D; (d) Frame D (wall).

CONCLUSIONS

Full-scale seismic testing of a steel braced building using pseudodynamic and was successfully performed at the ELSA Reaction Wall. The tests allowed to investigate the seismic response of different types of fire protection elements (boards with and without seismic design, sprayed fire protection, fire walls with and without seismic design) and their interaction with the structural elements. The four tested configurations showed similar response and no significant damage owing to the seismic loading. Then, tests conducted with a larger number of cycles at maximum amplitude caused buckling of the braces and damage on the fire protection elements. Future perspectives will be focused on performing seismic tests characterized by larger interstorey drifts.

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KEYWORDS
braced steel frame, fire protection, pseudodynamic, fire following earthquake, seismic behaviour.