# Experimental assessment of post-fire retrofitted RC columns tested under cyclic loading

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**Abstract:** Multiple combined hazards can affect the structures during their live span and may conditionate the future structural behaviour for some types of loading. That is the case of a structure previously damaged by fire and then loaded under seismic loading. For seismic hazard zones it is important assess the seismic performance of existing reinforced concrete (RC) structures designed according to old codes and without seismic detailing. This structural seismic assessment is even more important for buildings that were previously damaged by fire. Therefore, it is critical develop and validate fire retrofitting methods that can also improve the seismic behaviour. This paper presents the results of a novel experimental campaign carried out on four (two of them repaired and strengthened with CFRP wrapping after fire exposure) full-scale reinforced concrete columns previously damaged by a 30 or 90 minutes standard fire and then tested under uniaxial cyclic lateral loading up to failure. Moreover, two additional control columns, one as-built and another strengthened, were cyclically tested for comparison. A considerable decrease in the deformation capacity and dissipated energy was observed in the columns after fire exposure, even for the 30 minute fire. Moreover, the post-fire repaired and strengthened columns may reach similar seismic performance than analogous strengthened columns without previous fire damages.

**Keywords:** Existing RC columns, post-fire strengthening, post-fire cyclic loading, experimental cyclic tests

## 1. Introduction

Previous fire damages on structures located in a seismic hazard zone, may conditionate the seismic performance of the structures during an earthquake. The seismic assessment and retrofitting of RC structures previously damage by fire is even more important than the seismic assessment of non-damaged structures in seismic hazard zones once the structures may have larger vulnerability. Current design codes and guidance allow engineers to design for the primary performance drive, life safety (CEN 2002, CEN 2009, CEN 2014), and more recently for property protection (Standards 2019). However, there is little data on the damage of RC structure to fire, with qualitative assessments for damage (Society 2008), or quantitative assessments based on expert judgement (Ioannou, Aspinall et al. 2017, Rush and Lange 2017).

Several works have been developed on the post-earthquake fire behaviour of RC elements or structures (Behnam and Ronagh 2013, Kamath, Sharma et al. 2015, Shah, Sharma et al. 2016, Vitorino, Rodrigues et al. 2020) and have concluded that the structures previously damaged by the seismic loads and then exposed to a fire are more vulnerable than those that are not damaged. However, the post-fire seismic performance of RC elements or structures have been less studied (Ni and Birely 2018, Li, Liu et al. 2019, Demir, Goksu et al. 2020, Demir, Unal et al. 2021) and there are no guidelines for the engineers in this situation.

In the experimental study developed by (Li, Liu et al. 2019) on seismic performance of post-fired reinforced concrete frames it was possible conclude that the pinching effect was more remarkable for the post-fire RC frames than the corresponding unfired frames, higher stiffness degradation and considerably lower ductility and dissipated energy were observed in the post-fire specimens. Two works developed by (Demir, Goksu et al. 2020, Demir, Unal et al. 2021) about the time after fire of the post-fire seismic performance of RC columns and post-fire seismic behaviour of RC columns built with sustainable concrete have concluded that longer the time after fire higher is the stiffness of the columns due to the partial recover of the concrete properties (compressive and tensile strength and elastic modulus) and for the columns with sustainable concrete the fire expose did not significantly affected the ductility up to moderate fire, but for severe fire the ductility is dropped.

This paper presents the main results of cyclic tests performed on repaired and strengthened RC columns previously damaged by fire. The repair and strengthening techniques aim increase the concrete strength, previously affected by high temperature, the ductility and the energy dissipation capacity of the RC columns. More information regarding this experimental campaign can be find in (Melo, Triantafyllidis et al. 2022).

## 2. Details of the columns and experimental setups

### 2.1. Columns detailing and material properties

Six full-scale RC columns with the same square cross section and reinforcement detailing were built at the same time. The columns were designed according to the old Portuguese code (REBA 1967) and without any seismic and fire requirements. The geometry, cross-section details and location of thermocouples are presented in Fig.1. Each specimen represents a half-storey cantilever column of a 3.0m storey height, at foundation level, of a structure with three or four storeys. Despite the specimens have 1.65m length, the lateral load is applied at 1.5m from the top foundation. The columns have square cross-section with dimensions of 0.30x0.30m$^2$ and a stiff block with dimensions of 0.44x0.44x0.5m$^3$ simulate the foundation. The columns have eight 12mm longitudinal reinforcing bars (longitudinal reinforcement ratio of 1%) and stirrups of 6mm diameter spaced at 0.15m and with 90° anchorage hooks. The concrete cover is 25mm. The columns were casted together in a single phase and cured for at least 6 months at ambient laboratory temperature and relative humidity conditions, to reduce the risk of spalling during furnace testing.

The experimental campaign consisted in three different stages: i) fire tests; ii) repair and strengthening of the columns; iii) lateral cyclic loading test with constant axial load until failure. The six columns were distributed in two control columns (one as-built, C, and another similar strengthened column, C-S), two columns (M and M-S) were exposed to the 30 minutes fire duration (medium fire) and two more columns (L and L-S) were exposure to the 90 minutes fire duration (long fire). One column exposure to medium fire (M-S) and another exposure to long fire (L-S) were later repaired and strengthened before the cyclic tests. All the columns were cyclic tested under the same load protocol.

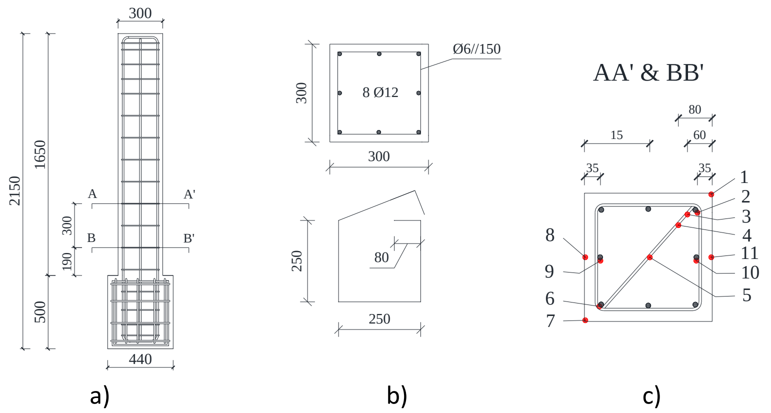



Fig. 1 - Geometry and reinforcement detailing: a) global dimensions; b) cross-section; c) location of thermocouples (dimensions in mm).

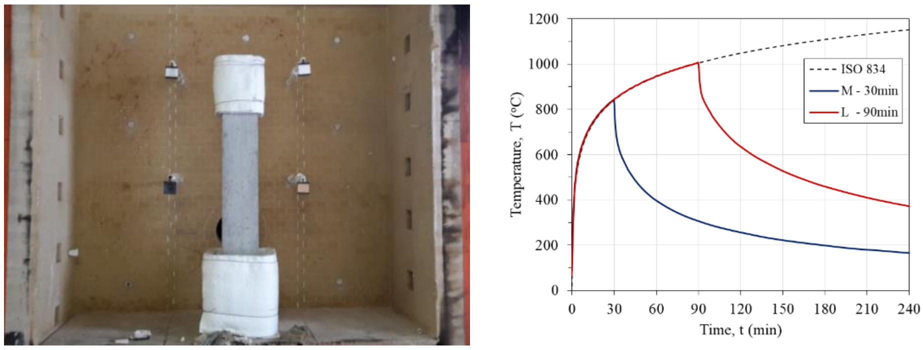
Table 1 summarises the mean values of the concrete and steel properties, where $f_{cm}$ is the concrete compressive strength of cylinder samples (Ø150mm × 300mm) according to the standard norm NP EN 206-1 (NP-EN206 2000), $f_{ym}$ is the yield strength of reinforcement, $f_{um}$ is the ultimate tensile strength of reinforcement and $\varepsilon_{cu}$ is the ultimate strain of reinforcement.

*Table 1. Mean values of the concrete and steel mechanical properties*

| Column | Fire | Strengthening | Cyclic test | Concrete | Steel – 12mm | | | Steel – 6mm | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | $f_{cm}$ (MPa) | $f_{ym}$ (MPa) | $f_{um}$ (MPa) | $\varepsilon_{cu}$ (%) | $f_{ym}$ (MPa) | $f_{um}$ (MPa) | $\varepsilon_{cu}$ (%) |
| C | | | Yes | 33.5 | 445 | 571 | 17.5 | 540 | 639 | 18 |
| C-S | | Yes | Yes | | | | | | | |
| M | 30 min | | Yes | | | | | | | |
| M-S | 30 min | Yes | Yes | | | | | | | |
| L | 90 min | | Yes | | | | | | | |
| L-S | 90 min | Yes | Yes | | | | | | | |

## 2.2. Fire exposure setup

The fire tests were performed using a vertical furnace with internal dimensions of 3.1 × 3.1 × 1.2 m$^3$ (h × w × d) located at the Structural and Fire Resistance Laboratory at Aveiro University, Portugal. The furnace can perform standard fire resistance tests on materials and construction elements according to the European Standards. Fig. 2-a shows the front view of the furnace. The columns were placed centrally in the furnace and tested individually under the respective fire conditions. The block foundation and the top column were protected from heat with a 50 mm thick layer of ceramic fibre blanket insulation (density of 128kg/m$^3$) to prevent fire damages where the column is restrained and the loads are applied during the cyclic test.




a) b)

Fig. 2 – a) Front view of the fire test setup; b) ISO 834 and imposed time-temperature curves.

The temperature evolution during the fire tests followed the ISO 834 (ISO 1999) standard fire curve for 30 minutes (medium fire) in columns M and M-S and 90 minutes (long fire) in columns L and L-S. Standard fire durations of 30 and 90 minute were selected in order to induce relatively light and severe fire damage to the column specimens in line with the fragility curves developed by (Ioannou, Aspinall et al. 2017). The ISO 834 and the imposed time-temperature curves are presented in Fig. 2-b. The peak mean gas phase temperatures measured by the furnace plate thermometers were 842°C and 1006°C (standard deviations of ±14ºC and ±8ºC) for the adopted medium and long fires, respectively. After the fire exposures reached the time set point (30 or 90 minutes), the propane burners were turned off and the furnace was allowed to cool naturally. In the cooling phase, the furnace was kept closed until the interior temperature dropped to at least 100°C. The cooling phase took 10 and 24 hours for the columns tested under medium and long fires, respectively.

### 2.3. Cyclic loading test setup

The cyclic tests were carried out in a rig available in the Structures Laboratory at Porto University for performing uniaxial and biaxial cyclic tests on reinforced concrete columns with constant or varying axial loads. The test rig includes a vertical actuator used to apply the axial compressive load and a horizontal actuator to apply the cyclic lateral displacements ($d_c$). The axial load ($N$) was set to a constant value of 410 kN. The lateral displacements ($d_c$) are imposed at 1.50 m from the foundation and each demand level cycle is repeated three times, with gradually increasing demand levels. The adopted lateral load path followed the nominal peak displacement levels of 3, 5, 10, 4, 12, 15, 7, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80 (in mm). In this test setup it is assumed that the P-Delta effects are neglected. More information on this test rig can be found at (Rodrigues, Arêde et al. 2013) and (Lucchini, Melo et al. 2022).

## 3. Fire tests results

### 3.1. Temperature evolution

The temperature evolution within the columns during the medium and long fire exposure and cooling stages are shown in Fig. 3 and Fig. 4, respectively. These show the envelope of all thermocouple measurements at each location for both cross-sections AA’ and BB’ (i.e. a set of eight readings for each of locations 1&7, 2&6, 8&11, and 9&10; and a set of four readings for locations 3, 4, and 5). The peak average temperature at the end of the 30 min heating phase was 772ºC (standard deviation ±41ºC) at the corner and 734ºC ±39ºC at surface centre of columns M and M-S. The respective temperatures for columns L and L-S at the end of the 90 min heating phase were 967ºC ±16ºC and 949ºC ± 15ºC. In the concrete, temperatures increased at a slower rate and continued to increase even after the decay phase of the fire exposure had started. Following the 30 min fire exposure, the centre of the concrete core reached a peak temperature of 153ºC ±12ºC after approximately 4 hours (280 minutes from start of the heating). In the case of the 90 min exposure, peak temperatures in the core reached 348ºC ±27ºC at 320 minutes from the start of the heating.

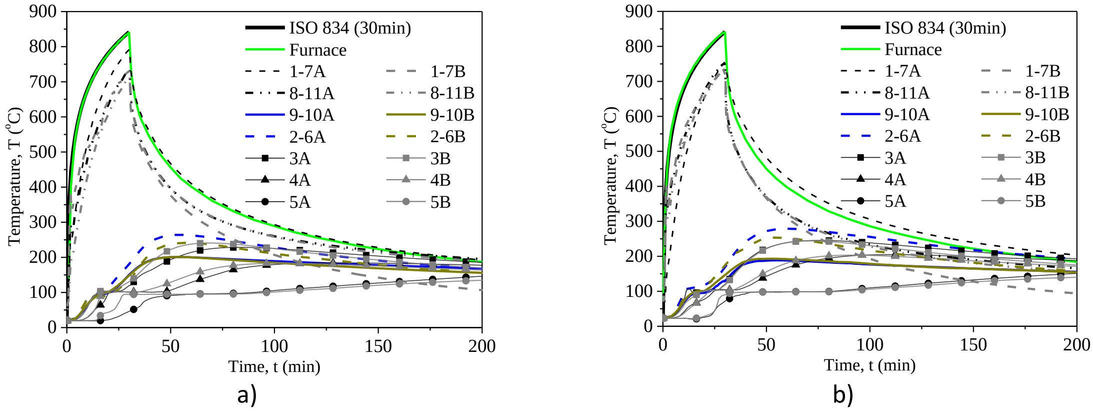



Fig. 3 – Temperature evolutions and average temperature evolutions: a) column M; b) column M-S.

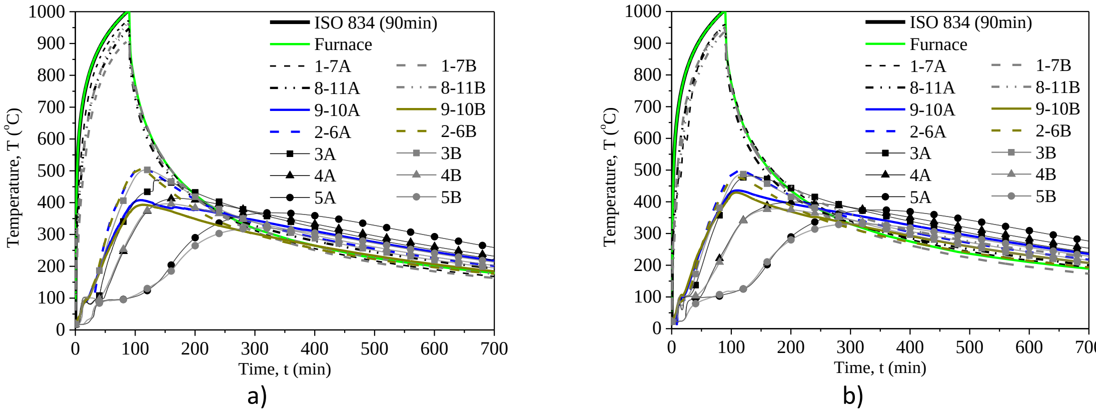



Fig. 4 – Temperature evolutions and average temperature evolutions: a) column L; b) column L-S.

## 3.2. Fire damage

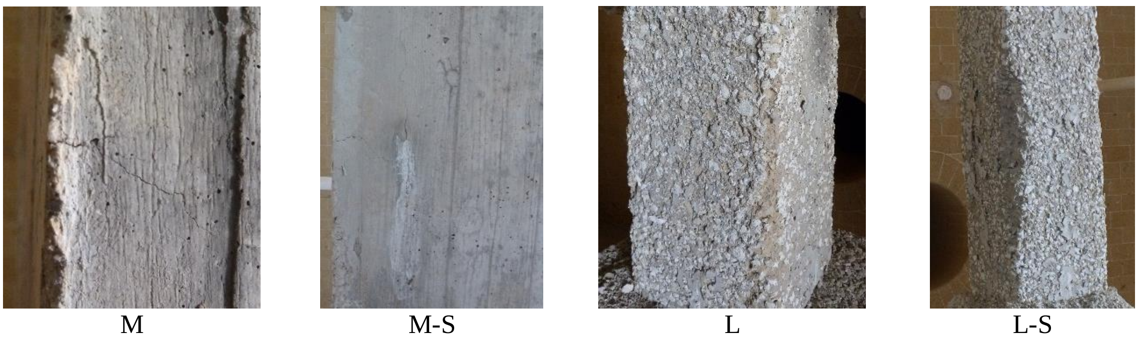



Fig. 5 – Damages observed after fire test.

The damage observed after the fire exposure are presented in Fig. 5. After the medium fire were only observed transversal concrete cracks in the corners of the column. After the long fire spalling and cracks on the concrete cover were observed. The large concrete aggregates are limestone and the sand is quartz and therefore larger aggregates were more affected by the long fire. After the long fire, the concrete surface was observed to became irregular and rough, and consequently it was not possible to identify the crack pattern.

## 4. Repairing and retrofitting of the columns

Columns M-S and L-S were repaired by replacing the concrete damaged during the fire exposure with new structural repair mortar. In column M-S only the concrete corners were removed while in column L-S the whole concrete cover was removed to the depth of the longitudinal reinforcement. The damaged concrete was replaced by a R4 class structural mortar according to EN1504-3 (CEN 2009) with a minimum compressive strength of 45 MPa. The original cross-section dimensions of the columns were kept, and the edges were rounded to a radius of 25 mm. In strengthened control column C-S, the edges were also rounded to a radius of 25 mm. In all retrofitted columns, the concrete surface was roughened and cleaned.

The columns were wrapped with three layers of CFRP sheet along their height to increase the concrete confinement. The wrapping method followed the same procedure already implemented in columns of beam-column joint specimens (Pohoryles, Melo et al. 2018). The CFRP sheet properties were: design thickness ($t_f$) – 0.168mm; ultimate strength ($f_{u,FRP}$) – 4300MPa; ultimate strain ($\varepsilon_{u,FRP}$) – 1.7%; and elastic modulus ($E_{FRP}$) – 240GPa.

## 5. Cyclic test results

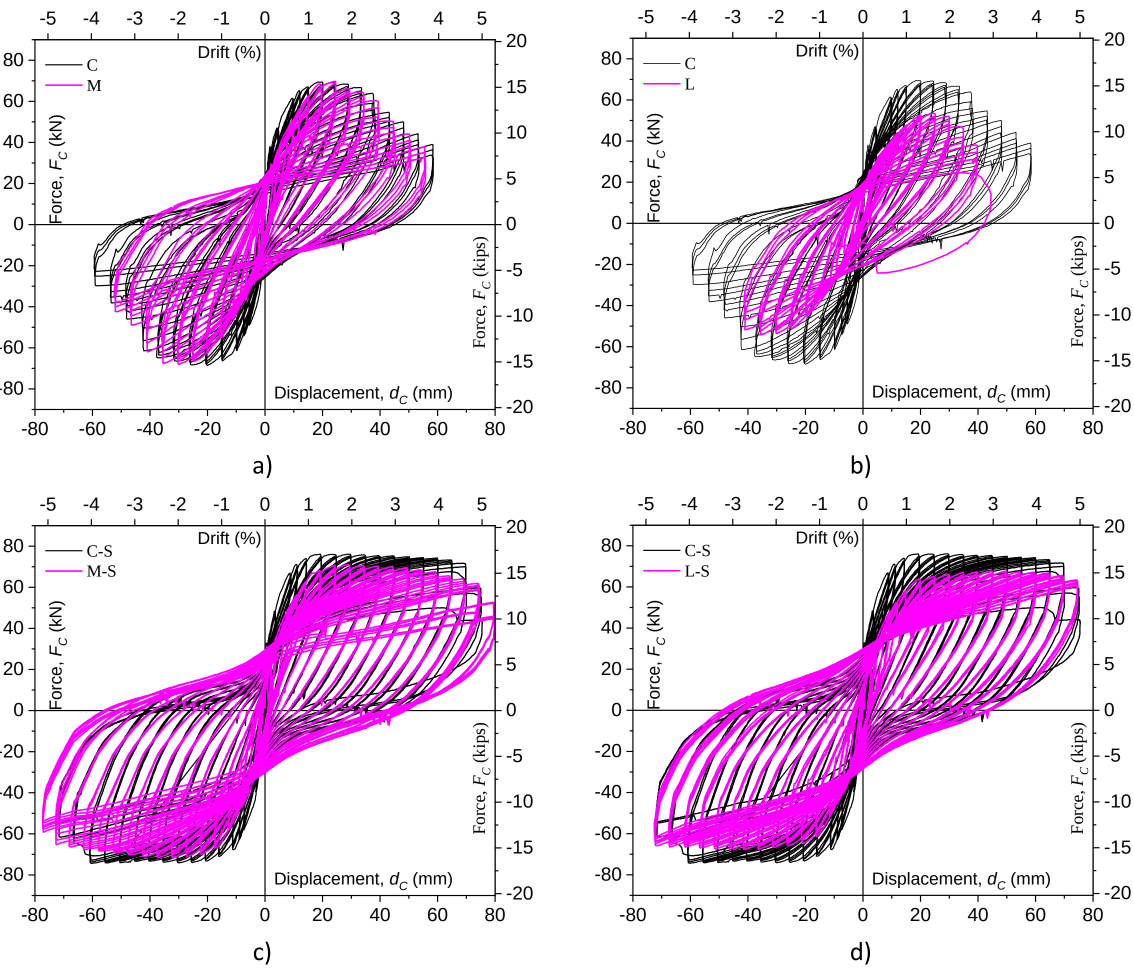



Fig. 6 – Lateral load-displacement relationship: a) C and M; b) C and L; c) C and M-S; d) C and L-S.

The direct comparison between the control columns (C and C-S) and the other columns of the experimental cyclic results are shown in Fig. 6. Column M shows a reduction in the initial stiffness but similar maximum force and deformation capacity as the control column (C). In the case of 90-minute fire, a reduction in initial stiffness as well as a significant reduction in peak force and column ductility are observed in the column L as compared to the control specimen. The retrofitted columns with fire damage (M-S and L-S) reached lower peak loads and had lower initial stiffnesses than the retrofitted control column (C-S). The unloading-reloading stiffness, and consequently the pinching effect, is similar for all the retrofitted columns.

Similar values of the peak force ($F_{c,max}$) were observed in columns C and M. Instead, columns L, C-S, M-S and L-S achieved -23%, +10%, +2% and -2% of the column C peak force, respectively. The $F_{c,max}$ of columns M-S and L-S were -7% and -11% of that of column C-S. The displacement ductility differences between column C and columns M, L, C-S and M-S were -44%, -66%, +79% and +8%, respectively. The cumulative hysteretic dissipated energy values at the ultimate drift for columns C, M, L, C-S and C-M, were respectively 53.7 kNm, 45.1 kNm, 25.0 kNm, 175.5 kNm and 187.7 kNm. Columns M and L dissipated 16% and 53% less energy than the control column C up to the ultimate drift and columns C-S and M-S dissipated 227% and 250% more energy than column C. The fire damage decreases the energy capacity of the columns and the CFRP wrapping increases significantly the dissipated energy capacity for cyclic lateral loading.

## 5. Main conclusions

An experimental campaign performed on six full-scale RC columns was developed to assess the post-fire seismic performance of the columns and after post-fire strengthening. Based on the experimental results, the following conclusions can be drawn: i) After the medium fire (30 minutes) only transversal cracks in the corners of the columns were observed, but after the long fire (90 minutes) concrete cover spalling and general cracking were observed; ii) the displacement ductility and dissipated energy of columns M and L were respectively 44% and 66% and 16% and 53% lower than in the control column C. This shows that the cyclic response of column L was severely compromised by the long fire, which justifies the need for the column to be strengthened after the fire; iii) columns M-S and L-S (repaired and strengthened with CFRP wrapping after fire) showed better cyclic behaviour than the control column C with cumulative dissipated energy 227% and 250% larger than column C, respectively; iv) the repair and strengthening method here studied improved the cyclic behaviour of the columns after a medium or long fire and they can have a significant better seismic response than the original control column C without fire exposure. Moreover, the post-fire of the strengthened columns can achieve similar seismic performance than analogous strengthened columns without previous fire damage.

## Acknowledgements

This research was conducted as part of the Challenging RISK project funded by the UK Engineering and Physical Science Research Council - EPSRC (EP/K022377/1) for which Prof. Tiziana Rossetto and Prof. Luke Bisby are Principal Investigators at UCL and University of Edinburgh, respectively. The work developed by the author José Melo was partly financially supported by FCT - Fundação para a Ciência e Tecnologia, Portugal, co-funded by the European Social Fund, namely through the post-doc fellowship, with reference SFRH/BPD/115352/2016 and by Base Funding - UIDB/04708/2020 and Programmatic Funding - UIDP/04708/2020 of the CONSTRUCT - Instituto de I&D em

Estruturas e Construções - funded by national funds through the FCT/MCTES (PIDDAC). This work is partial financially supported by national funds through the FCT/MCTES (PIDDAC), under the project PTDC/ECI-EGC/7244/2020.

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