Experimental and numerical analysis of in-plane compressed unprotected log-haus timber walls in fire conditions

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A B S T R A C T

The paper presents an experimental and Finite Element (FE) numerical analysis of the behavior of unprotected log-haus timber walls in fire conditions under in-plane compressive loads. The aim is to assess their overall structural performance and to provide possible design suggestions. In doing so, the main results derived from a full-scale experimental test of a log-haus specimen subjected to the standard fire curve and loaded in-plane in compression are first described. FE numerical simulations are then carried out, to further assess the test results and to perform - based on the rather close correlation between test and FE results - a parametric study on the examined structural system. The effects of several influencing parameters are then investigated, including the presence of an initial geometrical out-of-plane global curvature, the possible exposure to fire of orthogonal logs and carpentry joints acting as lateral outriggers for the main log-haus wall, and the compressive loading ratio acting in combination with the fire loading. The most significant effects of such influencing parameters are highlighted in terms of overall buckling resistance and failure mechanisms for the examined walls in fire conditions, providing evidence for the reduction of their actual load carrying capacity. In conclusion, aiming to derive useful design suggestions, a possible extension to log-haus systems of the “Reduced Cross-Section Method” (RCSM) currently in use for the verification of fire exposed timber members is proposed.

1. Introduction and research objectives

Log-haus (or log-house, or Blockhaus) structural systems are obtained by assembling multiple timber logs, which are stacked horizontally on the top of one another. Although based on conceptually simple resisting mechanisms, the structural behavior of log-haus systems is rather complex to predict. For this reason, no design recommendations are available in current timber standards (i.e. [1–3]). Since metal connectors are generally avoided or minimized, the typical log-haus wall can sustain the vertical loads as far as a minimum level of contact among the logs is ensured. At the same time, the very low modulus of elasticity (MOE) of timber in the perpendicular to the grain direction makes the usually slender log-haus walls rather susceptible to flexural buckling, hence requiring the implementation of specific analytical design models.

So far, several research studies have been carried out on log-haus systems especially under seismic loads, in order to assess their actual structural performance and dissipative capacity (see for example [4–7]), including investigations on small components, walls, full 3D buildings and related Finite Element (FE) simulations. In general, the actual lack of specific design regulations for the examined structural system still requires investigations under extreme events as well as under ordinary design loads, including buckling phenomena.

Compared to structural members and assemblies made of steel, for example, the buckling resistance of timber structural systems markedly depends upon the mechanical constitutive behavior and intrinsic anisotropy of the basic material. As a result, standardized and simplified analytical models available in current standards for steel and masonry structures (i.e. [8,9]) cannot be directly extended to timber log-walls and wood structural assemblies in general [10–14]. In this regard, past research efforts have been devoted to the implementation and calibration of standardized design buckling curves for the verification of log-haus timber walls under in-plane compression only or combined in-plane compression and orthogonal pressures, see Refs. [15–17], following existing approximate formulations [18,19]. There, it was also shown that additional steel profiles can be successfully used as reinforcements, to enhance the in-plane and out-of-plane stiffness and resistance of...
Compared to earlier research studies, the current investigation aims to further explore the buckling performance of log-haus systems in fire conditions. As such, the paper presents full-scale experimental results and Finite Element numerical simulations of a timber log-haus wall exposed to the standard fire ISO curve and subjected to in-plane compressive loads. The test results are first described and critically discussed, with the aim to highlight the typical behavior as well as possible issues in such systems when exposed to fire.

A full 3D solid, Finite Element model is then implemented in ABAQUS [21], v.6.12, aiming to explore the thermo-mechanical performance of the reference specimen. In the reference FE model, a key role is played by both the thermo-physical properties of materials and by the thermal boundary conditions of the specimen, as well as by the actual mechanical characterization of timber in fire conditions, including a set of mechanical contact interactions between the adjacent logs (see also [15–17,20]).

Based on the rather close correlation between test and FE results, both in terms of temperature distribution and mechanical performance, a FE parametric study is then presented, aimed to assess, for the same specimen, the effects of some major influencing parameters on its overall buckling performance. These influencing parameters include possible initial global out-of-plane curvatures, different thermal insulation configurations for the outriggers (hence providing evidence of the effect of lateral wall supports), and the level of applied in-plane compressive load in fire conditions. The sensitivity of the actual buckling resistance of log-haus walls in fire conditions is hence emphasized, investigating the combined effects of thermal exposure and applied load.

Since log-haus walls were found to be highly susceptible to buckling phenomena and fire loading, a further set of FE simulations is finally presented and discussed. The aim is to preliminary assess the applicability of the simplified design regulations currently given by the Eurocode 5 Part 1–2 for the design of timber members exposed to fire (i.e. the “Reduced Cross-Section Method” (RCSM), see Ref. [22]) to the examined structural system. It is expected that the proposed application of the RCSM design approach for log-haus systems could be further calibrated via extended parametric studies, inclusive of a wide set of geometrical and loading configurations for log walls representative of real full 3D buildings (i.e. walls with double/door openings and different log section, distance between outriggers, etc.).

2. Experimental investigation

The exploratory investigation presented in this paper was carried out in Italy in 2009 (laboratory facilities of CSI, Bollate (MI)). The most important geometrical details of the tested log-haus specimen as well as the experimental methods are provided in the following sections.

2.1. Geometrical and mechanical properties of the test specimen

The specimen was designed to be representative of a typical log-haus wall, featuring a main wall, two intercepting orthogonal walls and special carpentry joints at the corners.

The main log-haus wall, see Fig. 1, consisted of 16 timber logs, 3 m long, and 90 mm × 160 mm (width b × depth h) cross-sectional dimensions given by two glued lamellas of C24 strength class spruce [23]. The average density of these logs was about 420 kg/m³. By stacking horizontally these logs resulted in a main log wall with total height H = 2.96 m and actual buckling length of logs (i.e. distance between outriggers axis) L = 2.71 m. The same type of logs were then used also to build the 590 mm long portions of orthogonal walls (outriggers) acting as continuous lateral bracing for the main wall (see Fig. 1). ‘Standard’ carpentry joints were then realized at the corners between the main wall and the outriggers.

**Fig. 1.** Layout of a typical log-haus timber wall with ‘standard’ carpentry joints ((a) front and (b) top view), together with the instrumentation setup on the external side not exposed to fire.
and the orthogonal walls (Fig. 1, detail).

2.2. Summary on the compressive buckling resistance of log-haus timber walls at ambient condition

The geometrical and mechanical features of the specimen displayed in Fig. 1 are in close correlation with the experimental and numerical investigation carried out in Refs. [15–17] for the assessment of the compressive buckling resistance of log-haus systems at ambient condition. This similarity will be considered later on for additional discussion of test results (see Section 3).

For a log-haus timber wall subjected to a uniformly distributed in-plane compressive load acting on the top it was shown in Ref. [15] that the actual Euler’s buckling resistance can be calculated as:

\[
\left( N_{cr}^{(E)} \right)_d = \frac{k_{c} \pi^{2} b^{4}}{12 L} \frac{E_{i.a}}{\gamma_{M}} \left( 1 - \left( \frac{E_{i.a}}{\gamma_{M}} \right)^{2} \right)
\]

(1)

where the buckling coefficient \( k_{c} = 6.97 \) was estimated for walls without door/window openings, being \( \left( N_{cr}^{(E)} \right)_d = f(E_{i.a}, G_d, f_{c,90.d}) \).

In Eq. (1), \( E_{i.a}, G_d \) and \( f_{c,90.d} \) signify the design MOE in the direction perpendicular to the grain, the design shear modulus and the design value of compressive strength perpendicular to the grain:

\[
E_{i.d} = \frac{E_{i}}{\gamma_{M}} \quad (2a)
\]

\[
G_{d} = \frac{G}{\gamma_{M}} \quad (2b)
\]

\[
f_{c,90.d} = \frac{k_{mod}f_{c,90,k}}{\gamma_{M}} \quad (2c)
\]

where \( E_{i} = 370 \) MPa, \( G = 500 \) MPa, \( f_{c,90,k} = 2.5 \) MPa, while \( \gamma_{M} = 1.3 \) is the material partial safety factor, in accordance with EN 1995-1-1 [1], and \( k_{mod} = 0.7 \) denotes the partial modification factor for moisture and load duration influence (service class 1 and 2, long-term load).

As also shown in Refs. [15–17], the typical buckling response of log-haus walls under in-plane compressive loads is in close correlation with the buckling behavior of fully monolithic walls with simply supports along the top/bottom edges and – given the geometrical properties of ‘Standard’ carpentry joints – continuous clamps along the vertical edges (see Fig. 2). As a key feature of the examined structural system, however, the absence of any possible mechanical or adhesive connection along the main logs, with the exception of contact interactions only, typically results in reduced overall resistance and stability of log walls, compared to monolithic specimens with identical global dimensions and mechanical features. Further influencing parameters reducing the actual buckling resistance of log-haus walls are the typically high slenderness ratio of a single log, as well as the possible eccentricities, global curvatures, and presence of door and/or window openings. When single or double door/window openings are present, the design Euler’s buckling resistance is given by specific formulae of general use, see Refs. [15–17] for further details.

As a major outcome of the earlier research study, a standardized design formulation was calibrated in Ref. [15], so that the design in-plane compressive buckling resistance of a log wall with general geometrical and mechanical properties can be carried out – in accordance with the Eurocode 5 approach for the buckling verification of timber members in compression – by checking the condition:

\[
N_{d} \leq N_{cr.d} = k_{c} \times \left( N_{cr}^{(E)} \right)_d.
\]

(3)

According to the analytical model validated and assessed in Ref. [15] for log-haus walls in service class 1 and 2, under the action of long-term duration loads, the design Euler’s resistance \( \left( N_{cr}^{(E)} \right)_d \) in Eq. (3) is given by Eq. (1) or by other equations listed in Ref. [15], depending on the properties of the log wall. The coefficient \( k_{c} \) was then calibrated in Ref. [15] to account - at ambient temperature - for the geometrical properties of the log wall (i.e. cross-sectional dimension \( b \times h \) of each log, overall dimensions \( L \times H \) of the wall, number and position of door/window openings, etc.), as well as for the actual buckling resistance reduction due to initial load eccentricities and/or geometrical curvatures, including the presence of in-plane fully rigid or flexible inter-storey floors, or possible crushing phenomena in the timber logs under compression perpendicular to grain.

In accordance with Fig. 1 and the aforementioned data, an Euler’s design buckling resistance \( \left( N_{cr}^{(E)} \right)_d \) laying in the order of \( \approx 260 \) kN/m was calculated for the examined full-scale specimen, based on Eq. (1), corresponding to a mean Euler’s buckling resistance \( N_{cr.d} \approx 340 \) kN/m. In terms of buckling design resistance of log walls at ambient temperature, in this regard, it is important to point out that the so estimated Euler’s resistance value should be reduced by means of the \( k_{c} \) buckling coefficient (\( k_{c} = 6.97 \) for the examined configuration, see Ref. [15]), being accidental imperfections or eccentricities typically associated to a marked reduction of the theoretical buckling resistance. As such, the mentioned Euler’s load values are considered in this research contribution for comparative purposes only.

![Fig. 2. Typical buckling deformed shape of a timber log-haus wall under in-plane compression, (a) in absence of openings (ABAQUS) or (b) with door/window openings [15–17].](Image)
2.3. Experimental setup and methods

The specimen displayed in Fig. 1 was positioned in a large-scale vertical furnace with $3 \times 3$ m mouth opening size and 0.9 m depth, so that the main wall as well as the orthogonal walls could be exposed to fire. Based on the overall dimension of the specimen, as well as on the furnace opening size, possible gaps between the specimen and the furnace were then filled with REI 180 fire resistance panels, see Fig. 3(a).

Before the execution of the test, the specimen was conditioned for 4 weeks at a room temperature of 25°C ($\pm$2%) and 50% relative humidity ($\pm$10%).

During the test, the standard fire curve according to the ISO 834 regulations was followed on the internal side of the specimen. The test was then carried out following the provisions of EN 1363 regulations [3]. Throughout the fire test, the temperature within the furnace was continuously monitored via a plate thermometer. A total number of 12 additional thermocouples, type K IEC 584-1, were positioned on the external side of the specimen to record the temperature during the test at different locations, see Fig. 1 (P7-P18 control points). The thermocouples were divided into two groups, which were used to check the fire insulation capacity of the specimen by monitoring the average and maximum temperatures on the external side. The insulation verification can be regarded as fully satisfied as long as the increase in average and maximum temperature $\Delta T$ during the exposure to fire does not exceed the limit values of 140°C and 180°C respectively.

Linear Voltage Displacement Transducers (P1-P3 and P4-P5, see Fig. 1) were also positioned on the external side of the specimen to control its out-of-plane and in-plane deformation due to the combined effect of the fire and the uniformly distributed, in-plane compressive load applied on the main wall via a rigid steel frame (Fig. 3(b)). The in-plane compressive load was set to $N_{\text{test}} = 45$ kN/m, corresponding to $\approx 1/7$th of the mean value of Euler’s resistance at room temperature given by Eq. (1).

This load was applied 20 min before the onset of the fire test itself and kept constant during the experiment.

2.4. Experimental results

The fire experiment was stopped after 60 min, due to the collapse of

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**Fig. 3.** Photos of the full-scale specimen before testing. (a) Exterior side of the log-haus wall and (b) instrumentation layout.

**Fig. 4.** Photo of the specimen near to the failure conditions. (a) Fire propagation on the top of the specimen, with (b) abrupt increase of out-of-plane deformations and (c) collapse.
the specimen (see Fig. 4).

The log wall showed a relatively high fire resistance and a qualitatively stable behavior, despite the rather abrupt collapse mechanism.

In accordance with EN 1363-1 [3], the load-carrying capacity of the tested specimen can be detected (with the exception of the first 10 min of test) as the simultaneous exceedance of a limit deflection and a limit deflection rate.

For structural systems mainly loaded in bending, these limits are given as \( L^2/400d \approx 200 \text{ mm} \) and \( L^2/900d \approx 9 \text{ mm/min} \) respectively, with \( L = 2710 \text{ mm} \) and \( d = b = 90 \text{ mm} \) signifying the actual length and width of a single timber log. For structural systems mainly loaded in-plane, the EN 1363-1 gives reduced values of \( H/100 \approx 29 \text{ mm} \) and \( 3H/1000 = 9 \text{ mm/min} \) respectively, with \( H = 2960 \text{ mm} \) the total height of the main wall at the beginning of the test. Being the examined log wall representative of a vertically loaded specimen, in accordance with the EN 1363-1 regulations, the maximum axial shortening and shortening rate should be assumed as a reference parameter for evaluating the load-carrying capacity. Due to the nature of timber log walls, however, careful consideration should be given to the maximum out-of-plane deflections, as also observed in Refs. [15–17].

Fig. 5 displays the experimentally measured out-of-plane and in-plane histories of displacements. Positive values denote inward deflections and uplift of the main timber logs, respectively. As shown, despite a slightly unsymmetrical response of the main log wall to fire loading, the specimen proved to offer a rather stable structural response during most of the fire loading time. Average out-of-plane and in-plane deformations were in fact in the order of \( \approx 10 \text{ mm} \) \( (=L/270) \) and \( \approx 5 \text{ mm} \) \( (=H/600) \) close to collapse. An abrupt increase of measured deformations and corresponding deflection rates for the main logs occurred after 60 min of fire loading, due to the collapse of charred orthogonal logs and subsequent overturning of the unrestrained main wall (see also Fig. 4(c)).

In terms of increase in temperature on the external side of the wall, see Fig. 6, test measurements confirmed a rather stable insulation capacity and uniform distribution of temperatures on the whole specimen, for more than 30 min of fire loading (see for example Fig. 6(a)). Maximum peaks of temperature were observed, in the second part of the test only, especially close to the top logs, see P13 in Fig. 6(b), as well as close to the lateral edges of the specimen (i.e. P8, P15 – see Fig. 1). Such temperature distribution is in close correlation with Fig. 4, where it is possible to observe that the fire first propagated from the top logs (Fig. 4(a)) as well as in the orthogonal walls acting as bracing system for the main wall (Fig. 4(b)). The collapse of the orthogonal logs led then to failure of the full specimen (Fig. 4(c)), as the main logs became totally unbraced in the final instants of the test.

3. Finite element numerical analysis

3.1. Model implementation and calibration

A Finite Element (FE) numerical investigation was carried out to simulate and further assess the buckling performance of log-haus timber walls in fire conditions. The experimental test described in Section 2 was first reproduced. Due to the symmetry of the full-scale specimen, only half the nominal geometry was modelled, assuming an ideal uniform distribution of geometrical and mechanical properties for timber material, logs, boundaries and loads.

Three-dimensional 8-node solid elements were used to describe each timber log (DC3D8 type, diffusive heat transfer linear brick elements available in the ABAQUS library). In accordance with [5,16], careful consideration was given to the geometrical features of the timber members and to the carpenter joint with the perpendicular logs, which are crucial to ensure a proper structural behavior of the wall specimen. The small notches and protrusions along the top and bottom surfaces of each log, accordingly, were neglected assuming a nominal \( 90 \times 160 \text{ mm} \) rectangular cross-section, see Fig. 7.

A regular mesh pattern was then chosen for the 8-node elements. Based on a preliminary sensitivity study not reported in this paper due to space limitations, 4 solid elements were used through the thickness of
each log, with an average size in the range 20–50 mm. This choice ensured reliable FE thermal and mechanical results, as well as an appropriate computational time of the numerical simulations. The so assembled FE model consisted of 12000 solid elements and 21000 Degrees of Freedom.

3.2. Thermal analysis in fire conditions

An uncoupled heat transfer analysis was first carried out to describe the thermal state of the unprotected log-haus wall subjected to the ISO fire curve displayed in Fig. 6(a). The fire exposure of logs was simulated by means of appropriate boundary conditions of radiation and convection, at the interface between the main log-haus wall and the surrounding environment, as well as on the lateral surfaces of the orthogonal logs. As such, three different regions were separately detected in the lateral surfaces of the full FE models (see the ‘B1’, ‘B2’ and ‘B3’ surfaces of Fig. 8), and properly restrained, depending on the fire exposure of the experimental specimen, in order to account for the actual boundary conditions and symmetry assumptions.

Emissivity and convection coefficients of timber were set to 0.8 and 25 W/m²K, as suggested by Eurocode 1 [24]. The B1 and B2 surfaces were subjected to the standard fire ISO curve of Fig. 6(a) and to an initial ambient temperature of 20 °C, respectively. For the B3 surface, the presence of insulation panels enabling the propagation of fire was properly taken into account. Symmetry constraints along the vertical axis of the main logs were finally considered.

In terms of thermo-physical characterization of timber, see Fig. 8(c), conductivity and specific heat at ambient temperature were set equal to 0.12 W/mK and 1.53kJ/kgK [24]. Following the Eurocode provisions, a variation of these reference properties \( p \) with temperature \( T \) was considered in the reference heat transfer simulation, see Fig. 8(c). There, the input curves are provided as the \( p(T) \) to \( p(20\, ^\circ C) \) ratio versus temperature \( T \), for each \( p \) property.

3.3. Mechanical analysis in fire conditions

As a subsequent stage of the heat transfer simulation, an uncoupled nonlinear mechanical analysis was carried out on the same FE model described in Section 3.2, aiming to assess the effects of the simultaneously imposed ISO fire curve and compression load \( N_{\text{test}} \).

To this aim, the results of the thermal simulation were separately saved, in the form of a distribution of nodal temperature histories for all the nodes composing the FE model, and then imported as reference configuration for the mechanical analysis. In this way, under the assigned compressive load \( N_{\text{test}} \), the variations in mechanical stresses occurring in the timber logs due to temperature increases were properly taken into account. The same geometry and mesh pattern of the FE model was hence used for both the thermal and mechanical simulations. In the latter case, compared to Section 3.2, some modifications of the thermal FE model were introduced at three different levels, including the (i) type of 8-node solid elements, (ii) boundary and loading conditions, (iii) contact interactions and (iv) material mechanical properties. C3D8R type, linear brick elements with reduced integration were in fact used. The symmetry of the FE model was ensured by nodal restraints for the nodes laying on the vertical symmetry plane of the specimen. The full FE model was hence simply supported at the base, while the compressive load \( N_{\text{test}} \) was applied in the form of a uniformly distributed pressure, on the upper face of the main top log. Self-weight of timber logs as obtained from material...
density was also automatically assigned to the FE model. Another key aspect of the mechanical FE model was then represented by the surface-to-surface contact pair interactions, which were applied to all the possible contact surfaces of adjacent and overlapping logs. In accordance with [5,16], a penalty frictional interaction was defined, with $\mu = 0.5$ the average static friction coefficient. In terms of relative behavior of timber logs in the direction perpendicular to the contact surfaces, the ‘hard’ normal formulation was implemented, allowing for possible separation of adjacent logs once subjected to tensile stresses. C24 spruce anisotropy at ambient temperature was also accounted via engineering constants representative of the MOE and shear moduli along the principal directions of the resisting members. Nominal mean values provided in Ref. [23] were considered, with $E_1 = 11000$ MPa, $E_2 = 370$ MPa and $G = 500$ MPa. In accordance with [5,16], the material strength was assumed equal to the compressive resistance of spruce in perpendicular to the grain direction, with $f_{c,90} = 3.57$ MPa the mean value. Material density at 20 °C was finally set to the nominal value of 420 kg/m$^3$. A decrease in the timber properties with temperature was finally considered, in accordance with the Eurocode 5 provisions for standard fire exposures, whereby the MOE and strength reduce to zero at 300 °C (see Fig. 8(d)).

At ambient temperature [15–17], initial geometrical imperfections of $u_0 = L/300$ the wall span were considered to assess the compressive buckling performance. In fire conditions, however, no initial geometrical curvature $u_0$ was preliminarily taken into account for comparative purposes towards test measurements. No accurate experimental data on logs straightness was in fact available from the full-scale test, and production tolerances generally allow for logs maximum deviations up to 1–1.5 mm/m, as in the case of beam-like structural elements composed of timber (see also [16]). At the same time, due to the fire exposure of logs, the absence of any curvature defect was considered statistically correct and representative of an average, general initial configuration for the experimental specimen.

On the other hand, it is also expected that any initial global curvature $u_0$ with possible inward or outward deflection would have implicitly affected the overall fire performance of the same FE model, also in accordance with earlier studies related to buckling performance of log-haus systems in cold conditions (see for example [15–17]). As a result, in view of possible design suggestions, initial curvatures should be properly taken into account.

3.4. Discussion of thermal FE results

In Fig. 9, FE results of the thermal simulation are first compared with the corresponding test measurements, with special attention given to 60 min of fire exposure. A rather good correlation can be observed in

![Graph showing temperature evolution](image)

(a)

![Overall temperature distribution](image)

(b) $t=60$ min

![Details of temperature distribution](image)

(c) $t=15$, $30$, $60$ min

Fig. 9. FE heat transfer results (ABAQUS). (a) Experimental and numerical evolution of temperature on the external side of the main wall (selected control points), (b) overall distribution of temperature ($t = 60$ min) and (c) detail of temperature distribution in selected logs, with corresponding residual section, after $t = 15$, 30 and 60 min of fire exposure.
terms of temperature distribution on the external surface of the main wall, for most of the monitored control points. Larger differences were observed for the thermocouples close to the edges of the specimen, i.e. in the proximity of the corner joints (P15) as well as close to the top log of the main wall (P12, P16), hence suggesting possible local boundary effects for the experimental specimen, compared to the ideal assumptions of the FE models.

The evolution of temperatures over time generally proved to be rather uniform on the whole external surface of the specimen for all the FE control points corresponding to experimental thermocouples, as well as on the internal side directly exposed to fire (see Fig. 9(a) and (b)).

In terms of temperature increase in the proximity of the carpentry joints, in accordance with Fig. 9(b), three contour plots are displayed in Fig. 9(c), being representative of the typical temperature distribution in a main and orthogonal logs as well as of charring rate (box details), at selected time instants of the FE fire test (with $t = 15$, 30 and 60 min of exposure to the ISO standard fire curve). For each of the selected time instants, the contour of the temperature distribution within the main timber logs and the residual section of the notched 'Standard' joints at the corners of the wall (red color for $T > 300\,^\circ\text{C}$) are displayed.

Charred depth and fire performance of the tested specimen are further investigated and displayed in Fig. 10, in terms of evolution of temperature for several control points at various time instants, both for (a) a main log wall and (b) an orthogonal log. Assuming a reference temperature of $300\,^\circ\text{C}$ as the limit between charred timber without residual resistance and heated timber with residual load-carrying capacity, Fig. 10(a) displays the temperature values calculated on a main log cross-section, considering 5 control points laying on its middle horizontal axis (see also the schematic cross-section in Fig. 10(a)). The selected temperature profiles are displayed for 15, 30, 45 and 60 min of fire exposure, as a function of the distance of control points from the fire exposed surface, while the gray curves refer to intermediate time intervals (5 min time step). As shown, after 60 min of fire exposure, more than half resisting cross-section has charred. Since the second half section is also clearly heated, it is able to provide only limited load-carrying capacity. From the same figure, it can be also noticed that the nominal cross-section of logs is already severely affected from fire loading after 25–30 min of exposure to fire, with $\approx 1/3$ of nominal section charred.

Similar findings were also observed in terms of temperature distribution in the orthogonal logs, see Fig. 10(b). Due to the crucial role played by the outriggers and carpentry joints for the main logs, acting as bracing system for the tested log wall, a larger number of control points was considered for temperature monitoring and qualitative assessment of the observed fire performances. In accordance with the schematic drawing of Fig. 10(b), where the main timber logs are hidden from view for clarity of presentation, a reference control point was first selected in the middle lateral face of the joint notch. Additional control points (7 in total) laying on the middle $a_1$ axis were also considered, being part of the mesh nodes directly exposed to fire or laying on the external side of the furnace, respectively. Further 8 nodes belonging to the longitudinal axis $a_2$ of the log (with $b/2$ the distance from the $a_1$ axis) were also finally taken into account for comparative purposes.

Fig. 10. Numerical evolution of temperatures (a) in the main logs and (b) in orthogonal logs, at various control points (ABAQUS).
In Fig. 10(b), a highly non-linear evolution of temperatures is shown. Measurement points located on the B1 surface exposed to fire (a1 axis) proved to be subjected to extreme temperatures (>600°C) after few minutes of loading. It is also possible to notice, however, that the presence of main timber logs protects the carpentry joint notches from fire, with maximum temperatures up to ≈100°C after 60 min. Control points laying on the external side of the wall, finally, are not affected by fire loading. As far as control points along the a2 reference axis are considered, see Fig. 10(b), partially charred timber can be found after 60 min of fire loading, with a residual cross section of outriggers almost halved, hence giving further correlation with the experimentally observed failure mechanism and providing evidence of the fundamental role played by lateral restraints.

3.5. Discussion of mechanical FE results in fire conditions and fire endurance assessment

Beside the rather uniform distribution of temperatures through the width and span of timber members, as numerically observed and summarized in Figs. 9 and 10, a rather uniform distribution of out-of-plane displacements was also noticed from the non-linear mechanical analysis of the same specimen, see Fig. 11.

In Fig. 11, for clarity of presentation, evidence of observed FE results is provided for 60 min of fire exposure only, while more detailed discussion of the observed failure mechanisms is reported in Sections 4.1 and 4.2.

The largest out-of-plane displacements were numerically observed close to the centre of the wall, see the P1-FE plot in Fig. 11(a). FE displacements monitored at control points P2 and P3 further confirmed the occurrence of an almost cylindrical deformed shape for the full specimen, which is also in accordance with the experimental collapse mechanism of Fig. 4(b), with maximum out-of-plane displacements occurring close to the middle height of the wall and almost null displacements at the top and bottom logs. In terms of in-plane deformations at the base of the wall, hardly any displacement was observed, see P5 and P5-FE plots in Fig. 11(b).

Compared to the EN 1363 regulation for the estimation of the load-carrying capacity of the specimen, the test and FE results emphasized (in accordance also with earlier research studies [15–17]) the high flexibility of timber log walls, both at ambient temperature and in fire conditions. The implicit feature of such structural system is in fact the typically high slenderness ratio of the constituent logs, as well as the absence (with the exception of contact interactions only) of mechanical connections between the overlapping logs. As such, the structural performance under in-plane compressive loads is mainly associated to out-of-plane deflections rather than in-plane deformations as the EN 1363 suggests for vertically compressed logs. On the other hand, the deformation limits and deformation rate limits the EN 1363 provides for single timber members in bending do not capture at all the actual performance of timber log walls, being associated to a flexural performance laying between the limit conditions of n separate overlapping logs and a fully monolithic wall. As such, careful consideration and specific research studies, as well as the implementation and calibration of ad hoc design regulations, are required.

Finally, see Fig. 11(c), possible initial global out-of-plane curvatures of the main wall proved to have slight effects only for the examined specimen, for the specific fire exposure configuration and restraint condition of the tested log wall. In Fig. 11(c), in particular, evidence of both outward and inward initial imperfection with maximum amplitude $u_0 = L/300$ is provided for displacements at control points P1, P2 and P3. As shown, minor scatter was generally observed for FE plots corresponding to the same control point, by changing the imperfection amplitude, compared to test measurements, up to 60 min of fire exposure. Based on the observed outcomes, as well as on the lack of preliminary experimental measurements related to initial geometrical configuration of the tested specimen, the assumption of null geometrical
curvature for the FE simulations was hence justified for the development of preliminary FE parametric studies (see Sections 4.1 and 4.2). Generally speaking, however, it is clear that geometrical curvatures should be properly taken into account for design purposes, since markedly influencing the overall buckling performance of the examined structural typology (see Refs. [15–17] and Section 4.3).

4. FE parametric investigation

Following Section 3, an attempt to further explore and assess the compressive buckling response of log-haus walls under fire loading was carried out, in the form of a FE parametric study.

Based on the experimental and FE outcomes partly summarized in Section 3, careful consideration was in fact given to the effects of both thermal and mechanical aspects for the investigated log wall. The full-scale experiment and the corresponding FE simulation generally proved that the studied structural system can offer a stable compressive resistance under fire conditions. This finding, however, is strictly related to the experimental setup, i.e. to the case of log walls where the lateral outriggers are able to offer a rather linear clamp restraint to the main logs and the applied mechanical loads are limited, compared to load-carrying capacity at ambient conditions.

4.1. Compressive load level

A first set of FE parametric simulations was hence carried out on the reference FE thermal and mechanical models described in Section 3, by changing the amount of the imposed compressive load $N_{test}$.

Table 1

<table>
<thead>
<tr>
<th>$R_n$</th>
<th>EN criteria</th>
<th>FE loss of stability (max. absolute values)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Failure time</td>
<td>Type</td>
</tr>
<tr>
<td>[min]</td>
<td>[min]</td>
<td>[mm]</td>
</tr>
<tr>
<td>0.1</td>
<td>151.8</td>
<td>ID</td>
</tr>
<tr>
<td>0.5</td>
<td>116.6</td>
<td>ODR</td>
</tr>
<tr>
<td>1</td>
<td>66.7</td>
<td>ODR</td>
</tr>
<tr>
<td>1.5</td>
<td>n.a.</td>
<td>n.a.</td>
</tr>
<tr>
<td>2</td>
<td>n.a.</td>
<td>n.a.</td>
</tr>
<tr>
<td>3</td>
<td>n.a.</td>
<td>n.a.</td>
</tr>
<tr>
<td>4</td>
<td>30.3</td>
<td>ID</td>
</tr>
<tr>
<td>5</td>
<td>19.8</td>
<td>ID</td>
</tr>
</tbody>
</table>

Table 1 Expected failure time and mechanism type for log-haus walls under various compressive load levels (0.1 $\leq R_n \leq$ 5), with evidence of out-of-plane deformations as a function of time (n.a. – not attained).

Fig. 12. FE parametric results (ABAQUS), as obtained for the main log wall under various compressive load levels (0.1 $\leq R_n \leq$ 5), with evidence of out-of-plane deformations as a function of time (n.a. – not attained).

examined log wall geometry, the $N_{test}$ load was modified via a $R_n$ multiplier ($R_n = 1$ for the reference test setup) that was varied from 0.1 to 5, by multiplying the initial $N_{test}$ load by $R_n = 0.1$, 0.5, 1.5, 2, 3, 4 and 5. The $R_n = 5$ loading case was selected as representative of a top limit configuration for the examined log-haus wall, being the so imposed compression value close to the Euler's design resistance of the specimen in cold conditions (Eq. (1)). The lower limit case given by $R_n = 0$ was also finally considered, as representative of the main log wall under the action of self-weight only (i.e. $N_{asg} = 0$). The so obtained results are proposed in Fig. 12, where the evolution of maximum out-of-plane deformations measured at control point P1 is displayed, as a function of time. There, for clarity of presentation, the $R_n = 0$ case is not displayed since very small in-plane and out-of-plane displacements were measured, compared to $R_n > 0$ configurations, and the expected failure time is in the order of 300 min.

Additional comparative results are also provided in Table 1, in terms of numerical observation of loss of stability and failure configurations corresponding to EN criteria. In accordance with the EN 1363 provisions, the possible occurrence of collapse mechanisms due to exceedance of either maximum in-plane (i.e. walls) or out-of-plane (i.e. members in bending) deformation speed rates or displacement amplitudes was in fact monitored, as a function of $R_n$. Both values are representative of the expected collapse time and type.

All the FE simulations were stopped due to numerical loss of stability, which proved to occur later than EN expected failure times for most of the examined cases (with the exception of $R_n = 1.5, 2$ and 3 cases). As a general observation, for all the $R_n$ conditions FE loss of stability occurred for lateral deformation rates at least equal or higher than $\approx 7 \text{ mm/min}$ (see ‘OD’ values in Table 1). Numerically measured in-plane axial deformations and deformation rates (‘ID’ and ‘IDR’ values in Table 1) proved to effectively represent the actual failure condition and to provide reliable collapse criteria for the same log wall in presence of high compressive loads only (see $R_n = 4, 5$ in Table 1), with good correlation with expected EN failure detections, but also for the unloaded wall (see $R_n = 1$ in Table 1).

As shown in Fig. 12 for the examined $0.1 \leq R_n \leq 5$ configurations, a non-linear increase was noticed in the measured out-of-plane deformations of the main log wall, as far as the applied in-plane compressive load levels increased. A rather similar trend was also observed in terms of in-plane deformations monitored at the base of the specimen. Out-of-plane deformations, however, were always found to be markedly larger than vertical contractions of the examined wall configurations.

In this regard, following the EN 1363 regulations for assessing the fire endurance of the tested wall configurations, some preliminary conclusions can be drawn from Fig. 12 and Table 1. Assuming that the log-walls' structural performance is mainly affected by out-of-plane deformations, it is in fact possible to notice that the ‘OD’ EN failure criterion recommended by standards for single timber members in bending (i.e. $L_2^+(400d’ 200 \text{ mm})$ the reference value) would often lead to a marked
overestimation of the actual load-carrying capacity of the investigated walls. This finding derives from the actual deformation limit provided by EN 1363 for single members in bending is based on the assumption of ideal boundary conditions at their ends. This is not the case of the examined log-haus walls, where the fire exposure of orthogonal logs providing end restraints for the main logs typically results in an almost temporary rigid bracing system only.

Assuming an EN out-of-plane deformation rate (‘ODR’ values, in Table 1) equal to \( L^2/9000 \times 9 \) mm/min as reference parameter for detecting the failure configuration of the same walls, as in use for single members in bending, would also provide partial overestimation of FE predictions - but for some of the examined loading configurations only - since all the ultimate conditions were numerically detected for ODR values in the order of 7 mm/min (with the expectation of C. Bedon, M. Fragiacomo Fire Safety Journal xxx (2017) 1). The results displayed in Fig. 12, at the occurrence of lateral displacements exceeding 15–20 mm \( \approx L/130/L \approx 150 \) mm/min could represent a rational failure criterion for the examined structural systems.

4.2. Thermal insulation of outriggers

A second set of FE simulations was then carried out, in the form of uncoupled thermal and mechanical analyses, by progressively subjecting the B3 external surfaces of outriggers (see Fig. 8(b)) to scaled ISO curves obtained from the standardized fire loading path shown in Fig. 5(a). In other words, the beneficial contribution of the insulation panels filling the gap between the external side of outriggers, at the B3 interface with the perimetral wall of the furnace, was considered in this set of simulations.

To this aim, an \( R_F \) scale factor was hence applied, with \( R_F = 0 \) and \( R_F = 1 \) the limit cases representing, respectively, the unexposed B3 surface (like in the case of the full-scale experiment and the corresponding FE model described in Section 3) or the same B3 surface fully exposed to the standard ISO fire curve (i.e. no insulation panels). Such FE assumptions were considered well representative, from a theoretical point of view, of several fire exposure configurations for the logs composing the outriggers, hence representative of the sensitivity of the main log-haus wall performance to the actual end restraints. For all the so considered fire loading scenarios, a constant value \( N_{\text{test}} \) was taken into account for the imposed compressive load.

The so obtained FE results are displayed in Fig. 13. Parametric FE curves are provided in terms of temperature histories on the B3 surface of outriggers (Fig. 13(a)), as well as out-of-plane deflections of the main wall (Fig. 13(b)), as measured for the P1 control point. Additional comparative results related to expected failure configuration of the same FE models are collected in Table 2. From Fig. 13 and Table 2, as expected, it is possible to notice a direct correlation between the fire exposure of outriggers and the corresponding fire endurance of the wall, being the overall deflections of main logs strictly related to the integrity of orthogonal logs themselves. As soon as the B3 surface of outriggers is partly affected to fire loading, see Fig. 13(b), an abrupt increase of out-of-plane deflections can be observed, with large deformation rates achieved after 20–30 min of exposure. During fire exposure, the main logs compressing the compressed wall are subjected to temperature histories in accordance with Fig. 10(a), see the ‘B1 curve’ in Fig. 13(a). Variations on the overall structural performance of the wall, as a result, mostly depend on the lateral restraints only.

In accordance with Table 2, lateral deflection rates proved to represent the earliest failure criterion for all the tested walls, hence confirming the fundamental role of lateral bracings (see ‘ODR’ values in Table 2), but also the reliability of lateral deflection rates up to 6–7 mm/min as good indicator of the fire endurance for the examined structural system.

The FE results partly discussed in the paper highlighted that not only the fire exposure of the outriggers has a key role on the observed performance of the investigated walls, but also the relative deformations of their constituent logs (i.e. uplift and relative sliding of overlapping logs) play an important role on the ultimate resistance of the examined specimens. At the same time, it should be also pointed out that the current FE parametric investigation is representative of a limited number of possible cases of technical interest, and the high variability of real log-haus systems as a part of full 3D buildings would require an extremely wide set of investigations, including geometrical configurations of single logs, outriggers and carpentry joints, fire exposure of outriggers and their distance, etc. In this context, for design purposes, the assessment of the compressive buckling performance of a given unprotected timber log wall in fire conditions could be preliminarily carried out by assuming - in

<table>
<thead>
<tr>
<th>Table 2</th>
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</table>
| Expected failure time and failure mechanism type for log-haus walls under various fire exposures for the lateral outriggers \( 0 \leq R_L \leq 1 \), \( N_{\text{test}} \) the load, in accordance with EN regulations (measurements provided for the P1 control point).
| Key: ID = in-plane displacement; IDR = in-plane deformation rate; ODR = out-of-plane deformation rate; OD = critical failure condition. |
| \( R_F \) | EN criteria | FE loss of stability (max. absolute values) |
|---|---|---|---|---|---|---|---|
| Failure time | Type | Failure time | ID | IDR | OD | ODR |
| [min] | [mm] | [mm] | [mm/min] | [mm/min] | [mm/min] | [mm/min] |
| 0 | 66.7 | ODR | 108.1 | 57.4 | 3.1 | 210.5 | 10.2 |
| 0.1 | 63.3 | ODR | 108.8 | 59.3 | 3.5 | 321.2 | 16.5 |
| 0.3 | 68.1 | ODR | 78.2 | 18.1 | 0.9 | 265.2 | 24.6 |
| 0.5 | 32.1 | ODR | 46.3 | 8.9 | 0.8 | 111.7 | 18.5 |
| 1 | 20.4 | ODR | 20.5 | 2.9 | 0.7 | 16.7 | 9.8 |

Fig. 13. FE parametric results (ABAQUS), as obtained by subjecting the outriggers to partial fire exposure. Time histories of (a) temperature and (b) out-of-plane deflections, by changing the exposure level of the B3 external surface \( R_L = 0 \).
4.3. Simplified estimation of the compressive buckling resistance of fire-exposed log-haus walls

At a final stage of the current research study, a first attempt to assess the fire endurance of the examined log walls via simplified analytical calculations was carried out. To this aim, existing formulations currently in use for structural timber members and assemblies in fire conditions were taken into account.

For the design of fire exposed timber structural systems, the Eurocode 5 [22] provides in fact a set of reduction coefficients $k_{mod,t}$ accounting for the design strength $f_d$ and stiffness $S_d$ (MOE or shear modulus) reduction due to charred timber:

$$f_d = \frac{k_{mod,t} f_{mod}}{S_{mod}}$$

$$S_d = \frac{S_{mod} f_{20}}{k_{mod,t}}$$

(4a)

(4b)

with $f_{20}$ and $S_{20}$ denoting the 20% fractile of material properties at room temperature, $S_{mod} = 1$ the partial safety factor for timber in fire, $k_t = 1.25$ for solid timber.

The so called “Reduced Properties Method”, in this sense, requires the use of specific $k_{mod,t}$ values - actually given by the Eurocode as a function of cross-section properties, stress distribution of the fire exposed side, charring depth, etc. - that should be properly assessed for the log-haus systems. In general terms, as far as the actual compressive load is moderately high (i.e. in the order of 20–60% of $N_{20}$), the ultimate condition for the main logs proved to occur in an average time of 25–45 min (with $F_2$ the prevalent failure mechanism). Higher mechanical loads tending to the mean buckling resistance of the walls (i.e. $N_{90}/N_{20} > 0.6$) resulted in failure times shorter than 20 min, with large out-of-plane deflections of logs but premature numerical loss of stability. The failure time of the examined log-haus geometry was hence found to be exponentially dependent upon

estimations of their actual response and fire endurance, being able to account for the fire endurance of a single log belonging to a full assembly. If the examined log-haus systems are assumed to behave as unprotected walls made of edgewise laminations of solid timber, hidden together by prestressing or adhesive bonding, $d_0$ should be calculated as (see also [30]):

$$d_0 = 9 + \frac{h}{25}$$

(7)

As a result, compared to the constant zero-strength layer of 7 mm recommended in Ref. [22] as well as to Eq. (6), it is expected that Eq. (7) would provide more refined estimations for the structural system investigated in this paper.

In this regard, aiming to provide some useful design provisions for log-haus systems, a last set of FE simulations was performed. Such thermo-mechanical simulations were carried out on the typical models earlier described in Sections 3, 4.1 and 4.2. The difference from previous simulations was given by (i) the fire exposure of lateral outriggers and by (ii) the imposed mechanical loading path. The lateral outriggers - based on Section 4.2. - were in fact assumed to be fully protected both on the external (i.e. B3) and internal (i.e. B1) surfaces, hence to act as fully rigid, ideal continuous restraints for the main logs throughout the full fire exposure phase. The main log wall alone, at the same time, was subjected to the reference ISO curve of Fig. 6.

In terms of mechanical loading, the same main logs were contemporarily subjected to a given uniformly distributed, compressive load with constant value equal to $N_0$. Various thermo-mechanical loading cases were then analysed, by changing the imposed $N_0$ value. The full set of loading configurations (10 in total) included the limit reference cases representative of (a) the main log-haus wall under compressive loads at ambient temperature (with $N_0 = N_{20} = 340$ kN/m the imposed compression load, corresponding to the mean buckling resistance of the wall, see Section 2.1) and (b) the same log-haus wall under ISO fire loading only (i.e. $N_0 = 0$ the applied compressive load). Being the current FE study aimed to assess the reliability of simplified analytical formulations for design purposes, an initial geometrical curvature was also assigned to the same models (with $u_0 = L/300$ its maximum amplitude, see Section 3.5).

For all the examined loading conditions, the collapse configuration and the corresponding failure time were hence detected using the criteria obtained from the FE results summarized in Sections 4.1 and 4.2, namely: (F1) numerical loss of stability; (F2) exceedance of the lateral deformation rate of 7 mm/min (control point P1); and (F3) exceedance of 29 mm in-plane axial deformations, whichever occurs first. The so obtained FE results are displayed in Fig. 14, where the labels ‘F1’, ‘F2’ and ‘F3’ in Fig. 14(a) provide the strictest failure criterion numerically detected.

In the same Fig. 14(a), the failure time is displayed for each of the examined FE loading conditions, as a function of the imposed compressive loading ratio $N_0/N_{20}$. There, the full-scale experimental result is also shown for comparative purposes.

Despite the assumption of ideal mechanical boundary conditions for the main logs (i.e. lateral outriggers not affected by fire exposure), a rather close correlation can be observed between FE estimations and the test results. For the mechanically unloaded log wall ($N_0 = 0$), a failure time in the order of 300 min was numerically obtained, being representative of the time for full charring of the main logs sections. Low compressive load levels (i.e. $N_0/N_{20} < 0.15$) resulted in failure mechanisms associated to large in-plane axial deformations of the walls (F3), as also in accordance with earlier FE results presented in Table 1 (see $R_0 = 1$ values). As far as the applied compressive load is moderately high (i.e. in the order of 20–60% of $N_{20}$), the ultimate condition for the main logs proved to occur in an average time of 25–45 min (with F2 the prevalent failure mechanism). Higher mechanical loads tending to the mean buckling resistance of the walls (i.e. $N_0/N_{20} > 0.6$) resulted in failure times shorter than 20 min, with large out-of-plane deflections of logs but premature numerical loss of stability. The failure time of the examined log-haus geometry was hence found to be exponentially dependent upon
the $N_f/N_{20}$ ratio itself, see the fitting curve in Fig. 14(a).

The same FE comparative results are hence proposed in Fig. 14(b), where the charring depth ratio was used to replace failure time data. Given the nominal depth $b = 90$ mm of timber logs, as well as the failure times proposed in Fig. 14(a) for all the examined FE configurations, the charring depth $d_{\text{char}}$ was first calculated for each of them (i.e. based on the numerically estimated isotherm at 300 °C), and then divided by the nominal depth $b = d$ of logs. As shown, a non-linear decay of the actual load-carrying capacity of the examined log wall geometries was again observed, compared to the expected buckling resistance at room temperature. Based on these preliminary considerations and FE studies, it is clear that the actual in-plane compressive buckling resistance of a fire exposed, unprotected log-haus wall would require specific investigations and appropriate formulations of practical use to design.

To this aim, some further comparative results are proposed in Fig. 14(c). There, the $d_{\text{ef}}$ to $d$ effective section ratio was numerically derived for the analysed log walls, based on the expected buckling resistance at room temperature and the corresponding charring rate. In particular, the relative charring depth was first numerically calculated as in the case of Fig. 14(b), while in terms of $d_{\text{ef}}$ values, FE results were derived from Fig. 14(a). For all the examined cases, the so imposed $N_{\text{ef}}$ loads were considered to extrapolate - from Eq. (1) - the corresponding $d_{\text{ef}}$ size, then divided by the nominal depth of logs. Non-dimensional FE results are hence compared in Fig. 14(c) with their linear interpolation curve, as well as with further analytical results provided by Eqs. (5) and (6) or (7), respectively. In doing so, failure time values in Eq. (5) were derived from FE simulations (i.e. Fig. 14(a)), with $p_{\text{char,n}} = 0.65$ mm/min.

As shown, analytical estimations derived from Eq. (6) - suggested in Ref. [25] for timber beams in bending with compressed side under fire - proved to offer rather conservative predictions (with scatter up to 35–40%), compared to FE simulations. This effect can be first justified by the actual performance of log-haus walls, being characterized by (even partial) contact interactions among the overlapping logs. The overall bending performance of the examined log-wall (see also [15–17]) is in fact theoretically expected to lay within the limit conditions of (i) a fully monolithic wall under in-plane compression and (ii) a single timber member subjected to out-of-plane lateral buckling. As such, Eq. (6) fully disregards the structural interaction among adjacent logs and can only represent a lower limit estimation of the actual fire performance of multiple log assemblies belonging to a full structural system. Further reason of such a scatter could also derive from the effect of cross-section aspect ratios for log-haus systems, compared to beam properties examined in Ref. [25]. A $d_0$ value of 30.5 mm was in fact derived from Eq. (6).

Enhanced FE-to-analytical comparisons were indeed obtained, with maximum scatter in the order of 10% only, as far as the constant 7 mm layer given by Eurocode 5 or Eq. (7) were taken into account, see Fig. 14(c). In this sense, for design purposes, Eq. (7) could represent a rational assumption for preliminary and practical considerations on the fire performance of log-haus systems. The actual fire buckling resistance of a given log-haus wall could be estimated via the simplified RCSM formulations currently in use for timber structures in fire, see Eqs. (1)–(5) and (7).

At the current stage of the research study, however, further aspects should be properly assessed for a reliable calibration of the proposed design method, being the log-haus wall here investigated representative of only a case study. Further issues could in fact arise as far as single or double door and/or window openings are also present, as well as the distance of outriggers modifies. In this regard, it is hence expected that a further extension of the actual research study, inclusive of a wide set of geometrical configurations for timber log-haus walls, could provide...
confirmation to this preliminary outcomes, and leads towards the defini-
tion of a general design formulation.

5. Summary and conclusions

The paper explores the compressive buckling performance of unpro-
tected timber log-haus walls in fire conditions. To this aim, past results of
a full-scale experimental test carried out on a specimen representative of
a typical geometrical configuration of technical interest were first pre-
sented and critically discussed, aiming to provide evidence of its buckling
performance in fire conditions. The primary observations of the experi-
mental investigation are that:

- the actual buckling performance of log-haus walls in fire conditions is
  markedly affected by the intrinsic assembly features of the examined
  structural typology (i.e. disconnected logs assembled to compose a
  wall, geometrical properties of logs, absence of mechanical joints
  between them, etc.). As such, the observed bending performance
typically lays between the limit configurations representative of (a) a
fully monolithic wall under in-plane compressive load and (b) a set of
disconnected, single timber members subjected to out-of-plane
bending;
- the orthogonal walls, acting as lateral outriggers for the main logs
  composing the wall, proved to have a key role on the overall per-
formance of the full-scale specimen, being subjected to fire exposure
and providing appropriate bracing to the main logs.

The same experimental test results were then used for assessment and
validation of a reference, full 3D solid, thermo-mechanical Finite-
Element numerical models carried out in ABAQUS, aiming to further
explore the structural behavior of the examined structural system, as well
as to derive some preliminary considerations for design purposes. In such
FE models, a key role was given to mechanical contact interactions as
well as input parameters for timber constitutive law, hence allowing the
simulation of possible failure mechanisms (i.e. overturning and uplift of
logs) as well as damage propagation in timber. A rather close correlation
was generally observed between FE estimations and the corresponding
test results, giving evidence of temperature distributions and typical
deformations, as well as suggesting further extended parametric studies.

Based on a set of additional FE simulations derived from the reference
numerical model, the effects of various influencing parameters were in
fact properly emphasized for the examined structural typology.

Compared to available research studies for the buckling performance of
log-haus wall at ambient conditions, in particular, it was shown that
fire loading effects should be properly taken into account in the esti-
mation of the actual resistance of such systems. The FE investigations
partly summarized in the paper highlighted in fact that:

- a direct correlation exists between the overall resistance of log-haus
  walls and the fire exposure of orthogonal logs, acting as outriggers.
  Even in presence of slight in-plane compressive loads only, carpentry
  joints at the logs ends should be properly protected, to avoid pre-
mature loss of stability of main logs and consequent collapse;
- differing from the buckling analysis of the same structural typology at
  room temperature (see for example [15–17]), the presence of possible
  initial geometrical curvatures proved to offer minor structural effects
  on the observed fire performance for the investigated log-wall, being
  the effects of fire loading predominant. The observed trends, how-
ever, are structurally related to a series of key aspects, such as the size
  of the wall and of the single logs, the distance of restraints, the me-
  chanical loading ratios compared to the fire loading. Since such initial
  curvatures could be both inward or outward relative to the fire
  exposure, in addition, for design purposes the effects of these
  geometrical defects in fire conditions should be properly assessed;
- due to typically high b/h ratio of timber logs composing full walls and
  assemblies, a marked reduction of the buckling resistance at room
temperature was observed for the same reference specimen in fire
conditions, when considering various compressive loading ratios.
- while current EN standards and regulations assess the fire perfor-
  mance of a given wall by limiting the in-plane deformation rate or
deformation only, for most of the FE configurations presented in the
  paper - due to the intrinsic features of log-haus systems and the
  presence of only contact interactions among overlapping logs - it was
  shown that the out-of-plane deformation rates generally represent a
  crucial parameter, especially when larger than 6–7 mm/min. As such,
  they should be considered as failure criterion.

In conclusion, a possible application to log-haus systems of the
“Reduced Cross-Section Method” currently provided by the Eurocode 5
[22] for estimating the fire resistance of timber members was also
proposed.

It was shown that reliable and conservative estimations could be
obtained as far as RCSM provisions for unprotected walls made of
edgewise and pre-stressed solid timber laminations are taken into ac-
count. Beside the promising comparative results proposed in the paper
for the reference case study, however, at the current stage of the research
study further extended FE investigations inclusive of a wide set of
generical and mechanical configurations for log-haus walls should be
considered (i.e. by varying the distance between outriggers, the log cross-
sectional properties, the presence and number of openings, etc.), so that
the input parameters and accuracy of RCSM extension to log-haus sys-
tems could be further calibrated and validated.

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References

Seismic Actions and Rules for Buildings, CEN (European Committee for
Standardization), Brussels, Belgium, 2004.
General Rules, Seismic Actions and Rules for Buildings, CEN (European Committee
Committee for Standardization), Brussels, Belgium, 2012.
investigation of Blockhaus shear walls subjected to in-plane seismic loads, J. Struct.
www.series.upatras.gr/dev.
system via experimental testing and analytical modelling, Construct. Build. Mater.
102 (2) (2016) 1127–1144.
[8] EN 1993-1-1, Eurocode 3-Design of Steel Structures - Part 1: General Rules,
Seismic Actions and Rules for Buildings, CEN (European Committee for
Standardization), Brussels, Belgium, 2005.
Seismic Actions and Rules for Buildings, CEN (European Committee for
Standardization), Brussels, Belgium, 2005.
capacity of axially loaded wood studs under simulated fire exposure, in:
Proceedings of the International Conference on Timber Engineering, Seattle, USA,
structures under standard fire exposure, Fire Technol. 52 (4) (2016) 1015–1034,


[21] Simulia, ABAQUUS v.6.12 Computer Software, Dassault Systèmes, Providence, RI, USA.


