Available online at www.eccomasproceedia.org
Eccomas Proceedia COMPDYN (2017) 1464-1473

ECCOMAS
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COMPDYN 2017
6th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Rhodes Island, Greece, 15–17 June 2017

NUMERICAL INVESTIGATION OF THE IN-PLANE SEISMIC PERFORMANCE OF TIMBER LOG-HAUS WALLS WITH REINFORCED DOVETAILS

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Keywords: timber log-haus walls, seismic performance, experimental tests, Finite-Element numerical modelling

Abstract. *The paper investigates the structural response and vulnerability of timber log-haus walls under in-plane seismic loads. Careful consideration is given to the structural efficiency of additional metal fasteners introduced within the thickness of timber log-walls. To this aim, small-scale specimens are analysed, in order to preliminary explore the actual potential of the proposed solution. The steel reinforcing dovetails, as shown, are aimed to improve the in-plane stiffness and ultimate resistance of single log-walls, since traditionally obtained by overlapping simple timber logs by avoiding the use of mechanical fasteners. Taking advantage of past experimental tests, some conclusions are drawn on the observed structural performance of the examined structural typology, compared to unreinforced timber log specimens. A key role is assigned to refined Finite Element numerical models, in which the mechanical properties, as well as possible contact interactions among the timber logs, and the contribution of reinforcing steel dovetails are properly taken into account.*

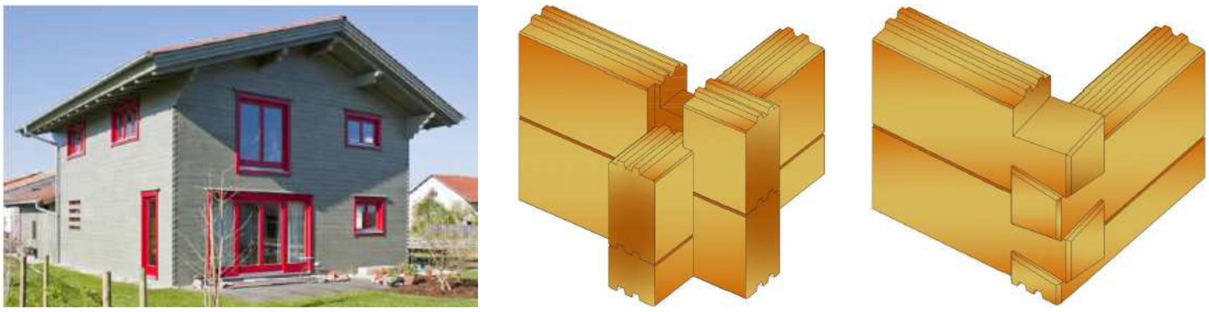
Peer-review under responsibility of the organizing committee of COMPDYN 2017.
doi: 10.7712/120117.5506.16995

1 INTRODUCTION AND STATE-OF-THE-ART

This paper investigates the structural response of timber log-haus wall under in-plane seismic loads, with careful consideration for the efficiency of steel reinforcing dovetails aimed to improve their overall resistance.

Despite the ancient origins, log-haus systems are used in daily practice for the construction of wooden houses as well as commercial buildings (see Figure 1(a)), that are typically located in urban regions with high seismic hazard such as Japan, the Mediterranean, etc. In current practice, these structural systems are obtained by placing a series of timber logs, horizontally on the top of one another, so as to form the a log wall. The mechanical interaction between the basic timber components, orthogonal walls, or walls and inter-storey floors is then provided by simple mechanisms such as simple joints and contact surfaces, while the use of metal fastener is reduced in them to a minimum (see for example Figures 1(b) and (c)).

In terms of design of such systems, currently available standards for seismic resistant timber structures do not provide specific analytical models and recommendations for their appropriate verification, see for example [1, 2]. In addition, especially under specific boundary and loading conditions, the actual seismic behaviour and vulnerability of timber log-haus systems still requires detailed investigations, aimed to assess the effects deriving from the interaction of several aspects, such as friction phenomena, nonlinear behaviour of the typical joints under cyclic loads, etc. In the last years, some research studies have been focused on the seismic performance of log-haus systems, both at a component or at a full-scale and assembly level (see [3-9]). As a general outcome, it was pointed out that as far as no mechanical fasteners are used between the timber logs, the resistance of such systems is mainly given by contact interactions and friction mechanisms, in which production tolerances, possible gaps or initial geometrical imperfections can have a key role in their actual response.



(a) (b) (c)

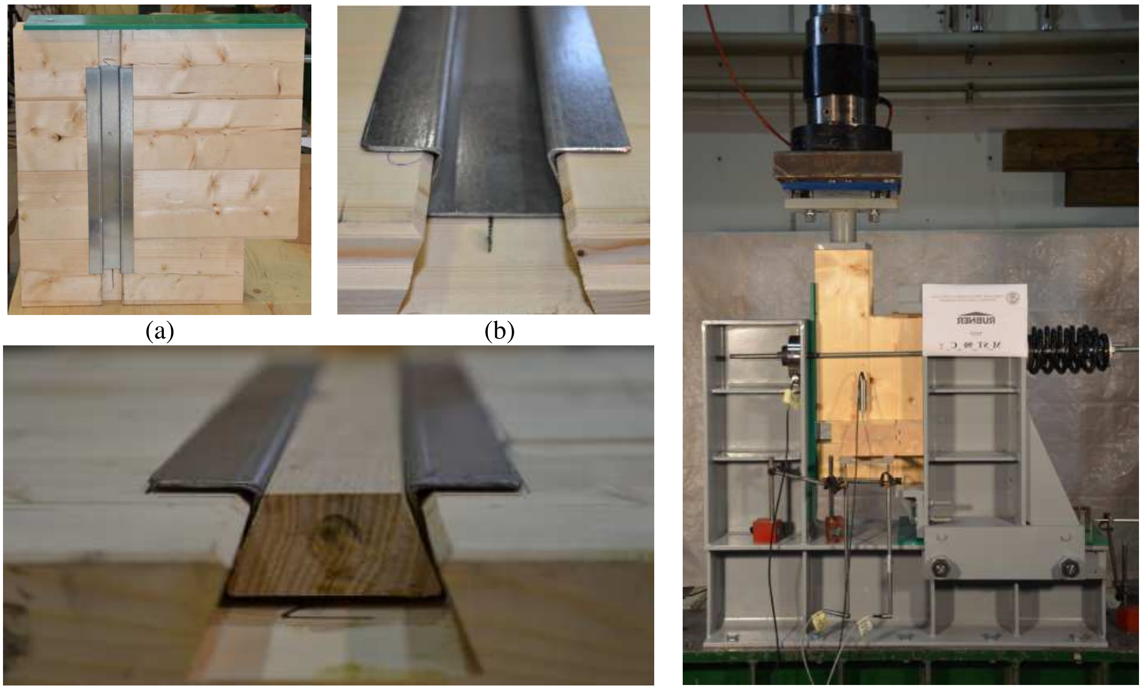
Figure 1: (a) example of log-haus residential building, with detail of typical (b) 'Standard' and (c) 'Tirolerschloss' corner joints at the interception between orthogonal log walls (www.rubner.haus.com).

In this paper, taking advantage of earlier experimental studies [7-9], the attention is focused on the assessment of the seismic performance of steel-reinforced log-haus walls. In them, a cold formed metal dovetail is introduced within the thickness of the wall, aiming to improve the interlocking between adjacent logs and to improve their stiffness and resistance under in-plane seismic loads. As a part of an ongoing research study, the investigation is carried out on small scale specimens, being representative of a portion of wall. Finite Element numerical models able to reproduce their geometrical and mechanical features, as well as the actual restraint and loading condition as a part of a log wall, are hence carried out in ABAQUS [10]. Validation of FE results is then provided towards the experimental tests, including some major considerations to be further extended for a full optimization of the examined reinforcing solution.

2 REFERENCE SPECIMENS

2.1 Past experimental study: summary of test methods and results

Quasi-static monotonic and cyclic experimental tests were carried out in 2012, at University of Trento (Italy), on a set of small scale specimens representative of the typical carpentry joint in use for log-haus structural systems. The displacement controlled tests were aimed at obtaining the load-displacement relationship for different geometries of corner joints in use for log-haus systems (i.e. Figures 1(b) and (c)), both in compression and in tension, according to a common testing protocol. Major outcomes of the full experimental study can be found in [7-9]. The typical specimen, representative of a small portion of log-haus wall, basically consisted of 4 timber logs, C24 the resistance class of spruce [11], with b=90 × h=160mm the nominal cross section and 500mm their length (Figure 2(a)).



(a) (b) (c) (d)

Figure 2: Reference experimental tests, with evidence of (a) typical small-scale specimen, (b)(c) steel dovetail reinforcements for S01 and S02 specimens, with (d) overview of the test setup [7, 8].

Within the full experimental investigation on carpentry joints, two monotonic (compressive) tests were dedicated to the assessment of the structural efficiency of a steel dovetail reinforcement to be installed within the thickness of a typical log wall (with 80×26mm the overall dimensions, 0.2mm the thickness and S235 the resistance class [12]), see Figures 2(c) and (d). A dovetail profile, consisting of a cold formed metal sheet shaped section, was in fact used in the here labelled 'S01' specimen, and designed to match the shape of corresponding groove cuts on the main timber logs (Figure 2(b)). In order to allow shrinkage deformations in timber, the length of this steel reinforcement was kept equal to 450mm (≈ 3/4 the height of 4 overlapping logs). No mechanical connections were used to provide the structural interaction between the steel profile and the adjacent logs, hence only friction and contact mechanisms were taken into account to exploit the potential of the steel profile.

For the 'S02' specimen, the same reinforcement technique was further assessed, but an additional hardwood member (D50 the resistance class [11]) was inserted within the metal dovetail, to act as stiffener and avoid local bending and failure of the dovetail itself, hence providing higher stiffness and resistance to the full specimen (see Figure 2(c)). As in the case of the S01 system, the structural interaction between the hardwood stiffener and the steel profile was given by contact mechanisms only.

In order to reproduce for the S01 and S02 specimens the actual working condition of a log-wall under compression (i.e. deriving from roof and inter-storey floors) and in-plane seismic loads, a bespoke loading setup was developed (Figure 2(d)), being inclusive of a L-shaped reaction frame and a vertical restraining system, connected to the reaction frame by using two M16 threaded rods and steel spring (with 100N/mm their stiffness). A total initial compression of 10kN was imposed to these rods and kept fix during the experiments. The assigned value was chosen since well representative of a uniform compressive load in the order of 20kN/m, hence to account for the typical compressive loading rate of a full-scale log-wall belonging to a 2-storey residential building. The typical specimen was positioned within the test setup, as summarized in Figure 2. Low-friction plates consisting in Polyzene foils [13] were finally interposed between the top/bottom logs and the steel setup. Given the imposed lateral displacement to the top log of each specimen, a linear displacement transducer was fixed to each log, to monitor their relative slip.

3 FINITE ELEMENT NUMERICAL INVESTIGATION

3.1 Modelling approach and solving method

Taking advantage of small scale experimental results presented in [7, 8], a Finite Element numerical investigation was carried out in ABAQUS [10], aiming to assess the monotonic performance of dovetail reinforced specimens.

In doing so, two FE models were implemented, being representative of the S01 (M01 model, in the following) and S02 (M02 model) specimens respectively. The difference between M01 and M02 consisted in the presence or removal of the hardwood dovetail stiffener (and related mechanical interactions).

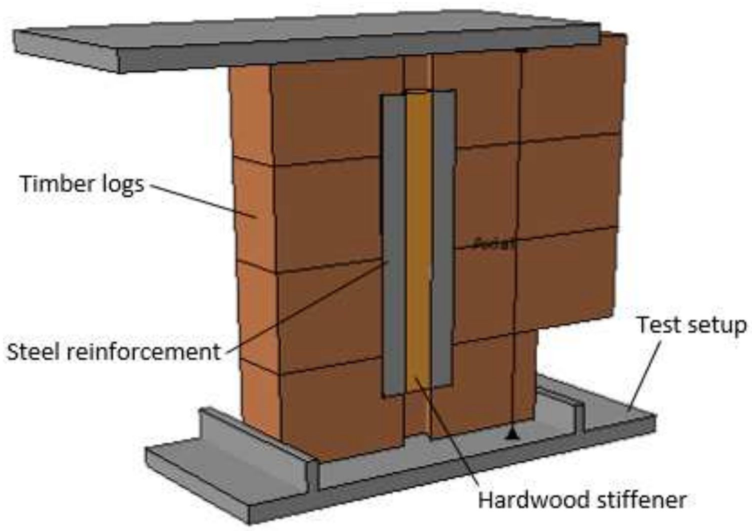


Figure 3: FE modelling of S01 and S02 specimens (example proposed for the M02 model), ABAQUS [10].

The typical FE model consisted in fact of 4 component types (see Figure 3), representative of (i) 4 timber logs, (ii) the steel dovetail reinforcement, (iii) the hardwood dovetail stiffener (for the M02 model only) and (iv) the main components reproducing the experimental setup

(L-shaped contrast frame plus rods and springs for the application of the assigned compressive load). Compared to ideal boundary conditions for the tested log assembly, the FE description of the setup components was preferred, aiming to correctly reproduce the actual in-plane performance of the reference specimens.

As such, 3D solid elements (8-node C3D8R type and 6-node C3D6R type) were used for all the FE components, with the exception of the steel rods and springs only, consisting of *axial* mechanical connectors with equivalent mechanical properties. Mesh pattern and size was then set in order to maximize the computational efficiency of FE models, but preserving the accuracy of predictions, especially in the regions of contact between the timber logs and the steel reinforcement. The final FE assembly consisted of 18,000 solid elements and 70,000 DOFs.

Based on earlier works (see for example [4]), each timber log was described with a regular $b \times h$ cross section and the characteristic small protrusions and tongues along the top and bottom surfaces were reasonably neglected. The nominal geometry was considered for the other FE components. A key role was indeed assigned to contact interactions, so that the actual performance of such specimens could be properly estimated. *Surface-to-surface* contact interactions were in fact implemented at the interface of all the adjacent FE components. The full set of interactions was basically subdivided into four groups, namely accounting for possible contact mechanisms among:

a) overlapping timber logs,
b) the top/bottom timber logs and the steel substructure (with interposed Polyzene foils),
c) each one of the timber logs and a portion of the steel dovetail profile,
d) the steel dovetail reinforcement and the hardwood stiffener (when present)

For all these surface contact algorithms, a general penalty approach was used, with minor variations in the basic input parameters associated to relative slidings only. Possible tangential displacements between the interested surfaces was in fact allowed (*tangential behaviour*), with μ the static friction coefficient assumed, for the (a)-to-(d) contact typologies, respectively equal to 0.5 [4], 0.2 [13], 0.6 and 0.9. For the (d) type interaction, a fictiosus reference value was calibrated to account the almost total lack of possible sliding at the hardwood-to-steel dovetail interface. During the experimental assembly of the reference specimen, a steel hammer was in fact used to position the hardwood member within the steel profile.

In the direction perpendicular to all the involved surfaces, a *'hard' normal behaviour* was also used, so that compressive stresses could be transferred among two FE model components in contact (as far as any kind of damage in materials occurs) and misleading overlapping could be avoided. Possible detachment of involved surfaces when subjected to null compressive pressures, was also taken into account, so that the influence of partial uplift and overturning of logs on the overall in-plane response of the examined specimen, as well as detachment of the steel dovetail reinforcement from timber logs, could be properly considered.

Concerning the mechanical characterization of materials, an equivalent isotropic constitutive law was considered for C24-class spruce. Due to the loading condition of timber logs, a mean average modulus of elasticity (MOE) in the direction parallel to grain $E_{//}$= 11556MPa (11000MPa the mean nominal value recommended by [11]) and a shear modulus G = 617MPa (690MPa the mean nominal value [11]) were taken into account, as experimentally derived in [14]. Possible failure mechanisms in the same logs were also accounted, in the form of a simplified Von Mises plastic law. To this aim, yielding stress was defined by considering the occurrence of possible crushing mechanisms in logs, as expected especially for the portions of logs subjected to contact with the steel reinforcement. Following [14], the mean value of compressive stress in the direction parallel to grain was hence assumed, being experimentally calculated for the same timber logs in the value of 33.59MPa (mean value given in [11] equal to $\approx f_{c,0,k}/0.7$= 30MPa).

An idealized elasto-plastic mechanical law was then implemented for all the steel components, with E_s= 210GPa the MOE, ν_s= 0.3 the Poisson' ratio, $\sigma_{y,s}$= 235MPa the yielding stress and $\sigma_{u,s}$= 360MPa the ultimate value [12].

For the M02 model only, an additional constitutive behaviour was also accounted for the hardwood dovetail stiffener. Due to lack of experimental calibrations at the material level, an equivalent elasto-plastic law was considered for hardwood, with $E_\perp$= 930MPa the MOE in the direction perpendicular to grain of D50-class timber [11]. Yielding stress was set equal to $\approx f_{c,90,k}/0.7$= 8.85MPa, being the latter value representative of the mean compressive resistance in the direction perpendicular to grain [11].

For both the M01 and M02 models, the typical simulation consisted in two sub-steps. First, the initial compressive load was imposed to the specimens, via the restraining steel rods and springs, as in the case of the reference tests. On the so pre-loaded FE models, a dynamic simulation with quasi-static increase of the in-plane shear load on the top log was hence carried out, up to the attainment of a lateral load equal to 40mm. In accordance with the experimental methods and results presented in [7-9], the failure condition for the examined specimens was set at the conventional displacement of the top log larger than 40mm, corresponding to a lateral drift of 0.0625H, with H= 640mm being the total height of the four overlapping logs. In doing so, maximum stresses and deformations in all the FE components were continuously monitored, aiming to verify the overall behaviour of the examined connections, as well as the accuracy of simplified mechanical assumptions in materials early described.

3.2 Discussion and assessment of FE results towards experimental tests

Generally, a rather interesting agreement was observed for both the FE models and corresponding test results, in terms of overall performance as well as local phenomena in each component. As a key advantage deriving from advanced FE methods, the numerical predictions here summarized also allowed to further explore the earlier experimental findings.

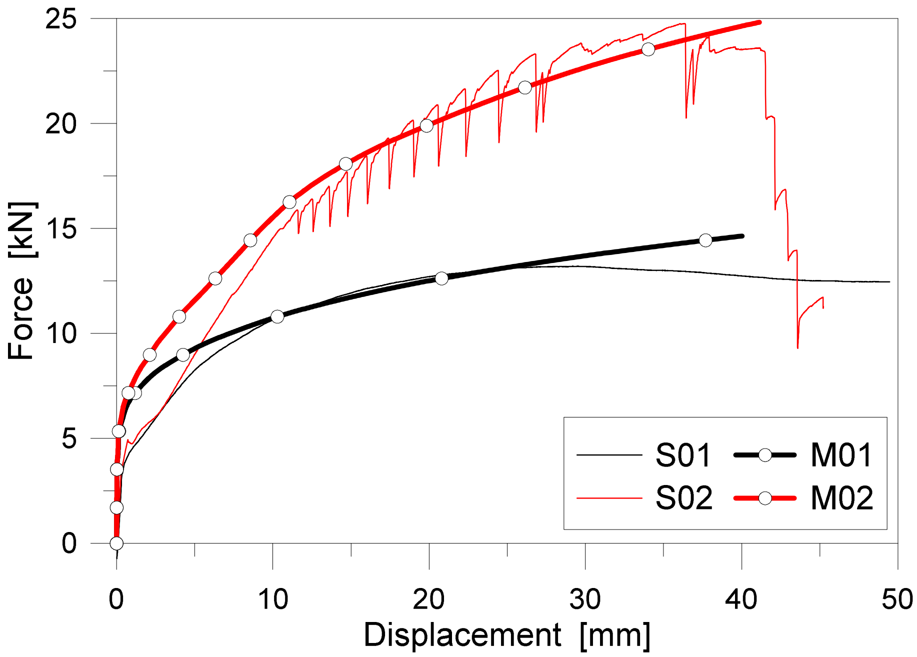


Figure 4: Load-displacement response of the examined specimens, as obtained from the reference experiments [7, 8] and via numerical simulations (ABAQUS [10]).

A load-displacement comparison is proposed in Figure 4, for both the specimen typologies, as obtained by monitoring the in-plane lateral displacement of the top log. As shown, the FE models proved to offer reliable estimations for the examined specimens, despite minor scatter due to possible mechanical (i.e. material properties) or geometrical simplifications in the description of the specimen components as well as of the test setup (i.e. grooves and tongues along the top logs, presence of possible small gaps at the interface between the specimen components, etc.). This scatter can be noticed especially in the first loading phase, i.e. when sliding and progressive uplift of logs begins. In any case, see Figure 4, a totally different in-plane response was observed for the S02 specimen, compared to the un-stiffened S01 system. The test on the S02 system showed in fact an increase of maximum resistance in the order of 150%, compared to the S01 specimen, up to 25kN.

A further qualitative comparison is proposed in Figures 5 and 6, where the typical deformed shape of the full specimens and the reinforcements only are emphasized, as experimentally and numerically obtained at an imposed in-plane lateral displacement of 40mm for the top log. Basically, as also derived from Figure 4, the hardwood stiffener proved to have a key role on the structural benefits given by the steel profile. From Figure 5, in particular, the deformed shape of steel reinforcements for the S01 and S02 specimens is shown, with evidence of the profile ultimate configuration.

(a) (b) (c)

Figure 5: Typical deformed shape experimentally observed for (a) S01 and (b) S02 specimens, with (c) evidence of the final configuration for the dovetail reinforcements [7, 8].

An almost identical overall behaviour was also numerically predicted, see Figure 6, where major discrepancies in the response of the M01 and M02 specimens are further emphasized in terms of stress distribution and local deformations in each FE component. As shown, in particular, crushing mechanisms where typically observed to occur in the timber logs, in the regions of contact with the steel profile.

While buckling occurred for the M01 steel dovetail, with limited benefits on the overall in-plane resistance of the full specimen, the M02 model proved to take advantage of the hardwood stiffener, as also in line with the earlier experimental findings. In the latter case, minor bending deformations only (mostly negligible) were in fact observed in the steel profile, as a consequence of local crushing initiation in the hardwood stiffener (see Figures 5 (c) and (d)).

Based on the well promising results here summarized, it is hence expected that the use of steel dovetails for full-scale log-haus walls could be further explored and optimized, aimed to improve the seismic performance of the examined structural typology.

Careful consideration, to this aim, should be given to the actual size of the steel profiles, as well as to their optimal position within a 3D assembly representative of a full building. A sensitivity analysis able to emphasize the potential of such reinforcements for log walls with multiple door/window opening, as well as subjected to various combinations of compressive loads and in-plane lateral forces should be also taken into account, being the actual seismic response of log-haus systems strictly dependent on a multitude of mechanical and geometrical aspects.

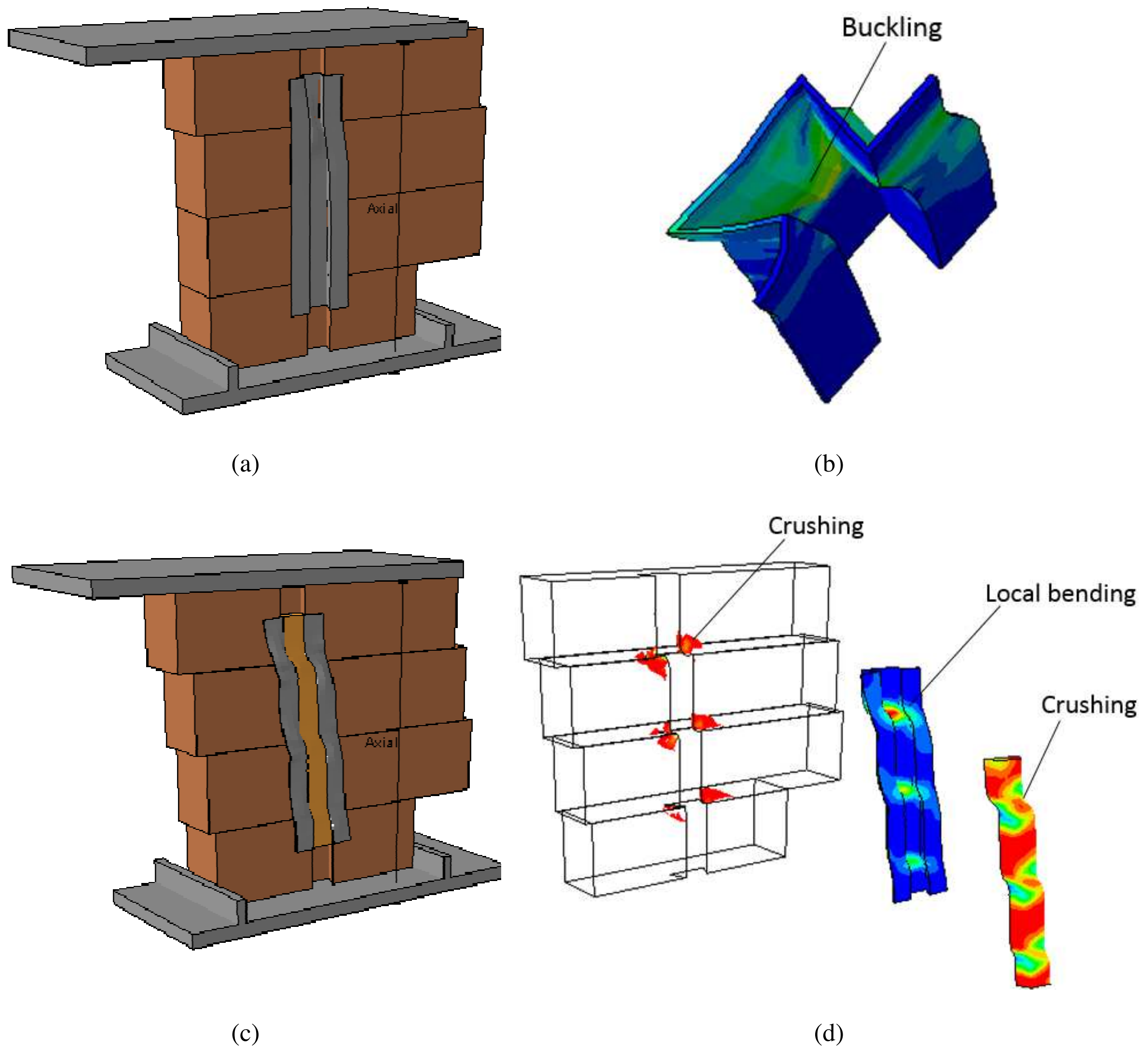


Figure 6: FE numerical response of M01 and M02 specimens, at an imposed displacement of 40mm (ABAQUS [10]). Overall deformed configuration and performance of single components (detail) for (a)(b) the M01 and (c)(d) the M02 models.

4 CONCLUSIONS

In this paper, the in-plane seismic performance of log-haus walls reinforced by steel dovetail profiles has been explored via Finite Element (FE) numerical models. Taking advantage of earlier experimental studies carried out on small scale specimens representative of a portion of full log-haus wall, 3D models able to capture the actual mechanical performance of the examined systems have been described in ABAQUS.

As shown, a key role was given to multiple aspects, including contact interactions among the FE model components as well as appropriate boundary/loading conditions, aimed to reproduce the actual working condition of such systems. Based on the rather close correlation between FE estimations and the reference experiments, it is expected to further optimize the explored reinforcing solution.

ACKNOWLEDGEMENTS

Part of this research project was carried out within a scientific collaboration between the authors and Rubner Haus SpA (www.haus.rubner.com), which is gratefully acknowledged for financial support. In this regard, Dr. Annalisa Battisti is also acknowledged for sharing data and results of the past experimental investigations carried out ad University of Trento, Italy, as well as for providing technical support in their interpretation.

DPC-ReLUIS is finally acknowledged for partially funding the research activity within the framework of the 'PR4-Timber structures' Italian project.

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