A post-seism rebuilding proposal for civil masonry buildings
A case study

Matteo Bagnoli, Università degli Studi Guglielmo Marconi, Rome, Italy

ABSTRACT. The seismic events which affected Central Italy in the last ten years, in the city of L’Aquila 2009 and Amatrice in 2016, had catastrophic consequences, in particular concerning the loss of human lives and buildings in old towns. The use of good masonry was not sufficient and the building materials, in many cases of low quality, did not resist strong shakings, crumbling and causing the sudden collapse of the buildings. The real problem for small villages having an historic importance, like in Abruzzo municipalities, was rebuilding, in particular civil houses with one or two overground floors. The case study treated in my master thesis consisted in designing a low-cost wood house, energetically self-consistent, simple and easily assemblable, for the post-seismic phase. This should be exploitable not only in the emergency phase, i.e. as a temporary solution, but also as definitive solutions, which on one side does not deface the architecture of affected old towns, and on the other side does not bring to the birth of a “new town”. This could be accomplished by preserving the external walls of a partially collapsed civil house, tying them with a new internal wood structure.

KEYWORDS: Case study, living solutions, seism, temporary living modules, wood structures

Introduction

History and architecture are the main feature of many old towns hit by earthquake. Therefore, post-seism living solutions should not have a strong impact in the existing scenario, so to appear as “new towns” (entirely new parts of populated areas, with completely different features from the existing buildings), but it should rather preserve an architectural continuity with the old town.

My direct experience on the field in the geographical area of L’Aquila in August 2009, which was the subject of my Bachelor thesis research, allowed me to verify the strong and unpleasant
impact of entirely new districts built in the city centre, either defacing the continuum of the existing buildings or replacing whole green areas, so far uncontaminated.

For this reason, I chose to continue my research in the field of seismic engineering proposing a new post-seismic living solution. This consists of a wood structure made of beams and pillars, to be internally strongly tied to external undestroyed walls of a house hit by the seism and only partially collapsed, in order to preserve the historical skin of the pristine building. In practice, the uncollapsed walls would keep the memory of the pristine house, thus largely preserving the pre-seismic aspect of the old town. This project could be applied not only in small areas within old towns, but also in external areas. In this way, it would be possible to give a new dignity to both the restored houses, but also to the corresponding streets of the villages hit by the earthquake. This idea should represent, at least in some specific cases, a practical post-seismic living solution allowing historical towns and villages hit by the earthquake to start back their life in a completely different, and in particular much faster way with respect to the traditional “how it was, where it was” approach.

The proposed solution is suitable for partially collapsed houses of a maximum number of three floors, preserving at least parts of the perimeter walls. In such situations, the owners, after the needed authorizations, should be able to quickly remove the debris within their houses (subjected to selection and recovery of historical materials), and afterwards begin the rebuilding. Since the basic idea is the preservation of the intact part of the perimeter walls, which guarantees the wanted historical external aspect of the buildings, some problems arise concerning the restoration of such walls and their stabilization and tie to the new internal wood structure. The latter aspect is particularly important since it must guarantee a positive behaviour of the new structure in the case of further shaking events, which should consist in the absorption of the seismic energy not only for itself, but also for the tied external walls. Indeed, the main aspects here taken into account concern the connections between the external walls and the new wood structure, which represent the most critical feature to be evaluated during the design stage.

The emergency phase: plans for the living solutions

The 6th April 2009 earthquake involved 57 municipalities, mostly in the province of L’Aquila (42), but also in those of Teramo (8) and Pescara (7), affecting a total population of 150 thousands inhabitants. A large part of the building stock resulted condemned after the earthquake: 32% for residential housing, 21% for public buildings, and 54% for historical buildings.

About 67.5 thousands persons resulted homeless: in February 2012, 33476 of them came back to their houses, while 21941 were accommodated elsewhere at the State expenses. The very first phase, lasted five-six months, consisted in a quick set up of temporary solutions (tents or hotels) for the homeless population. Afterwards, two distinct rebuilding programmes started in favour of displaced persons: C.A.S.E. (Complessi Antisismici Sostenibili ed Eocompatibili – anti-seismic sustainable and environmentally-friendly complexes) and M.A.P. (Moduli Abitativi Provvisori – temporary living solutions). The first programme (C.A.S.E.) concerned 19 areas in the suburb of L’Aquila, where 185 three-floor overground prefabricate buildings were built, for a total of 4600 flats, hosting about 17 thousands persons. Each prefabricate building was placed over a reinforced concrete plate, isolated from the ground through anti-seismic devices called “at sliding pendulum”, which guarantee the needed elasticity under horizontal strains.
The second programme (M.A.P.) concerned 107 settlements of single-floor wood small houses, which were realized in the remaining 53 municipalities of the seismic crater and in 29 villages in the province of L’Aquila, for a total of about 10 thousands hosted persons (Fusero, 2010).

In particular, I analysed the situation of a few small municipalities near L’Aquila, like Pizzoli and Montereale, interacting with local technicians to identify suitable areas which could host the temporary living solutions requested by the municipalities themselves, as well as to plan primary and secondary urbanization works.

In particular, I actively took part to the planning and map arrangement of 12 modules requested by the Montereale municipality. This project, realized by a team supporting the local municipality, was adopted in 2009 and two years later was realized (Bagnoli, 2015).

The most important aspect of these temporary living solutions consists in their unquestionable positive response after nine years from their realization and after the seismic events which hit the Center of Italy in August 2016. Indeed, after an accurate analysis performed by myself in July 2017, the results of which were reported in my Master thesis, it was clear that no damages were present in all structural and non-structural components (reinforced concrete basements, external frames, connections and roofing) of the buildings.

The biggest drawbacks of these temporary living solutions are mostly two. The first consists of the temporary character of the living solutions, which, however, is not fully respected by the municipalities of the seismic crater, nowadays still using such solutions to meet the problems connected to housing emergency or to assign them to the “Edilizia Residenziale Pubblica” (public housing). The second is related to the exploitation of large green areas, fully occupied by these settlements, the “new towns”, which disfigure the beautiful landscape of Abruzzo.

A project for an integrated wood solution for masonry buildings in old towns

The proposal described into the case study was possible by deepening concepts concerning the dynamics of the structures. First of all, the definition of simple oscillator (based on the Single Degree Of Freedom – SDOF – model) was analysed by considering all the main aspects characterizing it and acquiring all the peculiar features of the various possible cases. Another fundamental aspect of structural dynamics, which was deeply analysed, consisted of the Response Spectra, i.e. the diagram of maximum displacement, rate or acceleration as a function of the oscillation period, which allowed differences between real and “project” earthquakes to be understood and acquired.

Afterwards, an analysis of structures made of the three main seism-resistant materials (reinforced concrete, steel and wood) was performed, by highlighting the corresponding pros and cons. Moreover, the hierarchy of resistance, a very important project methodology, especially in seismic areas, was examined. Such methodology, which was introduced by the law, aims at obtaining ductile and dissipative structures in seismic areas, so to identify the elements devoted to dissipate the input seismic energy. These elements, in the case of wood structures, are the connections. Therefore, in order to obtain a ductile behaviour, it will be necessary to assign a larger resistance to fragile mechanisms.

On the basis of the state-of-the-art concerning on one side the seismic-resistant structures and on the other side the use of wood, especially in temporary living solutions, allowed me to perform a precise and detailed design, subject of my Master thesis and of the present publication.
The structural solution proposed, in fact, consists of the insertion of a wood structure made of beams and pillars, strongly tightened to the external walls of an existing building, partially damaged by the earthquake. Considering the previous experience matured in the municipality of Montereale (province of L’Aquila), this has been assumed to be the location of the project for the sake of simplicity.

The project did not concern the sole wood structure, but rather the whole integrated solution, where also the external elements, i.e. the external walls to be preserved, were taken into account. A low value for the structural factor was chosen, so to avoid large plastic displacements. However, this would involve the damage of the external masonry wall, but this was prevented by providing the insertion of steel struts, which allowed the stiffening of the wood structure. This solution allowed plastic displacements to be prevented and, at the same time, external walls to be safely preserved.

The case study

The proposed case study is articulated in six principal phases, listed in the following and better described in the underlying flowchart:

- Phase 1 – Project of the wood structure on the basis of the configuration of the existing masonry building
- Phase 2 – Analysis of project loads
- Phase 3 – Calculation of the seismic action
- Phase 4 – Structural analysis
- Phase 5 – Check of the wood structure
- Phase 6 – Analysis and check of the wood-masonry wall connections
Civil masonry house in old town partially collapsed

**PHASE n. 1**
Project of the wood structure on the basis of the configuration of the existing masonry building

* Service weights
* Overloads
* Wood seismic mass

Calculation of seismic masses of the external masonry walls (2nd data elaboration)

**PHASE n. 2**
Analysis of project loads

Analysis and modelling by considering the sole seismic mass of wood

**PHASE n. 3**
Calculation of the seismic action of the non-collapsed external walls

Law seismic parameters

Structural factor $q=1.5$
Vibrations modes=$15$

**PHASE n. 4**
Structural analysis

Acceptable displacements in last limit state (SLU): $\delta X=1.32\,$ cm, $\delta Y=1.73\,$ cm

**PHASE n. 5**
Check of the wood structure considering a floor mass increase

**PHASE n. 6**
Analysis and check of the wood-masonry wall connections

Bagnoli A POST-SEISM REBUILDING PROPOSAL

Civil masonry house in old town partially collapsed

* Service weights
* Overloads
* Wood seismic mass

Calculation of seismic masses of the external masonry walls (2nd data elaboration)

**PHASE n. 2**
Analysis of project loads

Analysis and modelling by considering the sole seismic mass of wood

**PHASE n. 3**
Calculation of the seismic action of the non-collapsed external walls

Law seismic parameters

Structural factor $q=1.5$
Vibrations modes=$15$

**PHASE n. 4**
Structural analysis

Acceptable displacements in last limit state (SLU): $\delta X=1.32\,$ cm, $\delta Y=1.73\,$ cm

Bagnoli A POST-SEISM REBUILDING PROPOSAL
Project of the wood structure on the basis of the configuration of the existing masonry building - Phase 1

The first step consisted in designing a wood structure of the same type and with the same features of the pristine house hit by the earthquake. The municipality where this case study is located is Montereale, situated at 938 meters above sea level. In the pre-seismic state the house was composed by 2 floors with a bearing masonry structure and slabs in hollow-core concrete and hipped roof. In the post-seismic state dramatic damages were detected for the internal non-structural elements configuring the distribution among the internal spaces, a portion of the external masonry walls, as well as the roofing (Figure 1) The new inserted solution has a framed structure composed of beams and pillars, slabs and roofing, and it is entirely made of glulam wood.

Figure 1. Planimetry (ground – first floor) of pre and post seism
Analysis of project loads - Phase 2

On the basis of what written into the “Norme Tecniche per le Costruzioni 2008” (technical laws for buildings, N.T.C. 2008), the project loads were taken into account in the project, first considering the wood structure only, then also evaluating the increase in the seismic mass due to the existing external masonry walls. In Table 1 the different types of loads taken into account for the first data elaboration, relative to the sole wood structure, and used as input for the program are described.

<table>
<thead>
<tr>
<th>N_{id}</th>
<th>TL</th>
<th>Description of the load</th>
<th>Type of the load</th>
<th>Service Weight</th>
<th>Permanent NON Structural</th>
<th>Accidental Overload</th>
<th>Load of the snow</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Description</td>
<td>SW</td>
<td>Descrizione</td>
<td>PNS</td>
</tr>
<tr>
<td>1</td>
<td>S</td>
<td>Wood panel</td>
<td>Permanent load</td>
<td>Outer covering (4 cm)</td>
<td>500</td>
<td></td>
<td>Internal plaster (skimming), foam polyurethane insulating</td>
</tr>
<tr>
<td>2</td>
<td>S</td>
<td>Wood slab for the stairs ramp</td>
<td>Houses</td>
<td>Planking s=40 mm</td>
<td>300</td>
<td></td>
<td>Planking’s pavement s=40 m</td>
</tr>
<tr>
<td>3</td>
<td>S</td>
<td>Wood slab of roof-space</td>
<td>Houses</td>
<td>Planking s=40 mm</td>
<td>350</td>
<td></td>
<td>Parquet’s pavement s=15 mm</td>
</tr>
<tr>
<td>4</td>
<td>S</td>
<td>Wood roofing</td>
<td>Roofings</td>
<td>Secondary frame and wood planking</td>
<td>300</td>
<td></td>
<td>Roof coating and insulation</td>
</tr>
<tr>
<td>5</td>
<td>S</td>
<td>Wood slab</td>
<td>Houses</td>
<td>Planking s=40 mm</td>
<td>350</td>
<td></td>
<td>Parquet’s pavement s=15 mm</td>
</tr>
</tbody>
</table>

LEGEND:

N_{id} Identify number of loads analysis
TL Identify the type of load: [S] = Surface - [L] = Linear - [C] = Concentrated

Table 1. Loads analysis for the wood structure

Calculation of the seismic action - Phase 3

The main aspect of the case study was the quantification of the seismic mass transmitted from the external masonry walls to the internal wood structure in the case of further seismic events. The resulting seismic mass was considered for each floor in the second data elaboration and used as input data in the program (Phase 5).
The calculation was performed using the following characteristic data:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum ground acceleration ($a_g$) on type A subsoil for the considered limit state [m/s²]</td>
<td>0.260</td>
</tr>
<tr>
<td>Maximum value of the amplification factor of the horizontal acceleration spectrum ($F_0$)</td>
<td>2.36</td>
</tr>
<tr>
<td>Topographic category</td>
<td>T2</td>
</tr>
<tr>
<td>Subsoil category</td>
<td>B</td>
</tr>
<tr>
<td>Structure factor ($q_0$)</td>
<td>2.0</td>
</tr>
<tr>
<td>Fundamental vibration period ($T_1$) of the building in the considered direction [s]</td>
<td>0.204</td>
</tr>
<tr>
<td>Maximum acceleration (in units of g) sustained by the element $S_a$ (external wall)</td>
<td>0.10</td>
</tr>
</tbody>
</table>

**Table 2.** Seismic parameters used for the calculation of the seismic action

The increase of the seismic mass on the wood structure, due to the presence of external masonry walls is:

$$F_1 = 1.41/2 \text{ kN} = 0.705 \text{ kN at a height of 6.35 m}$$
$$F_2 = (1.41/2) + (1/2) \text{ kN} = 1.205 \text{ kN at a height of 3.25 m}$$
$$F = F_1 + F_2 = 0.705 + 1.205 \text{ kN} = 1.91 \text{ kN}$$

**Structural analysis - Phase 4**

The calculation of the seismic actions was performed in modal dynamic analysis, considering the behaviour of the structure under linear elastic regime, with a total number of vibrational modes equal to 15. In this phase the modelling of the wood structure was carried out, considering the additional masses arising from the external masonry panels.

The regularity of the structure must be checked both for the choice of the calculation method and the evaluation of the adopted structural factor. In the case study the structure resulted irregular both in plan and in height.

Concerning the definition of the response spectra, in addition to the ground acceleration $a_g$ (depending on the seismic classification of the municipality), the structural factor $q$ has to be determined. $q$ is a reduction factor of the elastic forces introduced to take into account the dissipative capacity of the structure, which depends on the adopted building system, ductility class and height regularity. Furthermore, the topographic amplification coefficient (ST) was assumed equal to 1.00. For the structure under consideration the following $q$ values were determined:

- Limit state of life safeguard (SLD)
  - $q$ for a horizontal seism along the X direction: 1.50
  - $q$ for a horizontal seism along the Y direction: 1.50
  - $q$ for a vertical seism: 1.50
Moreover, the building was designed for a nominal life equal to 50 years and for an use class equal to 2. On the basis of the performed geognostic investigations the ground was classified as B category. This corresponds to the following values for the parameters needed to build the horizontal and vertical response spectra:

<table>
<thead>
<tr>
<th>Limit state</th>
<th>$a_g$</th>
<th>$F_O$</th>
<th>$T_c^*$</th>
<th>$C_C$</th>
<th>$T_B$</th>
<th>$T_C$</th>
<th>$T_D$</th>
<th>$S_S$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SLO</td>
<td>0.0783</td>
<td>2.400</td>
<td>0.270</td>
<td>1.43</td>
<td>0.129</td>
<td>0.386</td>
<td>1.913</td>
<td>1.20</td>
</tr>
<tr>
<td>SLD</td>
<td>0.1035</td>
<td>2.330</td>
<td>0.280</td>
<td>1.42</td>
<td>0.132</td>
<td>0.397</td>
<td>2.014</td>
<td>1.20</td>
</tr>
<tr>
<td>SLV</td>
<td>0.2603</td>
<td>2.360</td>
<td>0.340</td>
<td>1.36</td>
<td>0.155</td>
<td>0.464</td>
<td>2.641</td>
<td>1.15</td>
</tr>
<tr>
<td>SLC</td>
<td>0.3338</td>
<td>2.400</td>
<td>0.360</td>
<td>1.35</td>
<td>0.162</td>
<td>0.486</td>
<td>2.935</td>
<td>1.08</td>
</tr>
</tbody>
</table>

Table 3. Seismic hazard parameters

Check of the wood structure - Phase 5

The calculations and checks were conducted by the semi-probabilistic limit state method, following the indications of Ministerial Decree published in January 2008 (Ministry of Infrastructure, 2008). The loads acting on the slabs, arising from the loads analysis, are automatically split up by the program over the frames (beams, pillars, walls, etc.). The loads, due to secondary vertical elements, on both foundation and floor beams are assumed as linear loads exclusively acting on beams. Furthermore, on all structural elements it is possible to apply further loads concentrated and/or distributed (changing with a linear law and acting throughout the beam or on limited segments of it). The actions introduced directly are combined with others (permanent, accidental and seismic loads) by means of load combinations, from which the probabilistic values used in the checks were obtained.

Moreover, the resistance checks were carried out with reference to both the frames and the wood/wood and wood/masonry connections. As it is possible to verify in Figure 2, struts made using steel rods with a 20 mm diameter had to be inserted as a consequence of the increase of the seismic mass for each floor.

At the end of the analysis and modelling (Figures 5, 6), very satisfactory values were obtained in terms of maximum displacement of the structure, which for the last limit state (SLU) resulted to be 1.32 and 1.73 cm along the X and Y directions, respectively.

We can conclude that the use of a small structural factor (1.5) and the insertion of steel struts allowed us to remain into an elastic regime, avoiding big displacements in a plastic regime and, at the same time, by stiffening the structure further breaks in the external masonry walls were prevented.
Figure 2. Modelling of the wood structure carried out by considering the increase of seismic mass due to the external masonry walls

**Analysis and check of the wood-masonry wall connections - Phase 6**

A fundamental aspect of the case study under consideration is the connection between the external masonry walls and the internal framed structure in glulam wood. As it is possible to see in Figure 3, 30x60h cm steel profiles were fixed at the 1st floor slab and roof, composed by a 30x30 cm corner element weld together to a 30 cm plate. This was designed considering the constraint (groove-groove) chosen in the calculation of the horizontal seismic action transmitted by the seism to the external masonry wall, and in turn, by the latter to the framed internal structure (Phase 3).

The connection to the walls is improved through the insertion, on holes previously made on the profiles, of threaded steel bars, prepared with an external plate at the wall to ensure a connection of the chain type. At the intrados, the profiles are fixed to the beams through screws, while at the extrados the flooring is fixed at the same beams still through screws. On the other hand, the connection in correspondence of wood pillars is performed using “C” 25x25h steel profiles, fixed by screws and unified with the profile at the extrados and intrados of the slab through welding. This solution, indeed, confers a larger strength to the whole system. Calculations demonstrated that, in the presence of an earthquake, the walls almost completely transfer the horizontal force to the internal wood structure, both in the direction of the smaller side (Y) and, although to a minor extent, in the longitudinal direction (X). To this aim, the frames strutted with 20-mm diameter steel rods confer a sufficient strength to ensure the stability of the building. In order to achieve this, however, it is important to assume a infinitely rigid slab. This is a reliable hypothesis since the slab is realized by joining three-layer panels with a total thickness of 7.5 cm, perpendicular to each other, with 2x6m dimensions and male-female coupled.
From these considerations, it arises that an action towards X direction follows the path described below:

- onto the three-layer panels forming the level, strongly connected to the joists and beams of the slab through 6-mm nails fixed at an inter-axes distance of 7.2 cm, thus also ensuring stiffness and ductility; therefore, the resulting connections are able to transfer the entire seismic action coming from either direction X or Y
- from the wood beams to the steel profiles placed along the wall perimeter at the first floor and roofing through steel screws with a 12-mm diameter and 450 mm length, having an inter-axes distance of 7 or 7.1 cm; even in this case the connection elements were sized so to transmit the entire seismic action
- the seismic action eventually reaches the ground through the whole framed structure, which, however, has already absorbed most of the seismic energy

It should be notice that the study of the connections was made in observance of the materials and the properties prescribed at paragraph 7.7.2 of “Norme tecniche per le Costruzioni 2008” (Ministry of Infrastructure, 2008).

On the basis of what so far stated, we hypothesize to use M12 heavy anchors passing through the wall and fixed by steel plates, placed as described in the following:

Along Y direction => 10 M12 two-row anchors 112 cm spaced
Along X direction => 16 M12 two-row anchors 122 cm spaced

**Figure 3.** Particular of the flanged connection
**Figure 4.** Model of the particular element of connection between the external masonry walls and the internal wood structure

**Figure 5.** Displacements of the framed wood structure calculated considering the seismic mass of the external masonry walls along X – last limit state (SLU)
Figure 6. Displacements of the framed wood structure calculated considering the seismic mass of the external masonry walls along Y – last limit state (SLU)

Conclusions

In the present paper a case study was described concerning the proposal of designing a wood structure to be inserted within non-collapsed external walls to be preserved because of environmental conservation orders or for maintaining the recognizability of the places. Since materials with very different mechanical and resistance behaviour, such as masonry and wood, coexist in the final structure, a crucial aspect to be evaluated in detail resulted to be the connections between the external and internal structures.

By applying a structural factor of 1.5 to the proposed structure connections with dissipative properties satisfying law requirements could not be guaranteed; therefore, it was decided to apply a structural factor of 1.

The necessary strength for the proposed solution could be achieved on one side by inserting St. Andrew cross struts in both transverse and longitudinal directions within the framed wood structure, and on the other side by devising a suitable connection between the external masonry walls and the internal wood framed structure. This connection was realized through steel anchors, fixed to the external part of the wall by steel plates. Anchors are formed by corner steel elements weld together to “C” steel profiles, suitably fixed to the walls, to the intrados and extrados of the slab through welding. This connection aims at unifying the external masonry box to the internal wood structure mimicking a perfect interlock. By means of such devices, very good results were obtained from the calculation of the horizontal seismic forces.

At the end of the design and after the analysis of the data, it is possible to conclude that the two-floor house taken into account remains in a linear elastic regime, although it was located in an area
with very high ground acceleration, such as that of Montereale. After a strong earthquake, the structure does not suffer damages to both components and structure itself, and it also does not lose stiffness with respect to the horizontal actions. In conclusions, the performances obtained by the structure here proposed exceed the limits required by the law. Nevertheless, an aspect to be further developed consists of the detailed study of the local connections, with particular regard to the crucial junctions responsible for the transmission of the main forces.
General References


Fusero Paolo (2010), Un primo bilancio sulla ricostruzione in Abruzzo, “Urbanistica Informazioni”, V. 230, pp. 23-26

http://www.gazzettaufficiale.it/eli/id/2008/02/04/08A00368/sg