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SEISMIC BEHAVIOUR OF WOOD FRAMED BUILDINGS IN CADORE MOUNTAIN REGION - ITALY

Ario CECCOTTI[1], Paolo FACCIO[2], Monica NART[1] and Paolo SIMEONE[2]

SUMMARY

Three most common different types of wooden-frame are therefore considered for the purpose of the present study (mortar-wooden branches, bricks, and stone infilling).
Interpreting the structural behaviour of the buildings in question, and thereby understanding their mechanical behaviour under seismic actions is the objective of this paper.
The methods used to achieve this aim included breaking the building down into its different elementary parts in order to examine its horizontal and vertical load-bearing elements.
Then a set of quasi-static cyclic tests on elementary walls were carried out in order to achieve the hysteretic behaviour of the three different possible wall arrangements.
Finally, a non-linear analytical model in the time domain has been used to predict shear walls structural behaviour under different seismic excitations likely to be anticipated for the Dolomites region.
A results discussion, in particular about the possible strength-stiffness-ductility optimal combinations, concludes the work.

INTRODUCTION

Rural architecture in the Dolomites is constantly characterised by a feature that recurs irrespective of certain area-specific aspects and historical differences, i.e. the combined use of wood and stone, which were the only building materials available. The building typology can vary, however, and reflects the diversity of customs of the inhabitants of the various regions, or certain historical and political events that distinguished the different mountain areas.
The geographical position of the settlements, the people's different activities and the ready availability of wood rather than stone are nonetheless the main factors that influenced the "form" of architecture in this part of Italy.
Today, the landscape is still dotted with a very ancient type of Alpine rural settlement. Every ridge coincides with a system of small villages that go by the name of "viles", each comprising three or four elementary cells called "masi". A "maso" is a set of buildings serving specific functions, whose vertical load-bearing structures are characterised by the construction methods typically used in the area to build

[1] Trees and Timber Institute, National Research Council, Italy
[2] Dipartimento di Costruzione dell'Architettura, IUAV university of Venice, Italy

wooden-frames. The particular nature of the choices made in what can justifiably be defined as a building method markedly influences the seismic behaviour of these constructions, in terms of both their strength and their stiffness.

Two different types of wooden-frame are encountered and both are considered in the sample building chosen for the purpose of the present study.

Interpreting the structural behaviour of the building in question, and thereby understanding its mechanical behaviour, can be defined as “structural reading”.

The methods used to achieve this aim included breaking the building down into its different elementary parts in order to examine its horizontal and vertical load-bearing elements.

In the geometrical analysis, an alphanumerical code was used for each element forming this structural unit so as to facilitate the identification of the connections between them and thus define the degree of anchorage and constraint, and consequently interpret the mechanical behaviour of the building as a whole.

DEFINITION OF THE STRUCTURAL CHARACTERISTICS OF THE BUILDING TYPES

Building methods

The original building has an irregular layout organised on three levels. The ground floor is made of masonry, comprising partially-plastered, fairly regular blocks of stone, while the load-bearing structures for the first and second floors are made of timber. To be more precise, the wooden-frame can be distinguished according to two different categories, hereinafter generally referred to as “type 1” and “type 2” panels.

The first are made using the so-called “gardiz” technique, which derives from the earliest methods used by man to build the walls of huts, characterised by a web of poles driven into the ground with horizontally-placed branches forming a lattice, that was spread with a layer of mud that dried in the sun.

The building is composed of a main set of horizontal and vertical members (beams and wooden pillars) connected together with dovetail joints, and filling made of lime mortar and roughly-crushed stone “reinforced” with a trellis of hazelnut branches (sideshoots) subsequently covered with a layer of plaster to achieve a final thickness of 20 cm.

The dovetail joints are made without the aid of any additional wooden wedges or metal pins, nails, brackets or bolts.

The beams are rectangular, with an average cross-section of approximately 16 x 19 cm and are roughly 5 m long. These beams form the upper and lower floors, supported by three wooden pillars of much the same cross-section that are 140-150 cm high, positioned at the midline and at each end of the beam.

In addition to the three main wooden pillars, at irregular (approximately 20 cm) intervals there are further secondary vertical elements with an average cross-section of 7x7 cm, connected to the beams by means of approximately 1-2 cm mortices in which the ends of these vertical elements are forcefully inserted. These secondary elements are interwoven with hazelnut sideshoots that form a trellis, exploiting the flexibility of these branches, that can be bent without breaking.

In some cases, the joints in the “type 1” panels are more straightforward than the dovetailed type, but are completed with soft iron nails. In other cases, the same panels are stiffened with diagonal bracing nailed onto the outside, probably added after the wooden-frame was made, to cope with instability phenomena developing due to the connections in the main members progressively becoming less effective.

Generally speaking, the presence of nails, brackets or (occasionally) even hardwood studs is characteristic of the connections made using lapped joints and/or simply-resting beams, so as to stiffen the structural node and keep the mutual position of the elements steady.

A further variant is observable in the structure of the filling, where the lattice of branches is sometimes replaced by vertical planks (called “scorzoni”) to which perpendicular strips of wood of varying size (average section 4x2.5 cm) are nailed approximately 3-5 cm apart.

In some cases, there may even be two different types of filling in the same frame, i.e. the above-described "gardiz" variant and a second filling made with wooden planks and straw.
The straw occupies the gaps between the vertical planks held in place by the horizontal strips that have a cross-section of approx. 2x2 cm nailed to the upper and lower beams.
In this case, the load-bearing framed members are composed of beams and wooden pillars with an approximate average cross section that ranges from 10x12 cm to 16x18 cm, and may be up to 6 m long. The connections are made mainly by means of lapped joints and the panels are stiffened with diagonal bracing elements.
The "type 2" panel differs essentially from "type 1" in that it includes diagonal elements, lying this time on the same plane as the principal framed members with an filling that has no lattice support.
The load-bearing structure comprises of square beams with a cross-section of approximately 16x16 cm, with nailed lapped joints.
The diagonal planks have a cross-section of approximately 7x16 cm and they have no connections, being simply rested against the inside of the beams.
The filling is made of crushed stone (with a very large particle size) held together with a mortar made of lime, earth and sand.

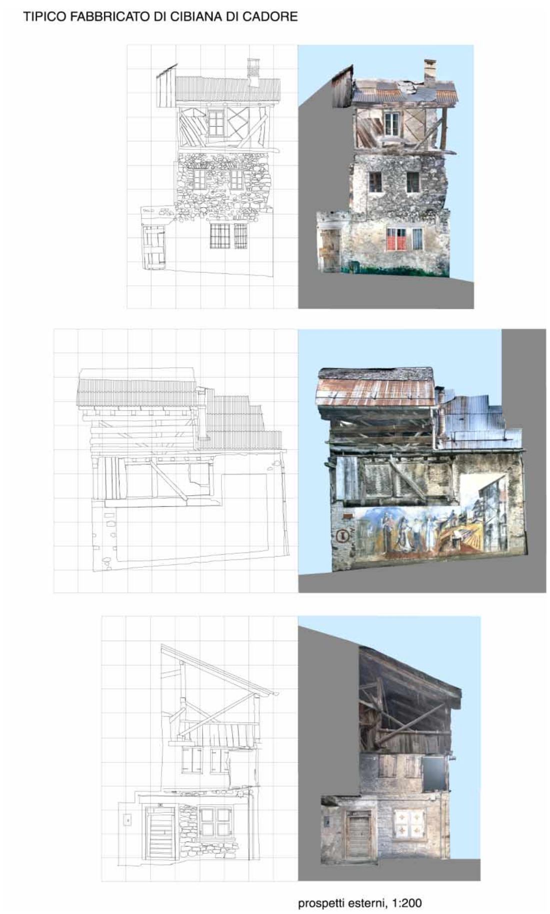


Figure 1: building of Cibiana di Cadore

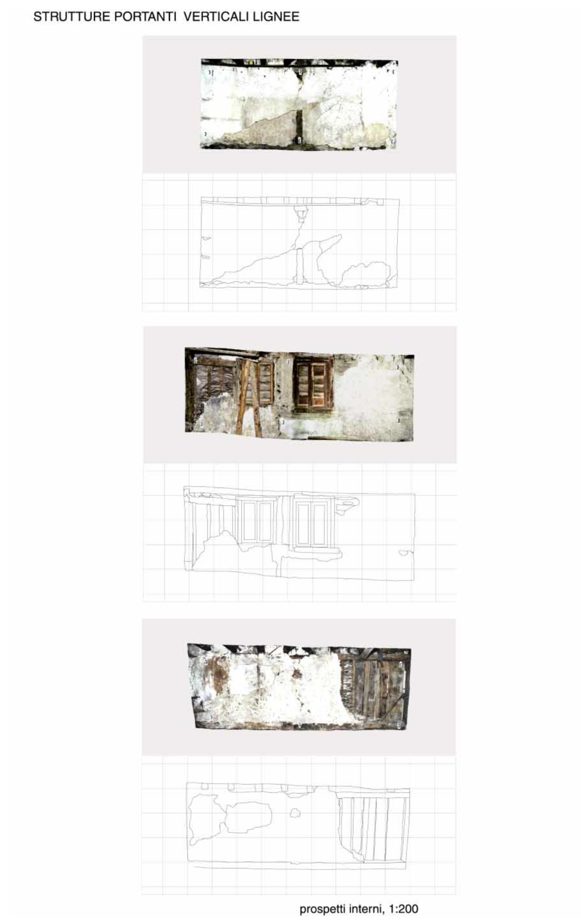


Figure 2: wooden-frames

Mechanical characterisation of the elements

The representative nature of the panels used in the tests, and hence the feasibility of transferring the data to theoretical models, depends on the characterisation of at least three essential factors that influence the building's mechanical behaviour, i.e. the connections, the strength of the timber and the type of filling.

The test results can therefore be referred to the original wooden-frames providing the necessary corrections are made.
Concerning the connections, their static behaviour can be considered as a function of the quality of their execution, the fundamental features of the timber and the degree of deterioration. Moreover, the first two of these elements introduce a fundamental aspect in terms of the building's seismic response, i.e. the ductility of the node acts as a safety valve for preserving the members and, more in general, the building as a whole because it enables part of the energy coming to bear on the node to be dissipated and maintains the connection, albeit at more limited levels.
The state of deterioration, on the other hand, like the inevitable presence of cracking and strain, is particularly indicative of the degree of uncertainty in any attempt to determine the real behaviour of the joints analysed on the sample building panels - a behaviour that is difficult to reproduce when the models being tested have newly-made connections in theoretically healthy timber. The calibration of the test results must consequently allow for the principal effects of any deterioration of the nodes, i.e. progressive widening of the dovetailed joint with deformation of the tenon and mortice, separation of the surfaces that were initially in contact and concentration of the stresses in increasingly limited areas.
On the other hand, the functioning of the joints, like the structural elements they connect, is strongly influenced by the rheological-mechanical features of the timber, which depends on the nature of the material itself and is influenced by three particular aspects:
— the anisotropy due to the structuring of the timber, whose elementary units are not iso- diametric, but elongated and lying with a certain orientation in concentric layers with a different degree of compaction;
— the frequency with which structural deviations and other defects occur in the tissues at the node, which is difficult to translate in simulations and which gives rise to an actual reduction in the strength of the structural element's section, with a genuine change in its geometrical characteristics;
— the strong influence that variations in humidity and temperature can have both on the strains, depending strictly on the elastic moduli, and on the ultimate loads, i.e. on the member's strength.
In addition to the above-described circumstances, another two factors that depend exclusively on the test methods can also have a considerable influence on the outcome of the tests, i.e.
— the rate of application of the load and the duration of loading;
— the loading frequency and alternation, if any.
In the light of the above considerations, the definition of the mechanical characteristics of the material used to make the wooden-frames was based on average values referring to coniferous wood with a humidity between 10% and 15% submitted to short-lived loading tests in which defect-free members reach failure in about 5 minutes:

Mechanical characteristics	Ultimate strength	Elastic modulus
traction parallel to fibers	10-40MPa	E= 10000-15000MPa
traction perpendicular to fibers	0.5-0.6MPa	E= 300-500MPa
compression parallel to fibers	25-40MPa	E= 10000-15000MPa
compression perpendicular to fibers	8-10MPa	E= 300MPa
shear	2-4MPa	G= 500MPa
simple bending	60-70MPa	E= 10000-15000MPa

Table 1: mechanical characteristics of material used

Finally, though it is not a structural element, the filling nonetheless plays a fundamental part in the global behaviour of the building. In the static arrangement of the wooden-frame, it is plausible for the filling to affect structural behaviour due to its position, stiffness and strength. Its mechanical characterisation proves particularly difficult, however, with a view to defining a theoretical model for at least two reasons, i.e. because its strength and stiffness characteristics cannot be reliably determined and because it is impossible to schematically reproduce the filling and the conditions of constraint between said filling and the other structural elements with a sufficient degree of realism.

Efficacy of historical technologies: strength and stiffness in relation to horizontal actions

The stiffness and strength of the two types of wooden-frame in withstanding seismic action, which can be synthetically represented as a horizontal force, depends partly on the capacity of the filling to brace the structure, but above all on the mechanical and technological features of its internal connections and their ability to assure a plastic, energy-dissipating behaviour.

Quantifying the lateral stiffness offered by the filling in terms of shear strength is somewhat complicated, however, because the values being sought depend largely on three fundamental characteristics:

— the frequency, regularity and capacity for containment of the building's internal connections, which enable elastic actions to be withstood and simultaneously ensure a ductile behaviour;

— the repetition of vertical elements approximately every 60-80 cm in the vertical structures;

— the presence of filling in the load-bearing wooden-frame.

Under the effect of an earthquake, it is thanks to the simultaneous presence of all three said characteristics in this type of building that the parts of filling can collapse while the wooden-frame remains standing: severe strain in the weaker parts of the building (and even their collapse) is not transmitted to the load-bearing structure.

A precise construction principle is thus stated: the elements forming the structure of the building (wooden-frame + filling) can become independently deformed while simultaneously co-operating with one another.

Eurocode 8 offers a plausible explanation for the diaphragmatic behaviour of such filling walls under seismic actions. In fact, it would be to extrapolate certain considerations that can also apply, albeit with all due approximation, to the historical buildings considered here.

EC8, part 1.3 "Specific rules for wooden buildings" states that filling panels exhibit an excellent antiseismic behaviour in that they dissipate the energy better than bracing, since the latter concentrates the energy in a small area where the frame is in contact with the bracing.

In this sense, it is worth emphasising that the construction characteristics of the "type 1" panels, which account for the majority of the buildings in Cibiana di Cadore, almost never include bracing systems such as struts or cross-stays. It should also be said that the "type 2" panels contain diagonal elements, but these are never connected in any way to the load-bearing frame.

This entitles us to assume that the filling in question has always assured a valid performance (subject to the inevitable effects of deterioration) in terms of resistance to horizontal actions.

The maximum efficacy of the entire system, however, should depend on the degree of anchorage (or interaction) between the wooden-frame and the filling panel, not just on the stiffness of the latter. Clearly, this is not easy to calculate in the original buildings, however.

We might nonetheless consider assessing the strength and lateral deformability of the frames, synthesising the effect of the fill by means of a diagonal strut, thus returning to the general layout of the mechanical behaviour of a masonry panel under shear actions. Using this schematic representation, moreover, the fundamental role of the frame's vertical elements in terms of earthquake resistance would emerge because the variation in their distance between centres would modify the slant of the strut: the closer this comes to the vertical, the lower the horizontal component of the thrust.

In the event of earthquake, the capacity to dissipate energy through the joints depends on whether the entire building can work as a hyperstatic framework with structural nodes where the timber inevitably comes under stress in a direction parallel and perpendicular to the fibers.

The connections in the original panels are characterised, in fact, by the direct wood/wood contact that comes under localised compression and is never, or virtually never a perfect fit.

In the case of seismic stresses, this aspect leads to the likelihood of displacements occurring in the joint without triggering unacceptable states of stress in the various timber structural elements and guarantees a strong energy dissipation due to a localised compression perpendicular to the fiber (in line with the structure's nodes) and a fair amount of energy dissipation due to friction between the various structural elements converging on the node. The properties of the connections thus enable a degree of ductility and energy dissipation to be obtained at joint level that it would be impossible to achieve from the basic material itself. In fact, the connections "can withstand very important strains before the wooden material reaches a genuine failure due to localised fracture".

EXPERIMENTATION

Geometrical and structural characteristics of the models

The panels used for testing were designed so as to reproduce the materials, dimensions and construction methods of the original frames as far as possible. That is why certain electrical tools and working methods were used in their construction to avoid altering the construction features and configuration of the historical building.
This was obviously fundamental to an analysis of the mechanical behaviour of these historical buildings, and not just of the traditional building methods involved, because – as mentioned earlier – what particularly influences the global behaviour of the building is the accuracy with which the connections are made.
The use of conventional woodworking tools, e.g. axes, various types of saw, chisels, gouges, etc., rather than the "modern" alternatives such as electric seamers and hacksawing machines, is a factor that can also characterise certain aspects of the mechanical behaviour of these elements.
The study frequently revealed the differences in the quality of craftsmanship of the original structural parts: none of the joints are the same size and shape, so the functioning, or efficacy, of the connection varies with the differences in the quality of the joints. The only electrical tool adopted was a hacksawing machine (with a reciprocating blade movement) that works rather like a normal handsaw. Making the joints with this tool assured a cutting precision of around 4-7 mm.
The end result was an imprecise contact between the surfaces of the connections that made it necessary to fill the gaps with spacers to make the connection as effective as the one assumed in the original frames.
The following materials were used to make the models:

— for the timber load-bearing structure: Norway spruce from Passo Falzarego;
— for the filling: Dolomitic limestone from Cevedale del Friuli;
— hazelnut tree sideshoots (branches) from Centignano (Viterbo);
— standard bricks;
— slaked lime, hydraulic lime and sand (for the mix design, see below).

Six models of wooden panel were prepared, 4 representing the "type 1" panels and 2 for the "type 2" panels. The following table shows the main features of the 6 panels, identified by means of codes. Panels M3 and M6 are 2 models that were modified with respect to the original features of the connections to represent two possible solutions for improving the mechanical behaviour of the joint.

PANEL	CODE	FILLING	CONNECTIONS
Type 1	M1 M2 M3 M6	Branches+plaster Branches+plaster Branches+plaster Branches+plaster	Dovetail joints Dovetail joints Dovetail joints stiffened with screws Joints with EPDM shock-absorber pads and screws
Type 2	M4 M5	Stones+plaster Bricks+plaster	Panel with lapped joints and screws Panel with lapped joints and screws

Table 2: wooden-frame

All the models were fixed to the floor using metal "squares" attached both to the panel's horizontal bottom beam, using a threaded steel bar 12 mm in diameter passing right through the beam, and to the floor using a pair of anchor bolts in the concrete slab of the floor. Each model has 6 "squares" on each side, for a total of 12, in order to prevent the panel from shifting in the direction of the load and avoid any oscillation with movements perpendicular to its principal plane.

Geometrical and structural characteristics of the "type 1" models

Models M1 and M2:

— the load-bearing structure is a wooden-frame comprising a main set of five beams, 2 horizontal and 3 vertical, with a 16 x 19 cm cross-section, held together with dovetail joints by 8 smaller vertical beams with a cross-section of 7 x 7 cm inserted directly in the two horizontal beams.

In the M1 panel, the dimensions of the joints are identical to the originals measured in situ, while the joints in panels M2, M3 and M6 are larger; the reason for these differences lies in the fact that the six dovetail joints of the second "type 1" model were during its construction, when the panel was shifted upright from the horizontal position in which it was prepared. This prompted us to increase the size of the projection on the joint in order to avoid any further damage during the assembly stages. As a result, only the joints in panel M1 have the dimensions of the original.

It should be noted that the sizing adjustment made to avoid the problem of joint failure has absolutely no effect on the mechanical behaviour of the joint, it merely increases its performance by comparison with the original because of its greater contact surface area.

— the filling is composed of a lattice made of hazelnut sideshoots (branches) arranged horizontally and woven between the vertical beams to provide a genuine reinforcement that guarantees the static arrangement of the filling. On top of the lattice, we spread a layer of mortar made of slaked lime ($Ca(OH)2$ + $nH20$), hydraulic lime and sand, with a mix design chosen to facilitate its workability and setting features even in the presence of water, and to avoid (by adding sand) any further porosity and cracking due to shrinkage during the carbonation of the lime.

Though the in situ study revealed the presence of various aggregates in the plaster, including earth, gravel and pieces of opus signinum, the features of the two compounds were perfectly comparable; in particular, the use of opus signinum was certainly related to the need to facilitate the hydraulic reactions obtained using hydraulic lime.

Models M3 and M6:

— the load-bearing structure was the same as for the models M1 and M2; the only difference concerned the technical characteristics of the dovetail joint, for which measures to improve its behaviour in seismic areas were envisaged. In fact, the connections in model M3 were stiffened by inserting special self-tapping wood screws (HecoTopix), which have the following essential features:

1. they can be screwed down without preparing a hole;
2. they ensure easy penetration (the body of the screw, between the head and the end of the thread, is specially shaped with sharp grooves serving a dual function: when it is screwed down, it has a self-tapering effect to facilitate the penetration of the head; when it is withdrawn, it contributes towards increasing torque and tensile strength);
3. they induce an extremely limited splitting of the wood fibers when they are inserted.

When the panel is loaded, the contribution of this system has the effect of stiffening the mortice and tenon joint, thus increasing the strength of the wooden-frame in its elastic phase.

The screws were inserted in a crosswise layout to achieve the maximum stiffening effect.

Their dimensions were: screw 8.0 mm, head 14.8 mm, screw length 260 mm, thread length 120mm.

The connections in model M6, on the other hand, featured the inclusion of a shock-absorbing EPDM (ethylene-propylene) pad and the same screws as in model M3.

A pad 23 mm thick was glued to the wood in the recess of the dovetail joint, using a special mastic.

A "soft" material was thus inserted between the two main contact surfaces of the projection and recess to offer the tenon a certain degree of freedom of movement (rotation) without changing its position in the dovetail joint and without giving rise to any severe splitting of the wood.

The pad material was an copolymer elastomeric rubber comprising two monomers, ethylene (E) and propylene (P), with a low percentage of double bonds, that also contains a certain amount of diene (D) through non-saturation and methylene CH2 (M), which forms the "backbone" of the polymer.
It is feasible to assume that this system lends the building's joints a greater deformability prior to failure.
— the filling materials were the same as for models M1 and M2.
It should be borne in mind, moreover, that model M6 was 15 cm shorter than the other panels. Such a difference has a bearing on the global behaviour of the building submitted to a horizontal force. In this sense, the EC5 (Eurocode 5) provides a mathematical formula for restoring the findings obtained with a panel whose height differs from the height of the other panels in a test series (h ? h_{test}) "if the strength of a test panel has been determined, the strength of a panel of similar construction but different h is given by $f_k = k_{h\ \ ftest,k}$; $k_h = (k_{test} / h)^2$, where $h_{test} > h$".

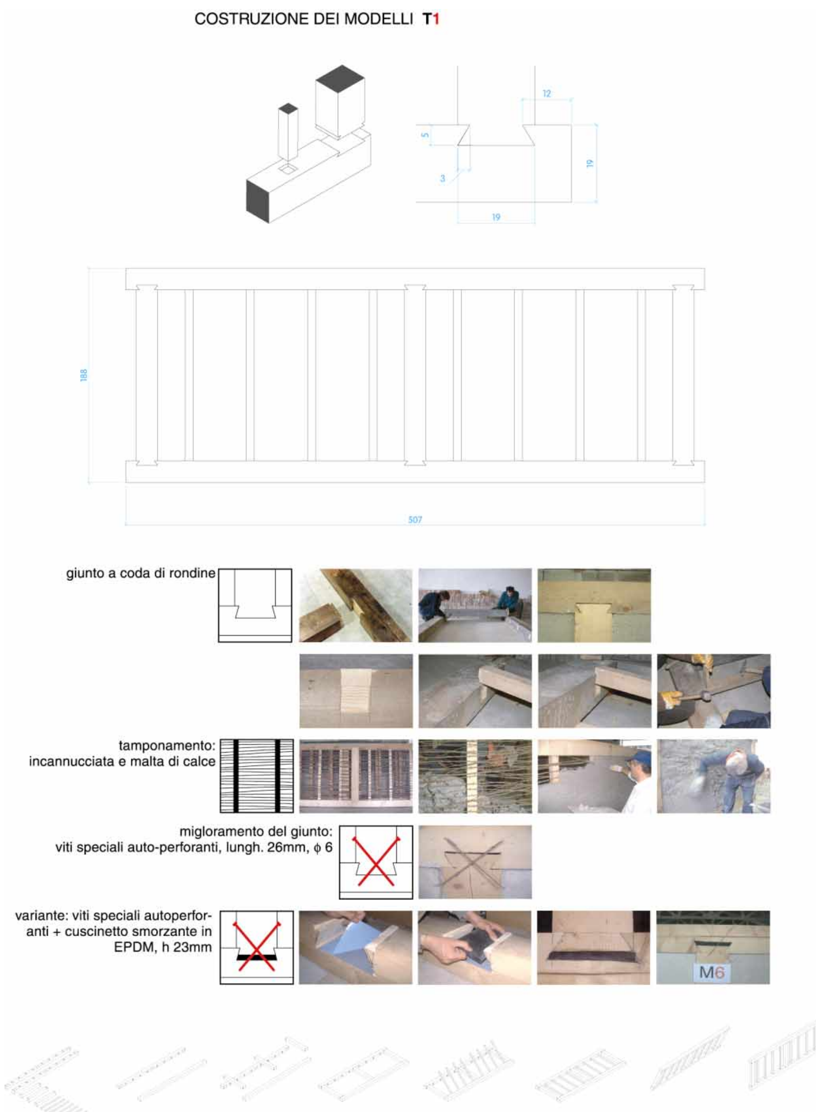


Figure 3: type 1 models

Geometrical and structural characteristics of the "type 2" models
Models M4 and M5:

— the timber load-bearing structure is composed of a main framework of 7 beams, 2 horizontal and 5 vertical, with a cross-section of 15 x 15 cm, connected by means of wooden joints screwed together to form 4 “quadrants” filled with masonry.
The wooden joints are held by a pair of normal wood screws in lieu of the traditional nails of square cross section made in the Zoldano-Cadorino area, found in the original frames and used to connect two or more timber members.
— the filling panels are composed of two different types of masonry.
In M4, the masonry is made of crushed Dolomitic limestone with a mortar whose mix design is similar to the one used for the “type 1” models. This type of masonry exactly reproduces the original found in situ. The stones were crushed with a sledgehammer to obtain a certain degree of regularity in the overlapping layers of stones bedded with mortar.
Each “quadrant” of masonry has three elements 3 cm high placed diagonally to form crosses.
Model M5 differs from the above only in that the stones are replaced with bricks: this means that the model is no longer a faithful representation of the original construction features (since no brickwork masonry was seen in the area), but it does not alter the structural general configuration of the panels, which comprise a timber load-bearing structure and a masonry load-bearing structure.

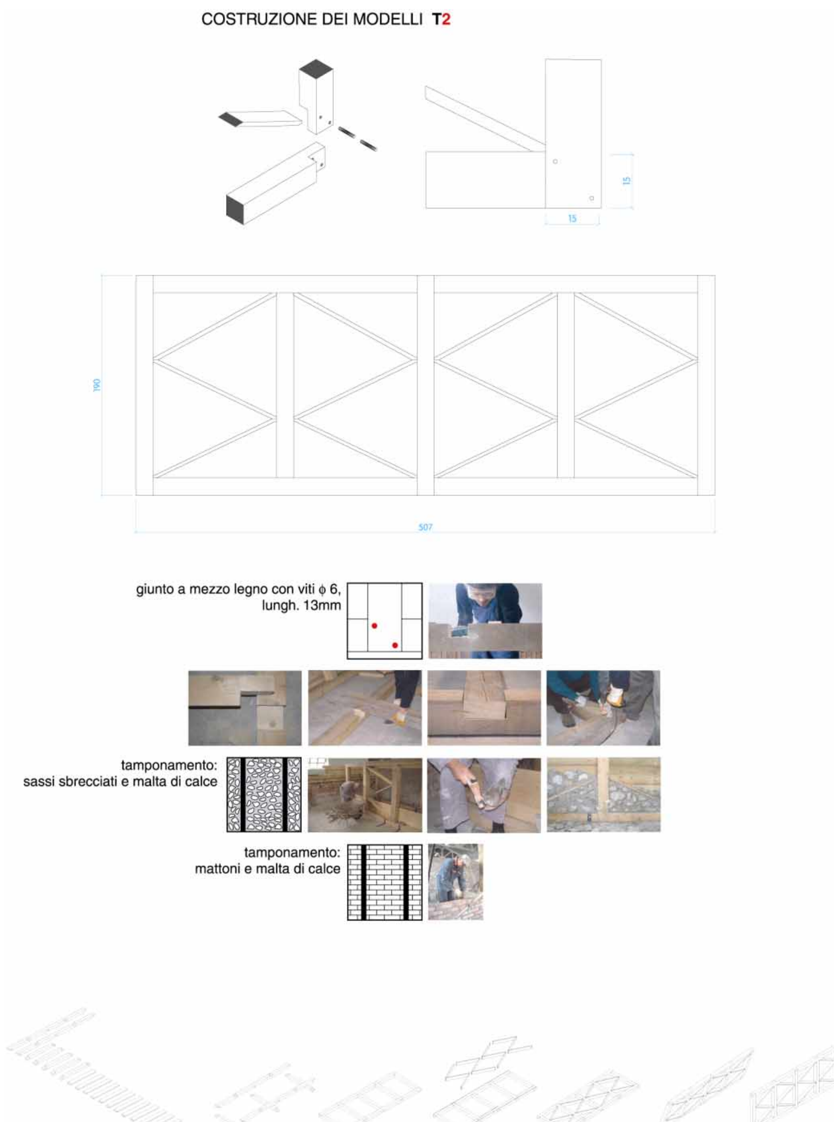


Figure 4: type 2 models

Test methods

The tests on the real-scale models involved a cyclic loading test, adding to the load in steps of around 10 kN up to failure. The load was brought to bear by means of two pairs of hydraulic pistons capable of developing a maximum force of 120kN, fixed with chains and metal angle bars to the four corners of the panel.

Two strain gauges with a sensitivity to 1/10th of a millimetre were attached to the wooden pillars 175 cm above floor level to record the horizontal displacements.

The data provided by the pistons' loading cells and the two strain gauges were processed in real time by a computer.

Two aspects strongly characterise the test and consequently also the results:

1. the action of an earthquake on a building is characterised not only by the intensity of the seismic force, but also by a given frequency: the test does not consider the rate at which the load was changed, but the instants, in hours, minutes and seconds, when the load was increased (or reduced);

2. during an earthquake, the cycles are less regular than those produced during the test because the inputs are - by their very nature - random and irregular, so the number of complete cycles up to the maximum loads (or displacements) is generally very low, while the smaller cycles are more numerous.

The vertical component of the diagonal force imposed by the pistons can be considered as a permanent load coming to bear on the structure of the panels. If they were loaded only horizontally, there would be a risk of the panel failing due to the top beam being lifted away from its wooden pillars.

Fundamental aspects to consider in the analysis and interpretation of the results obtained are the percentage of relative humidity in the timber in the panels and the air humidity and temperature inside the laboratory:

— surface humidity of the beams = 14%
— internal humidity towards the core of the beams = 18%
— air humidity = 69%
— air temperature = 22°C

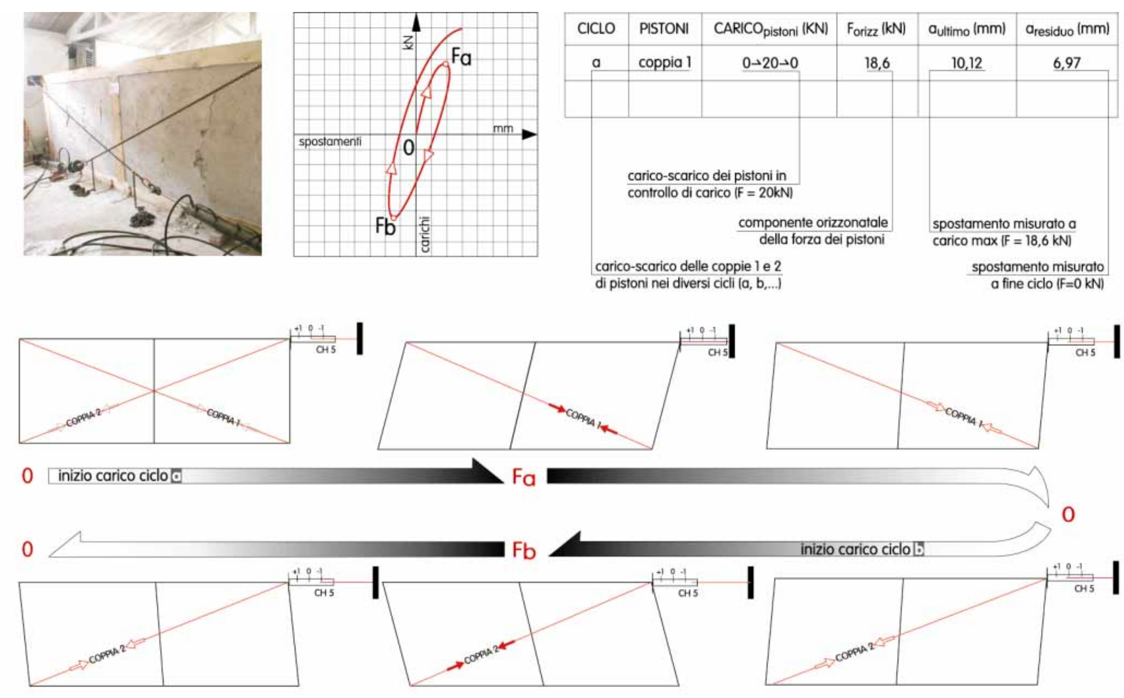


Figure 5: test method

ANALYSIS AND INTERPRETATION OF THE RESULTS

Generally speaking, the tests on the models produced clear experimental evidence. The results confirmed the predictions concerning the behaviour of the models under a horizontal action.
The two variants of the dovetail joint in the “type 1” panels M3 and M6 were conceived with a view to improving the mechanical behaviour of the joint in two different ways:
1. it was assumed that inserting screws in the dovetail joint (M3) would increase its stiffness and resistance to cyclic loading vis-à-vis the unimproved model (M2);
2. with the further addition of a shock-absorber pad made of EPDM (a perfectly elastic material) inside the joint (M6), as well as the screws, the plan was to lend the structure greater ductility in relation to ultimate loads, without reducing the strength provided by the screws.
Both improvements were successful, confirming the initial predictions:
— by comparison with M2, model M3 demonstrated greater strength but no change in ductility (Fmax for M3 > Fmax for M2);
— by comparison with M3, model M6 showed an increase in ductility with a strength at collapse that remained unchanged (Fmax for M3 = Fmax for M6).
By comparison with the “type 1” panels, the “type 2” revealed much greater strengths at failure, influenced by the technological characteristics of the internal connections. The wooden joint offered an improvement in terms of both strength and ductility. Moreover, when the wooden-frame reached critical conditions, the filling made with stones (M4) and bricks (M5) demonstrated a greater bracing behaviour can was assured by the trellis work and mortar.
It was the way in which the two different “type 2” panel fillings functioned that contradicted, at least in part, the baseline predictions, in that failure was expected to cause the expulsion of the filling in both cases – but the bricks proved stiffer than the stones. Being confined by the diagonal panels and by the main members of their wooden-frame, the brickwork masonry parts did not collapse at failure, they simply became cracked. Conversely, failure loads determined the collapse of the stones in model M4. This difference in behaviour can be explained by the typically random arrangement of the stones in the triangular portion of the quadrants. This allowed for movement between the elements during the displacement of the structure under stress, with a consequently more ductile behaviour of the stone wall than of the brick wall.
Considering that stones (Dolomitic limestone) are stronger than bricks, this result is surprising.

	F_{max} (kN)	a (mm)	F_{pc} (kN)	a_{pc} (mm)	F_0 (kN)	a'_{pc} (mm)
M2	20,46	25,81	16,74	31,2	0	20,37
M3	35,7	28,4	27,9	32	0	23,91
M6	35,52	38,57	27,71	42,47	0	23,31
M4	69,56	78,86	55,8	81,4	0	66
M5	98,4	65,6	93	67,7	0	54,9

TRATTO	F_{max} (kN)	a (mm)	F_{pc} (kN)	a_{pc} (mm)	F_0 (kN)	a'_{pc} (mm)
I	raggiungimento del collasso: a decrementi di carico la struttura continua a deformarsi		-	-	-	-
II	-	-	fase post-carico critico: la struttura viene caricata fino a Fpc; lo spostamento a dipende dall'intensità di Fpc decisa dallo sperimentatore		-	-
III	-	-	-	-	fase di scarico e fine della prova: la struttura rimane deformata con uno spostamento residuo a'	

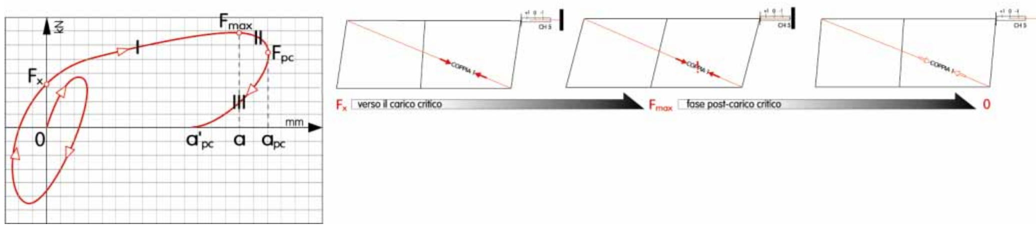
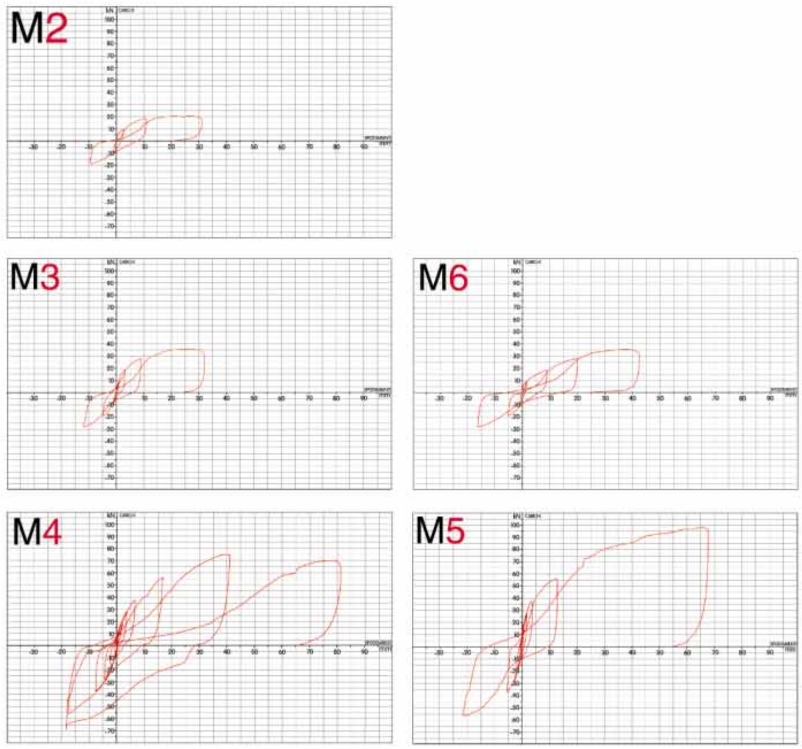
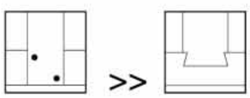


Figure 6: results

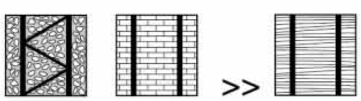
ANALISI ED INTERPRETAZIONE DEI RISULTATI



T1 *vs* T2

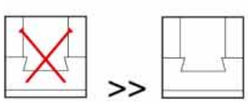


I telai "tipo 2" hanno dato valori di carico di collasso F_{max} molto maggiori di quelli del "tipo 1"; questo dipende dalle caratteristiche tecnologiche delle connessioni interne: il giunto a mezzo legno offre sia una resistenza maggiore sia un comportamento più duttile una volta raggiunto il collasso.



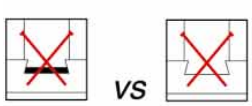
I muri di riempimento in mattoni e sassi, una volta entrata in crisi la struttura lignea, hanno offerto un comportamento di controventamento superiore rispetto ai muri di incannucciata e malta

M2 *vs* M3



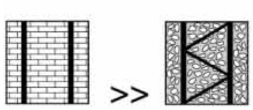
L'inserimento di una coppia di viti nel giunto a coda di rondine ha aumentato la rigidezza delle connessioni interne della struttura lignea; fino a piccoli spostamenti questo miglioramento ha dato al pannello una maggiore resistenza Fmax mentre la duttilità è rimasta invariata

M3 *vs* M6



L'inserimento di un cuscinetto smorzante in etilene-propilene (EPDM), perfettamente elastico, ha modificato la capacità di deformarsi del pannello: a parità di carico M6 ha dato degli spostamenti maggiori rispetto ad M3. La resistenza complessiva della struttura è rimasta invariata, mentre in M6 è aumentata la duttilità

M4 *vs* M5



Il muro di mattoni ha offerto una maggiore rigidezza rispetto a quello in sassi: i mattoni, confinati dalle tavole diagonali e dalle membrature principali del telaio ligneo, non sono caduti in fase di collasso ma si sono solo fessurati; al contrario, nel muro di sassi e malta è avvenuta la caduta del materiale sotto i carichi di collasso.

Figure 7: analysis and interpretation of the results

CONCLUSIONS

The outcome of the tests brought to light certain behavioural features of the buildings considered under horizontal actions that were entirely unexpected. The strength of the models prior to failure was quite remarkable.

It could therefore be said that the construction methods used in the panels studied here possess characteristics and qualities that are genuinely effective in terms of earthquake resistance.

The analysis and interpretation of the results offers a useful contribution to our understanding of the behaviour of wooden-frame structures and buildings in the event of seismic action on the basis of mathematical modelling.

A comparison between the test results and the direct reading of existing buildings revealed precious information with a view to the restoration and rehabilitation of these rural buildings without altering their original structural design. It also enables their seismic safety to be influenced by means of a reappropriation of the original construction methods integrated with measures for their improvement and structural adaptation.

Nonetheless, these findings remain circumscribed to the study of single structural elements belonging to a more complex organism, which will hopefully be the object of further studies on its global behaviour in reaction to earthquake.

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