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SEISMIC BEHAVIOUR OF HISTORIC TIMBER-FRAME BUILDINGS IN THE ITALIAN DOLOMITES

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SUMMARY

The historical building heritage in the Italian Dolomites comprehends traditional timber-frame constructions. This construction technique is the topic of this paper. Analysing the structure of the buildings in question and hence understanding their seismic performance is the main objective.

The methods used to achieve this aim included breaking down the building into its different elementary parts in order to examine its horizontal and vertical load-bearing elements. Two most common types of wooden frames are considered for the purpose of the present study (with three different infill configurations: wooden branches and mortar, bricks and stones).

The experimental part comprised quasi-static cyclic tests on wall specimens carried out to evaluate the lateral load resistance, the ductility and the capacity of energy dissipation of the three panel configurations. These results were then used for the theoretical part aiming to establish a numerical model which includes a pinching hysteresis model and is fitted to the cyclic test data. The numerical analyses were performed for several historical earthquakes. For each earthquake, the Peak Ground Acceleration (PGA_u) producing the "near collapse" ultimate state of this building type has been determined.

Having these results, a realistic statement can be given regarding the seismic performance of these traditional timber-frame buildings.

INTRODUCTION

Rural architecture in the Dolomites is constantly characterised by 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.

Today, the landscape is still dotted with a very ancient type of Alpine rural settlement. A system of small villages, the "viles", each comprising three or four elementary cells called "masi" is still present in the Dolomites. A "maso" is a set of buildings serving specific functions, whose vertical load-bearing structures are built with wooden frames. Two different types of wooden

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frames are encountered and both are considered in the sample building chosen for the purpose of the present study.
To preserve, maintain and restore this historical heritage, research work must be undertaken. Old techniques must be understood; load-bearing capacities predicted and deterioration and damage evaluated. Especially to fully understand the seismic behaviour of these buildings which was till now mainly assessed by visual and qualitative inspection, quantifying investigations are needed.
Therefore, cyclic tests evaluating the (for seismic performance important) dissipative and ductile properties of the construction technique and a subsequent numerical modelling were carried out.
In order to carry out “realistic” tests, the building structure was analysed. With the outcomes of this analysis, the wall specimens were then defined and in-plane cyclic tests were executed.
The test outcomes were necessary to evaluate the walls’ properties such to be able to calibrate a numerical model of a building starting from test data of walls. With this calibrated virtual model of a building, earthquake simulations were carried out investigating in the seismic behaviour of these structures under different earthquake loadings.
This paper is presenting the architectural, experimental and finally analytical part of the project.

STRUCTURAL CHARACTERISTICS OF THE BUILDING TYPES

The building serving as “model” structure can be found in Cibiana di Cadore, in the hinterland of Belluno province in the Dolomites of Northern Italy.

Building methods

The original building has an irregular layout organised on three levels. The ground floor is made of masonry, while the load-bearing structures of the first and second floors are made of timber.
As already mentioned above, the wooden frames can be distinguished in two different categories; “type 1” and “type 2”.
“Type 1” is made using the so-called “gardiz” technique characterised by a web of poles driven into the ground with horizontally-placed branches forming a grid, that was spread with mud that dried in the sun.
The wall is composed by a main set of horizontal and vertical members of spruce (its load-bearing structure) connected with dovetail joints. The filling of the frame is made of lime mortar and roughly-crushed stones “reinforced” with a trellis of hazelnut branches. This trellis is then covered with a layer of plaster to achieve a final thickness of 20 cm.
In some cases the joints are simplified, but are completed with soft iron nails. In other cases, the same frames are stiffened with diagonal bracing nailed onto the outside to cope with instability phenomena developing due to progressive degradation and the consequent loss of efficacy of the connections of the main members.
“Type 2” frames differed essentially from “type 1” in that they included diagonal elements (again spruce), lying in the same plane as the main frame members with a filling that has no grid support.
The load-bearing frame structure is made of square beams connected with lapped joints and nails. The diagonal planks were not connected to the main frame, being simply rested against the inside of the beams. The filling is made of large-sized crushed stone and lime mortar.

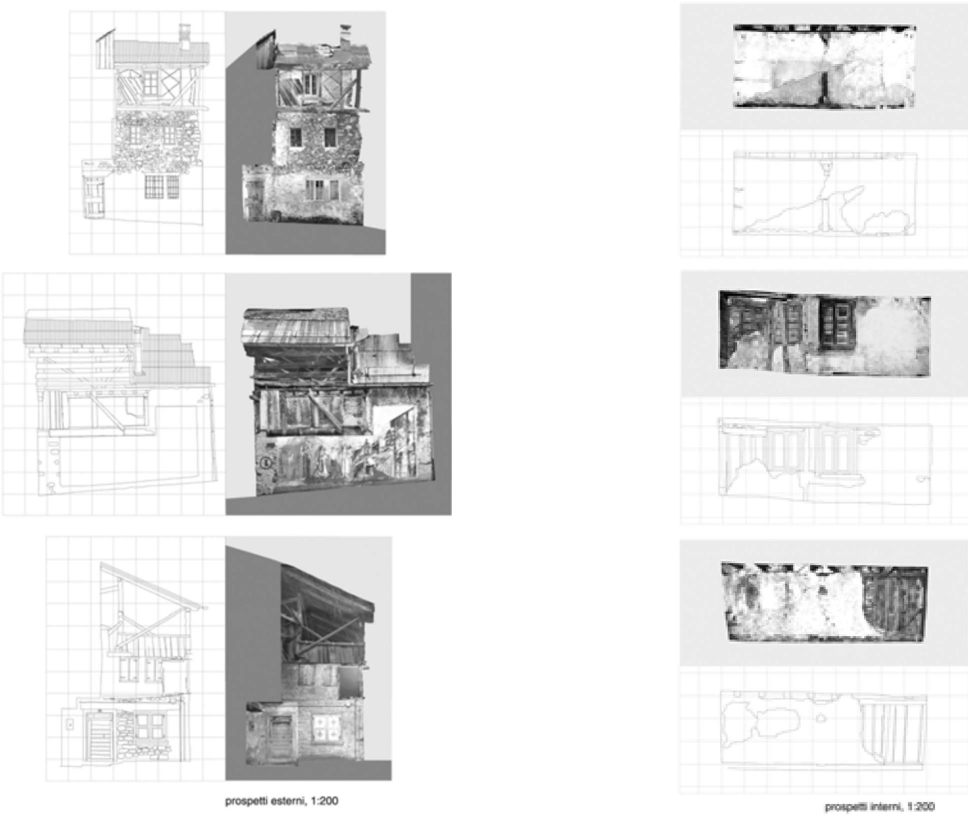

Figure 1: building of Cibiana di Cadore

Material properties and mechanical characterisation of the elements

There are essentially three features which influence the mechanical behaviour of the timber frames and which have to be evaluated correctly: the connections, the strength of timber and the type of filling.

The behaviour of the connections can be considered as a function of the quality of their execution, the material properties and the degree of deterioration. Moreover, the connections introduce a fundamental aspect in terms of the building's seismic behaviour, i.e. ductility that acts as a safety valve for the building as it enables dissipation of energy.

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 a sample frame - a behaviour that is difficult to reproduce when the tested models are newly built with healthy timber.

Moreover, two factors that depend exclusively on the test methods can also have a considerable influence on the outcomes of the tests, i.e.

— the rate of application of the load and the duration of loading;
— the loading frequency and alternation, if any.

Finally, although it is not part of the load-bearing structure, the filling plays a fundamental role in the global behaviour of the building. It is influencing the structural behaviour by means of its position, stiffness and strength. Its mechanical characterisation proved particularly difficult, however, for at least two reasons, i.e. because its strength and stiffness could not be reliably determined and because it was impossible to schematically reproduce the filling and conditions of constraint between 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 frames in withstanding seismic actions depend on the capacity of the filling to brace the structure and on the mechanical and technological features of its internal connections and their ability to assure a plastic, energy-dissipating behaviour.

Quantifying the lateral stiffness is somewhat complicated, however, because the values being sought depend largely on three fundamental characteristics:

— the frequency and regularity of the building's internal connections;
— the repetition of vertical elements approximately every 60-80 cm in the vertical structures;
— the presence of filling in the load-bearing wooden frame.

During an earthquake, the filling can collapse while the wooden frame remains standing; deformations and overstrain of weaker parts are not transmitted to the load-bearing structure. The elements of the building structure can become independently deformed while simultaneously collaborating.

Eurocode 8 gives an explanation for the diaphragmatic behaviour of such filling walls under seismic actions. EC8, part 1.3 "Specific rules for wooden buildings" states that "filling panels exhibit an excellent seismic 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" frames, most common in Cibiana di Cadore, almost never include bracing systems such as struts or cross-stays. "Type 2" frames contain diagonal elements, but these are never connected to the load-bearing frame. This entitles us to assume that the filling has always ensured a good seismic performance.

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 estimate in the original buildings, however.

During an 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.

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.

Therefore we have possible displacements without unacceptable states of stress in the various timber structural elements. It guarantees a good energy dissipation due to a localised compression perpendicular to the fibre and a fair amount of energy dissipation due to friction between the various structural elements converging in the node. The connections can withstand very important strains before the timber 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.

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.

Six models of wooden frames were produced, 4 representing “type 1” and 2 for “type 2”. Frames M3 and M6 are models that were modified with respect to the original features of the connections in order to represent two possible solutions for improving the mechanical behaviour of the joint.

FRAME	CODE	FILLING	CONNECTIONS
Type 1	M1	Branches+plaster	Dovetail joints
	M2	Branches+plaster	Dovetail joints (larger joints)
	M3	Branches+plaster	Dovetail joints stiffened with inclined screws
	M6	Branches+plaster	Joints with EPDM shock-absorber pads and inclined screws
Type 2	M4	Stones+plaster	frame with lapped joints and screws
	M5	Bricks+plaster	frame with lapped joints and screws

Table 1: typology of wall specimens tested

Geometrical and structural characteristics of “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 and 8 smaller vertical beams with a cross-section of 7 x 7 cm inserted directly in the two horizontal beams.

In the M1 frame, the dimensions of the joints are identical to the originals, while the joints in frames M2, M3 and M6 are larger; the reason for this is that the six dovetail joints of the second “type 1” model broke during the construction. This prompted us to increase the size of the joint in order to avoid any further damage during the assembly stages. This 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 grid made of hazelnut sideshoots (branches) arranged horizontally and woven between the vertical beams covered with a layer of hydraulic.

Models M3 and M6:

— the load-bearing structure was the same as for models M1 and M2; the only difference concerned the dovetail joint. They were stiffened by inserting special self-tapping wood screws (HecoTopix) in a crosswise layout. Model M6 features an additional EPDM damping pad between mortice and tenon.

The contribution of the screws is the stiffening of the mortice and tenon joint, thus increasing the strength of the wooden-frame in its elastic phase. The EPDM pad supports energy-dissipation.

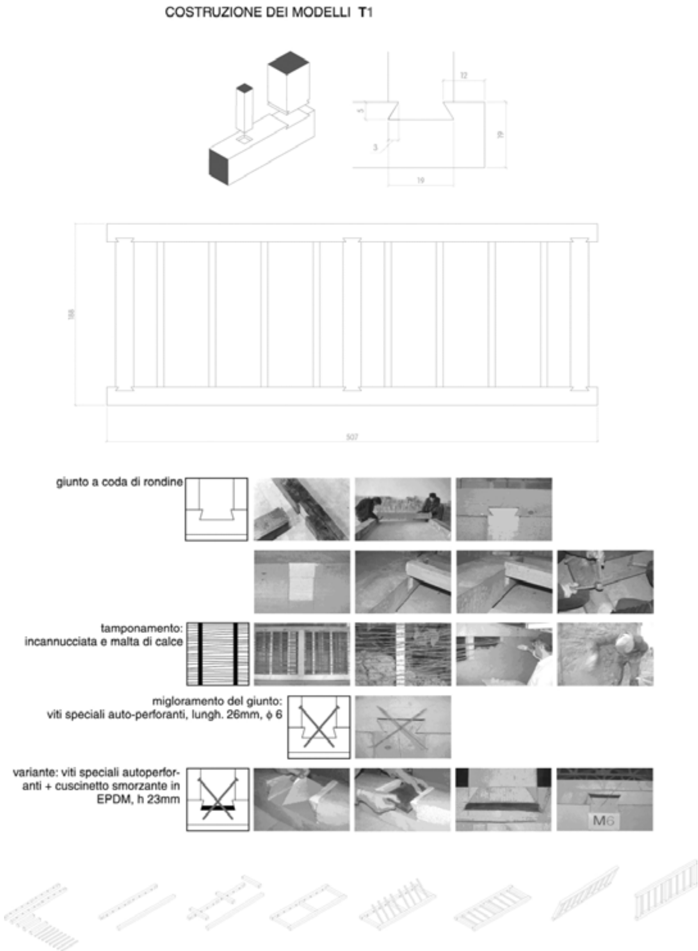

Figure 2: “type 1” models

Geometrical and structural characteristics of “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. This type of masonry exactly reproduces the original found in situ. Each “quadrant” of masonry has three elements 3 cm high placed diagonally to form crosses.
Model M5 has a brick infill.

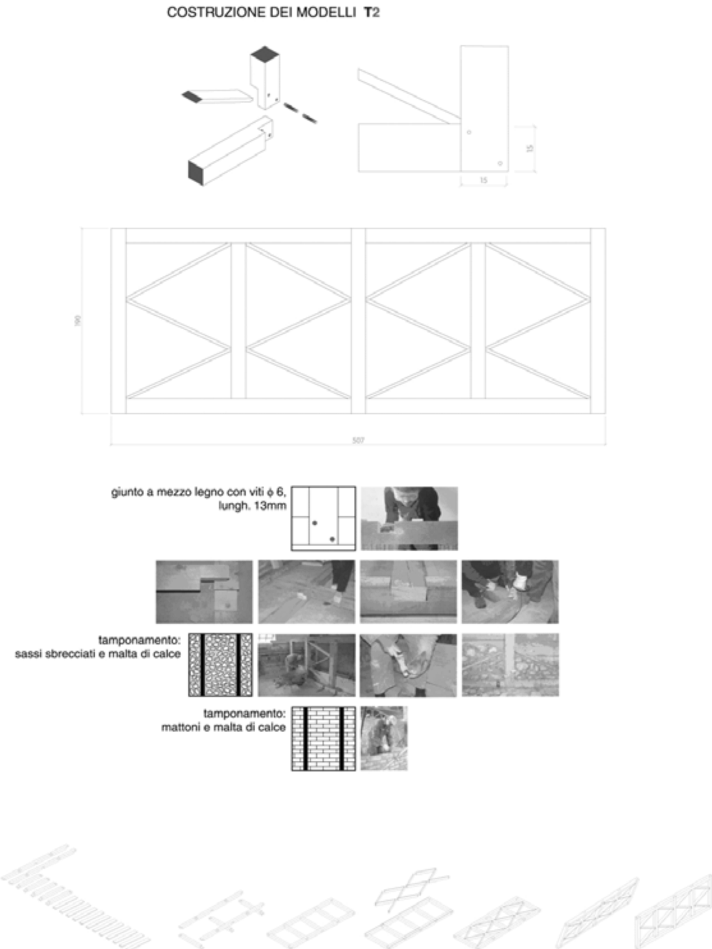

Figure 3: "type 2" models

Test methods

The tests on the real-scale models were carried out by means of load increments of 10 kN per cycle up to failure. The load was applied by 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 frame. The horizontal displacements were recorded by two strain gauges with a sensitivity to $1/10^{th}$ of a millimetre attached to the wooden pillars.

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.

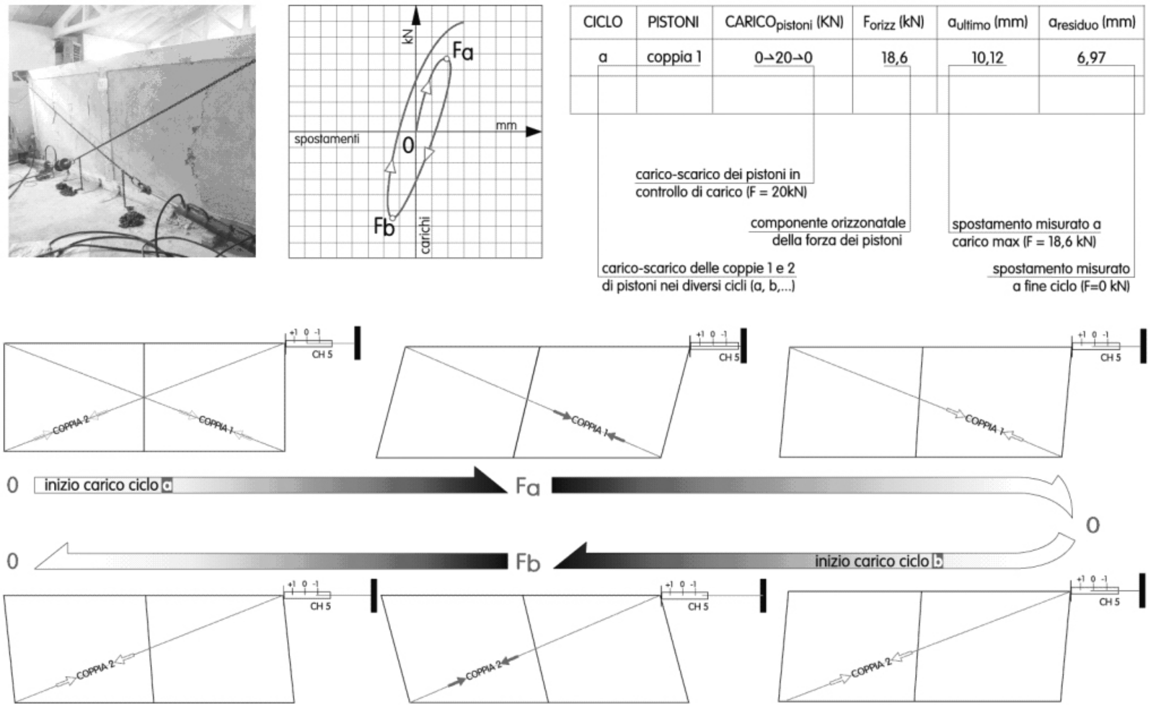

Figure 4: test method

ANALYSIS AND INTERPRETATION OF THE CYCLIC TEST RESULTS

The results confirmed the predictions concerning the behaviour of the models under a horizontal action:

— by comparison with M2, model M3 demonstrated greater strength but no change in ductility (F_{max} for M3 > F_{max} for M2);
— by comparison with M3, model M6 showed an increase in ductility with a strength at collapse that remained unchanged (F_{max} for M3 = F_{max} for M6);
— by comparison with “type 1” panels, “type 2” panels 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 than the one provided by the trellis and mortar.

It was the way in which the two different “type 2” frame 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 timber frame, the brickwork masonry parts did not collapse at failure, they simply became cracked. Instead, 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.

	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'	

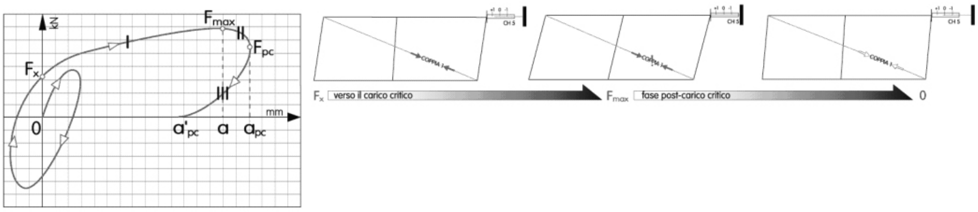

Figure 5: test results

NON LINEAR DYNAMIC ANALYSIS

Having the results of the cyclic tests, nothing is yet known about the seismic behaviour of an entire building except for qualitative statements. But in terms of “numbers”, of a quantitative investigation, a proper theoretical analysis must be undertaken. For this scope, a non-linear analysis was carried out to evaluate the seismic behaviour of this construction type under real earthquake excitations.

Non-linear analysis programme and model calibration

The Software DRAIN2DX was used to undertake the numerical analysis. This software works with simple structural models, but is capable to simulate the behaviour of an entire structure in contrast to many other sophisticated, even predictive models with which one is able to exactly repeat the behaviour of a shearwall under cyclic loading, but not the behaviour of more complex structures such as entire building structures.

The original DRAIN2DX software developed at the University of California in Berkeley was extended and modified by researchers of the University of Florence. [CECCOTTI, LAURIOLA, FOLLESA] They extended the programme with a subroutine describing the moment-rotation behaviour of a semi-rigid joint – which are describing mechanical connections in timber. In other words, they introduced rotational semi-rigid elements to simulate pinching hysteretic behaviour of the connections. The subroutine is based on an algorithm allowing for moment-rotation diagrams. The diagrams (see Fig. 6) represent a piecewise linear simplification of cyclic test data from which the different stiffness values, the “inclinations” of the lines, at various stages of displacement/force are derived.

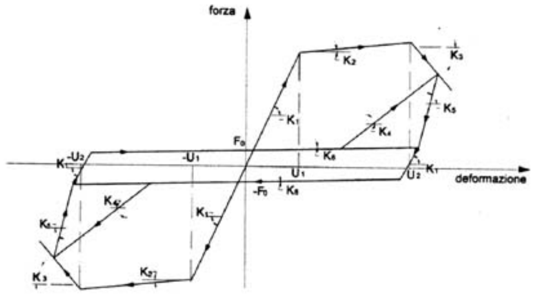

Figure 6: hysteresis cycle

The term “envelope curve” of a cyclic test is denominating the curve wrapping, “enveloping”, the loops as if it would be a monotonic test curve (which in fact should also be carried out when undertaking cyclic testing). Hence this simple curve is representing the initial elastic stiffness of a system, its ultimate load-carrying capacity and its ultimate slip.

First of all, the model had to be calibrated. The tested timber frames were simplified. The beams and columns were assumed to be infinitely rigid connected with rotational springs in the corners. The filling and other timber members were schematised by the viscous damping and the stiffness values of the springs derived from the cyclic tests. The mass of the frames was concentrated as a lumped mass in the two upper joints of the frame. This means, all structural behaviour of the model was depending exclusively on the stiffness values of the rotational springs having the beams infinitely stiff.

This was then used to calibrate the model by fitting it to the testing data. From the obtained load-slip curves, the various stiffness values of the hysteresis loops could be determined. The fitting of the model to the test data had to be done regarding three criteria. The dissipated energy (which is the area covered by the loops and should be the same (±10 %) for test and model), the maximum deformation and the maximum force. In Fig. 7 below, the fitting is illustrated on panel M4.

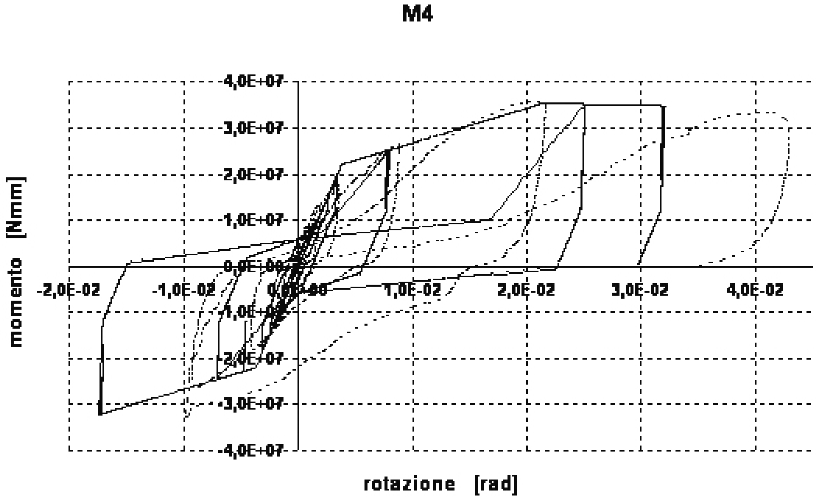

Figure 7: overlap of tests data (dashed line) and DRAIN2DX model (continuous line)

The excitation input to calibrate the model was the time-force history recorded during the tests. Only after fitting, earthquake calculation could be undertaken.

Earthquake simulation

The numerical model was calibrated on the cyclic test series and the stiffness values of the springs were adjusted. Having finished this preliminary work, the investigations on the seismic performance of traditional timber frame buildings could start.

For this purpose, a computer model of a two-storey house was built. The simulation was done in 2D, not in 3D and thus, the model was representing a wall in whose direction the earthquake input was acting. For simplicity, the length of a wall was 5.09m and the height of one storey was 1.90m which were the dimensions of the tested walls.

However, seismic research on modern “timber-frame” structures, the so-called platform frame constructions which are constituting e.g. ca. 80 % of the North-American residential housing, has shown that the relationship of the stiffness values between different lengths is more or less linear.

The design loads were taken from Eurocode 1; with the permanent action as resulting from the traditional structure and the variable action for residential housing. The masses were modelled as lumped masses concentrated in the two upper joints of each storey.

Five different “houses” were modeled; using wooden frames M2 (branches), M3 (branches, screws), M6 (branches, screws, EPDM pad), M4 (stones) and M5 (bricks).

The chosen real earthquake accelerograms can be seen in Table 2.

EARTHQUAKE	MOMENT MAGNITUDE	DURATION [sec]
Brienza 23/11/1980	6.9	20.014
El Centro N/S 19/05/1940	7.1	29.000
Izmit 17/08/1999	7.6	85.795
Kobe E-/W 16/01/1995	6.9	20.010
Loma Prieta E/W 18/10/1989	6.9	39.980
Mexico City 19/09/1985	8.0	53.500
Northridge E/W 17/01/1994	6.7	19.980
Tolmezzo 6/5/1976	6.5	21.029

Table 2: chosen earthquakes for analysis

These earthquakes were acting as ground motions at the base of the model (the stone plinth of the original building was neglected; the seismic action was assumed to push directly at the timber-frame structure). The variable to play with in order to change the intensity of an earthquake, is its Peak Ground Acceleration (PGA). The PGA is the maximum acceleration value reached by the quake and is expressed as a fraction of the gravity force g. In the Italian seismic code (2005) for instance, the most hazardous earthquakes have a PGA value of 0.35g. The big Japanese earthquake of Kobe in 1995 had a PGA of 0.8g and the classical earthquake for seismic research, the Californian El Centro Quake in 1940 had a PGA of 0.31g. During the analysis, this PGA value was changed and increased until the collapse of the house had occurred. Therefore, before showing the analysis results, a collapse criterion must be introduced.

Collapse

Establishing a collapse criterion for timber structures in general is always a difficult issue. It is difficult to define a point when a timber structure has to be regarded as collapsed. “Collapse” means the non-repairable damage of a structure, such extensive that no further action than demolition can be undertaken. The collapse is usually defined as the deformation at 80% of the

maximum load-bearing capacity. Apparently, it is important to agree on a certain criterion to be able to compare results and research outcomes.
The difficulties with the definition of the structural collapse at 80% of the maximum load-bearing capacity is evident when looking at the load-slip graphs of the cyclic tests. When we look for example at the envelope curve of panel M4 (fig.7), the decrease of the force after reaching $F_{max,}$ is nearly not existing. In this case, the collapse displacement was reached for a displacement value smaller than the one at 80% of the maximum load bearing capacity.
Therefore, the chosen values for the collapse displacement were the “elbow values” at the sharp bend of the envelope curves of the cyclic test data (these values can be taken as the wall of the house has the same length as the wooden frames tested). Below the values:

FRAME		COLLAPSE DISPLACEMENT [mm]
Type 1	M2	31
	M3	32
	M6	50
Type 2	M4	81
	M5	68

Table 3: collapse displacements

RESULTS OF EARTHQUAKE SIMULATION

The following Fig. 8 shows the outcomes of the earthquake simulation carried out on a 2D model of a two-storey timber-house. The calculations were carried out varying the Peak Ground Acceleration (PGA) value until the collapse displacement as defined in Table 3 for the five “house” types was reached.

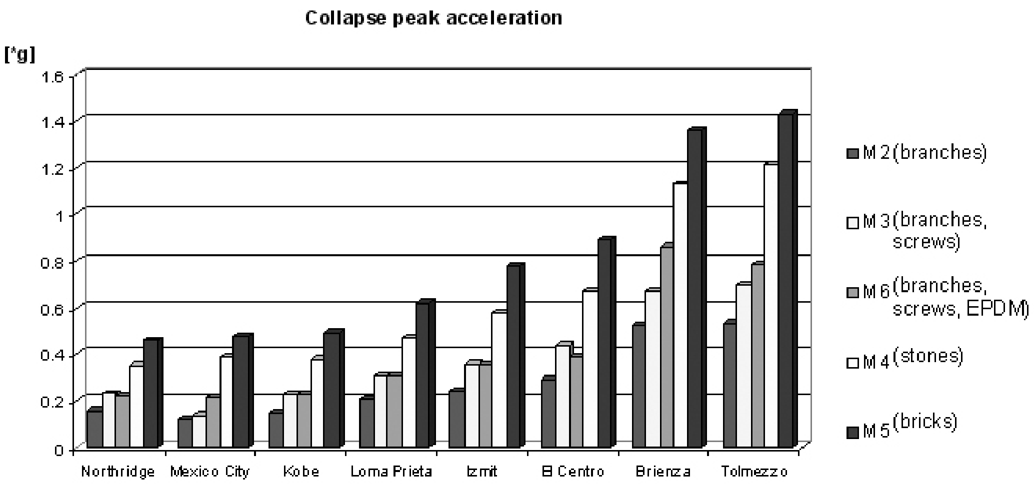

Figure 8: outcomes for the eight chosen earthquakes

The most devastating earthquakes were Northridge, Mexico City and Kobe. “Type 1” houses were less resistant for the Mexico City quake whereas “type 2” houses were less resistant for the Northridge quake. Especially for ‘house’ M3, the bearable maximum peak acceleration differs for Mexico City and Northridge earthquakes. It is evident that the resistance of a structures varies with the input quake. A structure can resist a certain earthquake, but collapse for another.
The analysis results show clearly that for every eight chosen earthquakes, ‘house’ M5 resisted the best. Panel M5 with brick infill had the highest static load-bearing capacity, but at the same time it was the stiffest panel. It was expected that ‘house’ M6 would resist better, that it may even overtake ‘house’ M5 for some earthquakes, as panel M6 was less resistant but had a much

higher ductility. ‘House’ M5 resisted also better than ‘house’ M4 (stone infill). Panel M4 was, like panel M6, less resistant but more ductile than panel M5. Unlike panel M6 though, M4 was not completely opposite to M5 in terms of static resistance and stiffness but still the higher load-bearing capacity of panel M5 turned the balance.
The results for the “type 1” ‘houses’ with infill of hazelnut branches were not as clear as expected. ‘House’ M2 was the least seismically resistant as expected. But it was estimated that ‘house’ M6 would resist better than ‘house’ M3. Which was the case for three earthquakes (Mexico City, Brienza, Tolmezzo) but no trend can be established. Mostly, M6 with its damping pads and screws resisted in almost the same manner as M3 only with screws. It should be underlined that panel M6 did not show strength impairment during cyclic testing in contrast to panel M3. However, the enhancement of “type 1” frames with modern devices is evident.
Hence, the two most interesting panels were M5 as the most resisting and M6 as the generally most resisting and most sophisticated “type 1” panel.
When considering the ratio between these two panels in terms of static load-bearing capacity and seismic resistance, one can see that panel M6 was catching up. Fig. 9 shows this relationship.

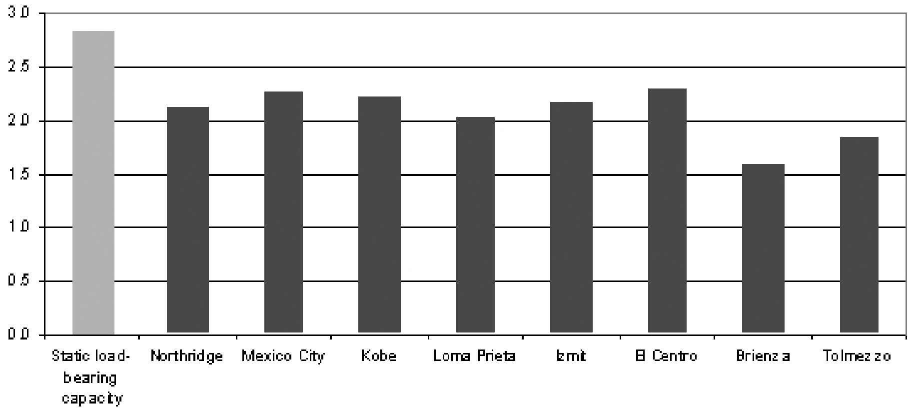

Figure 9: ratio between resistance of M5 and M6 in terms of static (grey column, frame) and seismic (black columns, ‘house’) load-bearing capacity

Whereas panel M5 was nearly three times as resistant as panel M6 in terms of static load-bearing capacity, the seismic resistance of ‘house’ M5 is reduced to averagely two times the resistance of ‘house’ M6. For the two “smaller” Italian earthquakes, this factor was even more decreasing up to 1.5 for Brienza. The “type 1” panels were hence “catching up” with the most seismically resistant panel M5.

SEISMIC DESIGN

In Eurocode 8, for regular structures, the dynamic seismic actions are transferred into horizontal static forces. These forces depend above all on the mass of the building and the expected peak ground acceleration for this region. Other parameters to consider are dealing e. g. with soil types and the frequency of the building, but for the sake of simplicity the concept of seismic design is explained here only very briefly.
As we have seen till now, the reaction a building shows towards a seismic action is also depending on its capacity to dissipate energy, on its ductility. Therefore, an action reduction factor, the so-called behaviour factor q, is introduced in most seismic codes. Depending on the type of structure, the horizontal forces are reduced by this factor q. This means that an elastic structure without any ductility will have a q-value of 1 whereas a very ductile hysteretic structure

that dissipates a lot of energy may have a q-value of 5. The horizontal forces are reduced 5 times in this case. In Eurocode 8, the behaviour factor q is, at the moment, reaching from 1 to 5.
Knowing this, it is obviously important to know which “dissipation class” a building structure is belonging to in order to establish its seismic behaviour factor q. For the traditional buildings of the present study, such a verification was carried out.
A PGA of 0.25g was assumed and the floors were assumed to be rigid. The q-value was assumed to be 2. Knowing the mass of the considered building, the horizontal forces were calculated. Hence the forces to which the building must withstand were known. This allowed for the re-calculation of the stiffness values. These new stiffness values were evaluated knowing the lateral resistance per meter of the walls from the cyclic tests (static load-bearing capacity) and the horizontal force acting during an earthquake with 0.25g. This allows to determine the necessary length, “quantity of wall”, that is necessary to withstand the seismic event. Assuming stiffness proportional to strength and a linear relationship of stiffnesses for different wall lengths, the new stiffness values necessary to resist a design earthquake with 0.25g were calculated.
Again, the non-linear analysis is carried out applying the eight earthquakes as excitation function and varying the PGA until the collapse displacement is reached. In Fig. 10 below, the results for the analyses with the “new” stiffness values for a design earthquake with 0.25g is shown.

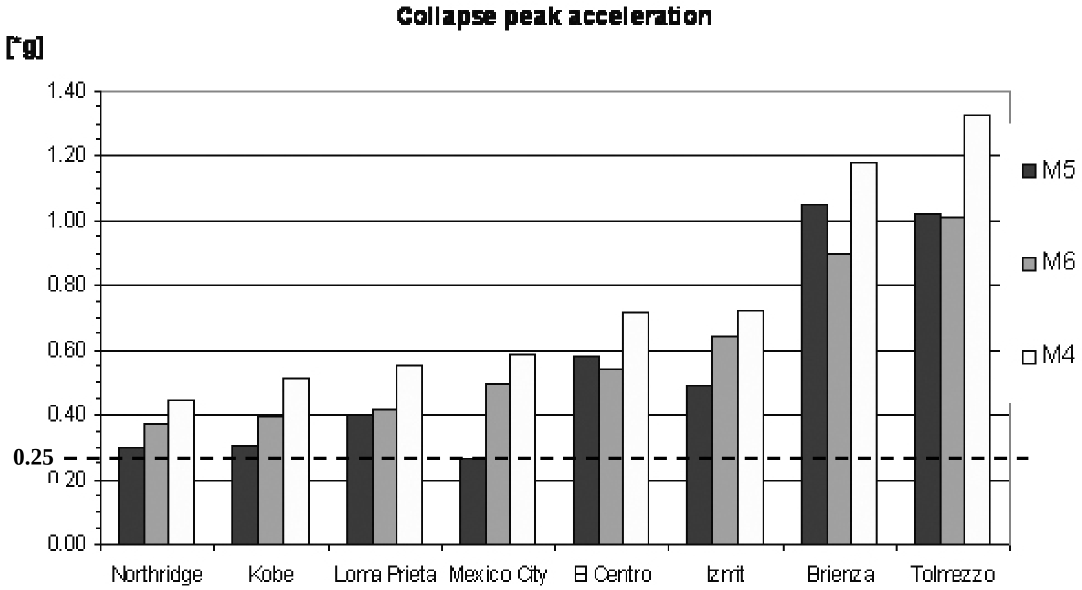

Figure 10: seismic design, ultimate peak ground accelerations

All the three ‘houses’ resist the earthquakes, the ultimate peak ground acceleration PGA_u sustainable by the structures is always higher than the designed 0.25g – although ‘house’ M5 just resists with an PGA_u=0.263g for the Mexico City earthquake. Therefore, the behaviour factor q=2 suggested in Eurocode 8 proved to be sufficient. For most of the cases, with a behaviour factor q=2, the seismic resistance of the panels is very much on the safe side; especially for the two Italian earthquakes of Tolmezzo and Brienza where the PGA_u of the panels is at least thrice as high as the design PGA of 0.25g.
Once again the well-known fact in seismic design is evident – a structure has not the same resistance and behaviour for different earthquakes. When comparing these outcomes to the analysis results shown in Fig. 8, for the Mexico City earthquake ‘house’ M6 resists e. g. much better when seismically designed (PGA_u of 0.498g versus 0.212g for the non-design case).

Apparently, the behaviour factor q must be chosen very carefully. Whereas q=2 may be very suitable for one construction type, for another it may be less suitable because too conservative. Hence the q-factor must be properly designated in the Eurocode 8 for this kind of structure.
When applying Eurocode 8 in analysing the earthquake behaviour of the structures, 'house' M4 resists the best. For the Brienza, Tolmezzo and El Centro accelerograms, M5 has a higher collapse peak acceleration than M6. For the other 5 accelerograms though 'house' M6 is the clear winner over 'house' M5.
As the static load-bearing capacity is considered in the seismic design procedure and not only the mass of the building structure as during the first earthquake simulation, obviously the influence of the much higher resistance of panel M5 will reduce the seismic resistance of 'house' M5. Therefore, "type 1" panel M6 is further catching up regarding the ratio between resistance of panel M4 (here the most resistant) and panel M6 in terms of static and seismic load-bearing capacity.
As before, the more rigid structure (M4 with stone infill) has a higher seismic resistance than the more ductile structure with a lower static load-bearing capacity (M6 with infill of hazelnut branches). Contrarily to the previous outcomes though where both the "type 2" panels with stone resp. brick infill resisted better, in the case of seismic simulation based on seismic design after Eurocode 8 panel M6 resisted for some earthquakes much better than the panel with brick infill.

CONCLUSIONS

The present study showed that the construction methods used in the historical buildings studied here possess characteristics and qualities that are genuinely effective in terms of earthquake resistance.
A comparison between the test results and the direct reading of existing buildings revealed precious information with a view to restoration and rehabilitation of these rural buildings without altering their original structural design. It showed also how their seismic safety can be improved by means of retrofitting with modern tools.
These first quantitative investigations on the seismic behaviour of historic timber frame constructions proved to be very valuable. An approach with methods known from similar investigations on modern timber structures, especially the Platform Frame system, is possible. DRAIN2DX improved with the Florence pinching hysteretic model is a valuable and powerful non-linear dynamic simulation programme which idealises the seismically most important parameters, i. e. stiffness and ductility, of the timber frames in a satisfying way.
The main objective of this work was achieved. The cardinal question of whether rigid structures with a high static load-bearing capacity or ductile structures with a low static load-bearing capacity are seismically more resistant was answered. The rigid structure with a high static load-bearing capacity resisted better (as can be expected anyway). This was the case in both investigation types. When approaching the problem with a simple "handicraft" method as well as when undertaking a seismic design procedure, for every single chosen earthquake, the rigid structure with a high load-bearing capacity supported higher peak ground accelerations.
An important part of the study though is the quantitative survey between "static" and "dynamic" behaviour. It is a well-known fact that ductile structures dissipate more energy which is quite favourable when the action on the structure is of dynamic nature – as are earthquakes
When considering hence the ratio between the resistance of the ductile "type1" panels and the resistance of the rigid "type 2" panels in terms of static and seismic load-bearing capacity, the

‘houses’ made with ductile panels catch up. Whereas the rigid panels have a static resistance about three times the resistance of the ductile panels, the seismic resistance of the rigid panels is only about two times the one of the ductile panels.
When analysing a structure designed with the European Standard, none of the structures designed with a behaviour factor q=2 collapsed. Therefore, the behaviour factor suggested in Eurocode 8 proved to be sufficient, even overly conservative.
The analysis results of the seismic simulation applying Eurocode 8 also indicate the influence of the behaviour factor q. Whereas in the analysis results using no seismic design code (the “handicraft” method) always panel M5 supports the highest ultimate peak ground acceleration this is not the case for the analysis using Eurocode 8. In the seismic design case, panel M5 is the worst and panel M6 is catching up considerably. Therefore, a behaviour factor q=2 may be very suitable for one panel type it may not be a good choice for another type because it is too conservative.

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