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PROCEDURES FOR SEISMIC CHARACTERIZATION OF TRADITIONAL AND MODERN WOODEN BUILDING TYPES

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Abstract: *The paper analyzes different wooden buildings types used in past and nowadays to realized low-rise and mid-rise timber structures from the seismic point of view.*
A preliminary overview about the procedures prescribed by codes for the seismic characterization of the timber building systems is given. Then the definition of the behaviour q-factor in the literature and its relevance in design of structures in seismic areas is treated. Available research methods for estimating the q-factor based on the verification of the non-linear seismic response of entire buildings by means of experimental tests and numerical simulations are presented and analyzed.
The relevance of a proper definition of the yielding limit and of failure condition in the seismic characterization of wooden building systems is treated.
Moreover, a comparison between the q-factor estimations obtained using different calculation methods is presented. Lastly, the appropriate q-factor values are given for a reliable and safe seismic design of buildings realized using the examined wooden constructive systems.

Keywords: *wooden structure, tmber shearwall, seismic behaviour, q-factor, ductility, bi-linearization criteria, numerical method*

1. Introduction

Timber constructions subjected to earthquake actions provide relevant advantages if compared to traditional materials. Forces in an earthquake are proportional to the structure's weight and wood is substantially lighter than steel or concrete, also with a favorable strength/weight ratio.

In timber structures the energy is dissipated during cyclic loading in the connections via mechanical fasteners. Decisions about expected ductility levels for particular structural systems are based on the expected capability of connections within them to do plastic work if overloaded (i.e. loaded over yielding limit) (Piazza *et al*., 2011). Timber elements themselves have limited capacity to deform inelastically when overloaded (Smith *et al*., 2003). This capability to resist actions above the elastic range should be considered when designing timber structures.

In seismic design of structures a careful balance of stiffness, strength and ductility is required in order to ensure a good structural performance in terms of serviceability and

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safety in case of minor and major seismic excitation respectively (Schädle *et al*., 2010).

Buildings of regular geometry can be designed for the effects of seismic loadings based on equivalent static force design criteria (CEN, 2013). Available seismic codes that allow equivalent static design practices as alternatives to full dynamic analysis of the effects of design-level seismic events on building superstructures follow the so called force modification design or FMD method (Chopra, 1995). In these cases, the construction method requires the suitable choice of a 'seismic force modification factor' or 'behaviour factor', represented as R in North America and q in Europe (ASCE, 2010; CEN, 2013; IRC, 2010).

To date, there are many different building techniques and systems for wood structures, and these have undergone the normal process of evolution, with progression from traditional to modern methods of building element assembly and the use of engineered wood elements in place of solid wood.

Seismic codes have adapted their provisions to new technologies but the seismic design of timber structure is not as well detailed as the other more common materials yet, and a proper definition of the most suitable behaviour factor for the available timber building systems is a fundamental issue of the codes for structural seismic design (Ceccotti and Sandhaas, 2010).

The seismic characterization of most common traditional and modern timber structures (generally adopted in European area) is presented referring to calculations performed using data both from experimental tests and numerical simulations available in literature.

2. Procedures for seismic characterization of wood systems

The design of structures that can dissipate seismic energy through inelastic deformation allows for the design with reduced lateral forces. Referring to Eurocode 8 (CEN, 2013), lateral forces are reduced by the q-behaviour factor, this represents the ability of the structure to dissipate energy and to withstand large deformations without ruin. In summary, the q-factor gives the reduction of the forces obtained from a linear-elastic analysis, in order to account for the non-linear response of a structure.

A proper definition of the most suitable q-factor for timber building systems is a fundamental issue of the codes for structural seismic design. In fact, available seismic codes provide the q-factor only for standard building typologies and refer to the outcomes from specific experimental cyclic tests to give an estimation of the ductility class and therefore of the most suitable q-factor range.

2.1. Background on q-factor definition

Seismic codes have long relied upon the concept of inelastic spectrum for specifying design actions to be used for elastic analysis of structures which are expected to respond inelastically to the design earthquake. The q-factor used in Eurocode 8 (CEN, 2013) is introduced to reduce "the forces obtained from a linear-elastic analysis, in order to account for the non-linear response of a structure, associated with the material, the structural system and the design procedures".

Based on such definition the q-factor value is strictly correlated to the design procedure prescribed by codes and adopted by engineer for the design of the structural elements. It means that the behaviour factor q is code dependent therefore varying the design assumptions (i.e. reference code, designer safety coefficients) the q-value must vary to account for the different assumptions.

This dependency between the q-value and design codes and rules implies that the q-factor is not representative of the actual intrinsic dissipative capacity of the structures but also take into account a quote representing the effects of design assumptions. According to Pozza (2013), the behaviour factor q can be estimated as the product between an intrinsic part q_0, accounting for the total dissipative capacity and all intrinsic over-resistances and the design over-strength Ω accounting for the code's partial safety factor and for the differences between the design resistance and the applied external force due to design assumptions, Equation 1.

$$q = q_0 \, \Omega \tag{1}$$

Estimation of the intrinsic q_0 factor and of the overstrength factor is not immediate. A graphical representation of such factors is reported in Figure 1 considering an hypothetical bilinear load displacement curve representative of the nonlinear response of a structural system.

According to Elnashai and Mwafy (2002), the intrinsic q_0-factor is given by two contributions: (1) the inelastic capacity of the structure (q_0*) defined as the ratio between the force corresponding to the elastic ultimate strength and the inelastic one, (2) the intrinsic overstrength factor (Ω_i) defined as the ratio of the actual strength to the yielding strength of the structure (Equations 2-4).

$$q_0 = q_0^* \, \Omega_{intrinsic} \tag{2}$$

$$q_0^* = V_{\text{u\_elastic}} / V_{\text{u\_inelastic}} \tag{3}$$

$$\Omega_{intrinsic} = V_{\text{u\_inelastic}} / V_{\text{y}} \tag{4}$$

Design over-strength Ω is given by two contributions, Equation 5.

$$\Omega = \Omega_{designer} \;\; \Omega_{code} = V_y / V_{Ed} \tag{5}$$

Where the first term (Ω_{designer}) allows to switch from external applied force to the design force while the second one (Ω_{code}) allows to switch for the design force to the yielding force, Equation 6-7

$$\Omega_{designer} = V_{\text{R}d} / V_{\text{Ed}} \tag{6}$$

$$\Omega_{code} = V_{\text{y}} / V_{\text{Rd}} \tag{7}$$

This factorization of the q-factor stresses that only when the external applied force (V_{Ed}) coincides with the structure yielding force (V_y) the q-factor and the intrinsic q_0-factor are identical and representative of the actual dissipative capacity of the structure.

In addition this q-factor factorization allows to better understand the actual behaviour of the investigated structural systems because it univocally identifies the contribution of the dissipative capacity and ductility. This is of extreme importance just think, for example, to a structure that behave elastically to external seismic action but is designed using large safety coefficient. In this case the intrinsic q_0-factor is unitary (i.e. elastic behaviour) but the overstrength factor is great. Consequently the global q-behaviour factor could be much greater than 1 inducing to consider the structure as dissipative when actually is not dissipative but respond elastically.

From a research point of view there are substantially three different methods for the q-factor evaluation: (1) experimental methods, (2) numerical methods, (3) hybrid experimental-analytical method. A scheme of available methods is reported in Figure 2.

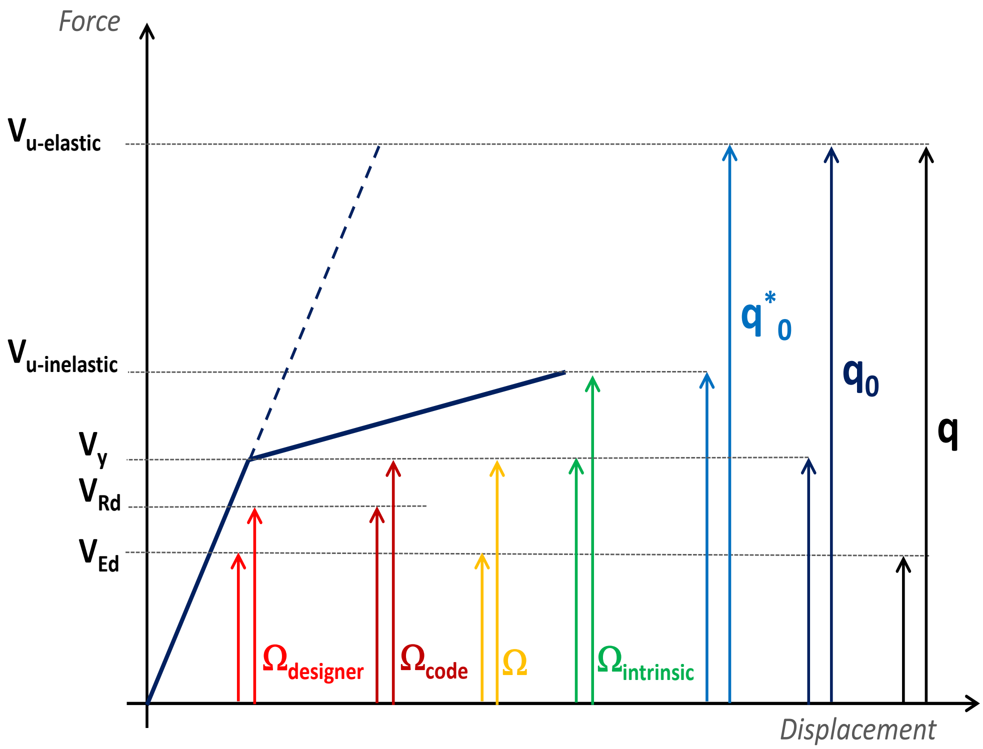


Figure 1. Relationships between the behaviour factor q, the overstrengths Ω, and the intrinsic reduction factor q_0 for a structure characterized by elastic period T<Tc

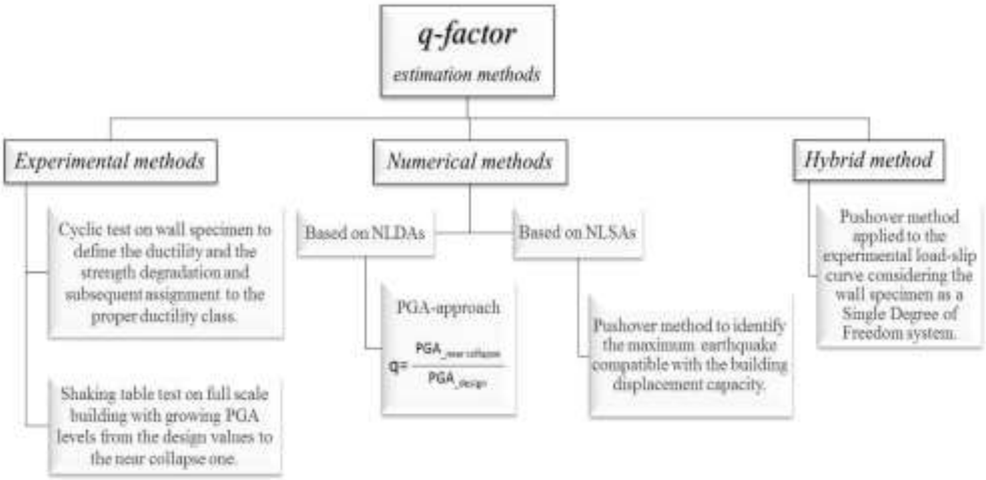


Figure 2. Basic procedure for q-factor evaluation

The three different methods are critically descripted and analysed below. Strengths and limitations of each method are presented and discussed with reference to the intrinsic dissipative behaviour and the overstrength factor.

2.2. Experimental methods

There are substantially two different methods for the q-factor evaluation using experimental tests. The first one is based on the execution of quasi static cyclic tests on assembled wall specimens in order to fully define the hysteretic behaviour.

The second method is based on the execution of a shaking table tests on entire building samples in order to verify the building seismic response under simulated earthquakes.

2.2.1 Method based on quasi static cyclic tests

The experimental tests define the hysteresis behaviour of single connectors or assembled wooden elements (e.g. entire walls). Such experimental cyclic tests are generally performed following the EN 12512 (CEN, 2006) provisions, and they allow defining the following characteristic features: ductility ratio, strength degradation at each ductility levels, equivalent viscous damping.

First attempt to define the behavior q-factor was related to the concept of static ductility as the ratio of ultimate displacement over yield displacement. In Eurocode 8 (CEN, 2013), construction typologies are assigned to different ductility classes. Three ductility classes exist: Low Ductility Class with a correspondent upper limit value of q=1.5; Medium Ductility Class with a correspondent upper limit value of q=2.5; High Ductility Class with a correspondent upper limit value of q=5. The three different classes must fulfill certain requirements of static ductility ratio in order to ensure that the given q-factors may be used. For instance, in Medium Ductility Class: “the dissipative zones shall be able to deform plastically for at least three fully reversed cycles at a static ductility ratio of 4”. Otherwise in High Ductility Class, “the dissipative zones shall be able to deform plastically for at least three fully reversed cycles at a static ductility ratio of 6” (CEN, 2013). For the Medium and High ductility classes the strength degradation between first and third cycles should not exceed 20%.

Although the method is approximate, the q-factor estimation is based on the ductility therefore the intrinsic (q_0) quote of the q-factor is provided.

2.2.2 Method based on shaking table tests

In the last years the construction of bigger and more powerful shaking tables allowed to carry out tests on full scale multi-storey wooden buildings. In literature results are available from these shaking tests on different wooden building systems. The most relevant are briefly described below.

Several researches have already been undertaken to determine the seismic behaviour of timber buildings, mostly in Japan, Canada and the USA. Mainly, these research projects deal with modern wood frame buildings or post and beam structures, as they represent about 50–90% of the residential buildings in these three countries. Full-scale 3D shaking table tests on light-frame wood buildings have also been carried out (Van de Lindt *et al.*, 2010, Van de Lindt *et al.*, 2011). The tests on the 18.6x12.4m six-storey woodframe building and the same building with a steel moment frame on the ground floor served to verify the direct displacement design approach (Pang *et al.*, 2011). The six-storey building was subjected to two-scaled Canoga Park records of the 1994 Northridge earthquake.

Another is the CUREE- Caltech Woodframe project, in which testing activities are included as full scale shake table testing of two story single family houses, three story apartment building with soft story and other simplified models (Reitherman *et al.*, 2003).

In addition a study of wood light–frame buildings under earthquake loading conditions at the University of British

Columbia (UBC) was the Earthquake 99 Project. A specially designed unidirectional shake table was constructed to accommodate the test specimens with plan dimension of 6.1 x 7.6 m and an inertial weight of 200 kN (Taylor *et al*., 2002).

In the last years, the most relevant experimental program involving shaking table tests was the SOFIE project (conducted by CNR IVALSA – Trees and Timber Institute, Italian National Research Council). Aim of this project was the seismic characterization of the Cross Laminated Timber (CLT) building system. This project began with wall tests and pseudo-dynamic test on a single story (Ceccotti *et al*., 2006) and ending with a shaking table test on a CLT-three story building (Ceccotti, 2008). Furthermore, in 2007 an additional shaking table test was carried out on a CLT-seven story building (Ceccotti *et al*., 2013).

Included in the SERIES (*Seismic Engineering Research Infrastructures for European Synergies*) project, a research at the Gratz University of Technology (TU Gratz) regarding the cyclic performance of CLT-buildings was structured in multiple steps, including shake table tests on a three story building. Insert in the same European project the research group of the University of Trento, in cooperation with other European Universities, has been the leading proponent of a program turned to the seismic characterization of three different type timber building: blockhouse, platform-frame (Tomasi *et al*., 2015) and CLT technology.

Method based on shaking table tests firstly involves an extensive experimental program setting up with full-scale earthquake tests according the following steps (Ceccotti *et al*., 2010):

- design the building with q=1 (elastic) and a chosen $PGA_{\_design}$ value according to the available seismic code
- undertake full-scale shaking table tests on the building increasing the seismic intensity until a previously defined near-collapse criterion is reached;
- note the $PGA_{\_near\ collapse}$ value for which the near-collapse state is reached during the test;
- evaluate q_{test} as the ratio $PGA_{\_near\ collapse}$ over $PGA_{\_design}$;
- q_{test} is the experimentally established behaviour factor q.

The thusly established behaviour factor q is only valid for the tested building and the chosen earthquakes. Furthermore the q-factor values is strictly dependent on the seismic code used to design the case study building and therefore to establish the $PGA_{\_design}$. The estimation of the q-factor adopting this method account both for the intrinsic part and for the overstrength one.

Finally in order to generalize the q-factor, more tests on different buildings (same construction technology, different geometry and masses) using different earthquakes should be done. This of course is very costly and time-consuming and rather a theoretical approach as it is not practicable.

2.3. Numerical methods

According to Ceccotti *et al*. (2010), instead of undertaking full scale shaking tests, buildings are modelled and analysed using numerical procedures.

Two different independent procedures can be used to investigate the building systems and performed the q-factor: the first one is based on the output from the Non Linear Static Analysis (NLSAs) while the second one is based on the building load-displacement curve carried out based on Non Linear Dynamic Analysis (NLDAs).

The main aspect of this method is the development of a numerical model which is suitable to reproduce the seismic response of an entire case study building. The most promising approach seems to be reached by cyclic testing (for instance according to EN 12512 (CEN, 2006) of wall elements combined with numerical modeling using the test results as input parameters for complete building models.

2.3.2 Non Linear Static Analysis

A proper application of the pushover method (Fajfar and Gaspersic, 1996) allows defining the maximum spectrum compatible with the displacement capacity of the building performed using of NLSAs. Once this maximum spectrum is defined, the q-factor can be calculated as the ratio between the design spectrum and the maximum compatible one, according the *Figure 2*. This procedure refers to the design spectrum therefore is code dependent providing jointly the quote due to the intrinsic and overstrength factors.

The pushover procedure defined by Fajfar and Gaspersic (1996), with the so called N2 method is specific for an elastic perfectly plastic bi-linearization of the behaviour of the building. Timber buildings generally present hardening post elastic behaviour therefore the procedure described in Albanesi *et al.* (2002) for hardening systems seems to be more suitable for wooden structures.

The definition of the q-factor using the pushover procedure can be affected by the bi-linearization criteria used to switch from the actual building pushover curve to the equivalent bi-linear curve. It should be noted that the bi-linearization procedure affects mainly the elastic branch of the pushover curve and therefore only the elastic period while the displacement capacity is not affected by the bi-linearization criteria. Low and mid-rise timber buildings generally presents a principal elastic period in the plateau range (Schädle and Hans Joachim Blas, 2012) therefore the variability of the elastic stiffness of the bi-linearized pushover curve does not affect significantly the q-factor estimation.

2.3.3 NonLinear Dynamic Analysis

The definition of the seismic response of a building using NonLinear Dynamic Analyses appears to be the most suitable for timber structures as it is independent from the yielding limit definition and refers only to the design condition (defined by $PGA_{\_design}$) and to the ultimate condition (defined by $PGA_{\_near\ collapse}$) for an elastic and an inelastic building response.

Similarly to the procedure used to define the q-factor from the output of the shaking table tests, the so called PGA based procedures start with the definition of a conventional near collapse condition of the building. Using a proper calibrated numerical model the case study building is analysed under growing level of PGA starting from to the design level till the PGA that induce in the structure the near collapse condition. Then the q-factor is defined as the ratio between the calculated $PGA_{\_near\ collapse}$ and the a $PGA_{\_design}$.

This methods allow to define the q factor for a certain building configuration designed following the rules of a specific code and using various engineering hypotheses. Consequently the calculate q-factor is code-dependent. However if a specific design of the case study building is performed avoiding the overstrength factor (Ω), the obtained behaviour factor is representative of the structural intrinsic dissipative capacity (i.e. $q=q_0$).

2.4. Hybrid experimental-analytical method

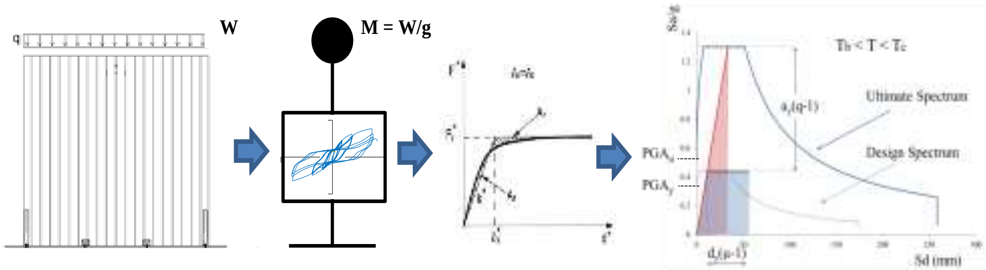
The hybrid experimental-analytical method was firstly applied to timber structures by Boudreault *et al*., (2007) and then improved by Pozza *et al*., (2015-a). Such procedure is a mixed analytical-experimental method which is based on the direct application of the pushover procedure to the load slip curve carried out by means of quasi-static experimental tests on representative wall elements. The procedure is based on the idea to consider the wall specimens as a Single Degree Of Freedom system and allows taking into account the influence of the period of the structure in the calculation of the behaviour q-factor. The main steps of this procedure are (see Figure 3):

- choice of a wall element representative of the investigated building system;
- execution of a quasi-static pushover test under constant vertical load applied to the top of wall to obtain the capacity curve, that is the plot of the applied shear load versus the horizontal top displacement of the wall;
- schematization of the wall as a SDOF system characterized by its capacity curve and mass corresponding to the constant vertical load applied during the pushover test;
- bi-linearization of the capacity curve with consequent definition of the yielding limit and therefore of the elastic stiffness and ductility (Munoz *et al*., 2008);
- application of the pushover method to define the maximum earthquake spectra compatible with the displacement capacity of the wall (ultimate spectra) (Fajfar, 1996);
- definition of the q-factor as the ratio between the PGAs of ultimate spectra and the yielding spectra (Fajfar, 1996).

The procedure provides directly the intrinsic q_0-factor because it refers to the yielding condition disregarding the design of the examined wall system.

The choice of a wall element representative of the timber building system is a crucial issue for a proper estimation of the suitable q-factor. This choice should satisfy the following criteria:

- height and more in general geometrical wall characteristic strictly similar to those effective used in the building structure;
- fasteners typology and arrangement as the typical construction methodology used for entire buildings;
- load condition as that due to the floors and roof dead and live loads.



Characteristic wall specimen | Sdof schematization | Load-slip curve bi-linearization | Pushover procedure for the q-factor evaluation

Figure 3. Main steps of the new SDOF procedure

Taking into account these criteria for the choice of the wall specimen, the q-factor obtained with this hybrid procedure can be assumed as a representative value of the building system and used for the seismic design.

2.5. Bi-linearization criteria , definition of the yielding limit and“near collapse” condition

A relevant aspect for the application of the above presented procedure contains on the execution of experimental tests in order to obtain the capacity curve of the wall. This capacity curve can be obtained based on a monotonic ramp test, performed e.g. according to the EN 594 (CEN, 2011) test protocol. Otherwise the capacity curve can be defined as the envelope of the hysteresis load-slip curve carried out with a cyclic test, performed according to the EN 12512 (CEN, 2006) test protocol. The envelope curve can be obtained using the analytical formulation proposed by Foschi (1977) to fit the trend defined by load-slip cycles.

The capacity curve of a real structure is not regular and generally does not show a well-defined yielding limit. For timber structures the definition of the yielding limit and failure condition is generally made referring to the EN 12512 provisions. This standard proposes a bi-linearization of the experimental curves using the following two different criteria:

- Method (a) is adequate for load-slip curve with two well-defined linear branches: “yield values are determined by intersection of these two lines”;
- Method (b) can be applied for a load-slip curve without two well defined linear branches. The yield values are defined by intersection of the following two lines: “the first line will be determined as that drown through the point on the load-slip curve corresponding to 0.1 Fmax and the point on the load-slip curve corresponding the 0.4 Fmax; the second line is the tangent having an inclination of 1/6 of the first line”.

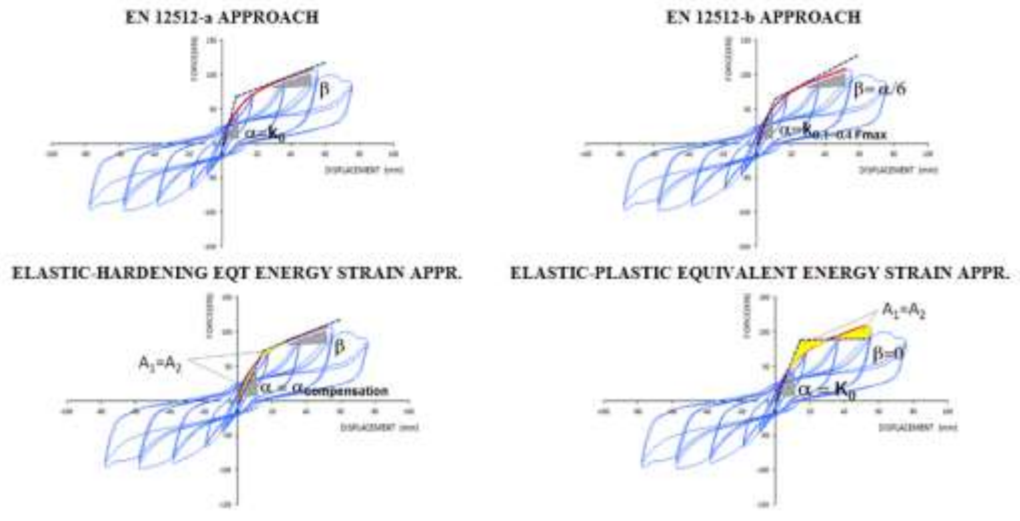


Figure 4. Considered bi-linearization criteria: $A_{1(2)}$ correspond to the strain energy, for other symbols see Figure 5

If the capacity curve presents nonlinear behaviour and relevant hardening phase, the EN 12512 (CEN, 2006) methods should not provide a suitable estimate of the yielding limit and therefore of the ductility. According to Piazza *et al*. (2011) and Jorissen *et al*. (2011) an alternative method to determine the yield limit is to adopt an energetic approach. In literature the most common bi-linearization approach, based on energy strain balance, is the so called Equivalent Energy Elastic Plastic (EEEP) method (Munoz *et al*. 2008). An enhanced of such procedure is proposed by Pozza *et al*.

(2015-a) by means of an elasto-hardening approximation that ensures the equivalence of the energy strain between the envelope and the bilinear curves. In this paper, for a proper bi-linearization of the capacity curve and for a suitable estimation of the ductility ratio, all the four methods for the definition of the yielding limits are used, as summarized in Figure 4.

Another relevant aspect to consider for the applicability of the methods for the q-factor estimation is the definition of the “near collapse” condition. In fact the experimental methods based on shaking tests and the numerical methods based on the usage of NLDAs refer to the so called “near collapse” condition to define the most suitable q-factor of the investigated structure. According to Ceccotti (2008), this “near collapse” condition defines the ultimate criterion for the tests or for the numerical simulations. For timber buildings the definition of the near collapse condition is dependent on the constructive system. As reported in Ceccotti (2008), for solid CLT shear wall the near collapse condition can be defined referring to the failure of the connection elements (i.e. holdown and steel angle) used to join the structural CLT panels at the base and between storeys. Obviously the failure of the connection is defined only by means of experimental tests. When the structure is made by sheeted timber frame the most suitable near collapse condition refers to the inter storey drift as defined in (Schädle *et al.*, 2010).

3. Assessment of the q-factor of various building systems

In this section eight different wooden building systems are analysed and characterized from the seismic point of view. Starting form cyclic experimental tests, the calculation of the bilinear curve that best fit the experimental data is defined and then the estimation of the suitable behaviour q-factor is given for the various building system adopting the previously described methods and referring to results available in literature.

3.1. Description of the case study building systems

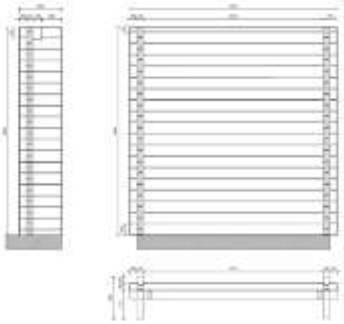
The eight case study building system analyzed in this work, can be classified into three different constructive typologies as follow:

1) Wall composed by linear boards assembled with carpentry joints
   a. Blockbau wall
   b. Layered wall with dovetail insert
2) Wall composed by rigid glued CLT panel assembled with mechanical connections
   a. Un-jointed CLT wall
   b. Jointed CLT wall
3) Wall composed by deformable panel assembled with metal fasteners
   a. Stapled wall
   b. Light frame timber wall
   c. Heavy frame timber wall
   d. Mixed wood-concrete frame

Geometrical features and necessary data of the investigated wall specimens are listed in Table 1. For a more exhaustive description of the investigated wall specimens refer to the listed bibliography.

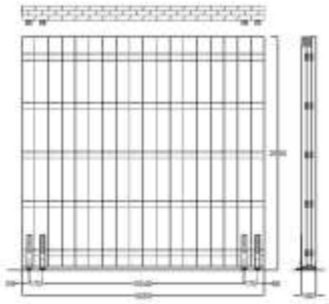
Table 1. Geometrical characterization, connection detail and test protocol of case study wall specimens

1.a – Blockbau wall



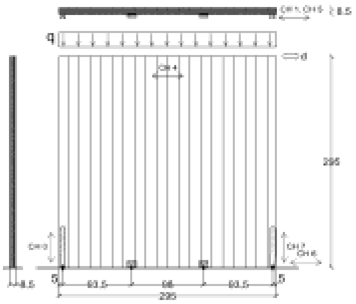
Test Protocol: EN12512 (CEN 2006)
Vertical Load: 10.0 kN/m – Global Mass 2.45 t
Wall dimension: b=2.95m; h=2.95m
Wall characteristic: 90mm x 160mm x 2950mm crosspiece (with tongue and groove interlocking) lay to obtain the main wall. Orthogonally to the wall elements are disposed short elements 90mm x 160mm x 600mm to simulate the effect of two walls orthogonal to the main tested wall. The wall is fixed to the steel base using standard angle brackets and a ϕ 10mm steel cable to prevent the uplift. For an exhaustive description of this construction system see Bedon *et al*. (2014).

1.b – Layered wall with dovetail insert



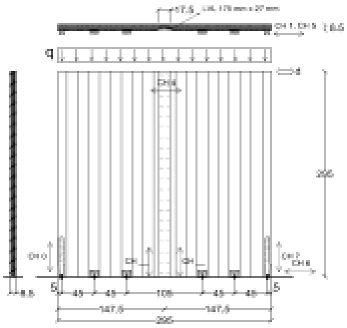
Test Protocol: EN12512 (CEN 2006)
Vertical Load: 18.5 kN/m – Global Mass 5.45 t
Wall dimension: b=2.95m; h=2.95m
Wall characteristic: three layers of vertical sawn spruce boards, thickness 60 mm, coupled with five pairs of horizontal spruce elements inserted horizontally creating a so called dovetail joint. Timber panels were connected to a base larch beam using 18 pairs of cross-screws inclined at 45°with respect to the vertical layer. In addition two hold-downs on each side ware used. For an exhaustive description of this construction system see Pozza *et al.* (2015-b).

2.a – Un-jointed CLT wall



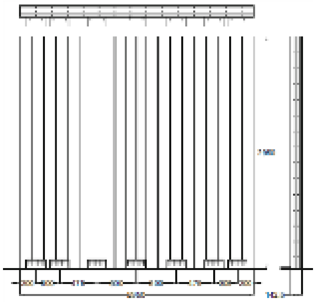
Test Protocol: EN12512 (CEN 2006)
Vertical Load: 18.5 kN/m – Global Mass 5.45 t
Wall dimension: b=2.95m; h=2.95m
Wooden elements: 5 layer CLT panel 85mm thick connected to the foundation by means of: 2 holdwon simposn HTT22 with 12 ϕ 4x60 anular ringed nails to prevent wall uplift and 2 angle BMF 90x48x3x116 with 11 ϕ 4x60 anular ringed nails to prevent wall sliding. For an exhaustive description of this construction system see Gavric *et al*. (2014).

2.b - Jointed CLT wall



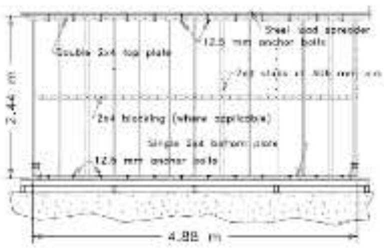
Test Protocol: EN12512 (CEN 2006)
Vertical Load: 18.5 kN/m – Global Mass 5.45 t
Wall dimension: b=2.95m; h=2.95m
Wooden elements: two 5 layer CLT panels 85mm thick connected to the foundation by means of: 2 holdwon simposn HTT22 with 12 ϕ 4x60 anular ringed nails to prevent wall uplift and 4 angle BMF 90x48x3x116 with 11 ϕ 4x60 anular ringed nails to prevent wall sliding.
The two CLT panels are assembled olong the vertical junction line using 2x20 screws HBS ϕ 8x100 – spacing 150mm - inclination 35°. For an exhaustive description of this construction system see Gavric *et al.* (2014).

3.a – Stapled wall



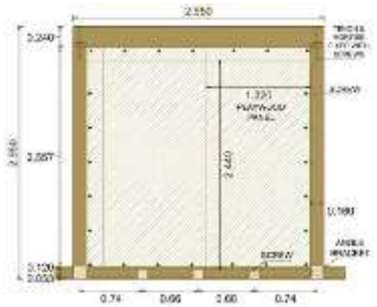
Test Protocol: EN12512 (CEN 2006)
Vertical Load: 18.5 kN/m – Global Mass 5.45 t
Wall dimension: b=2.95m; h=2.95m
Wooden elements: five crossed layers of C24 spruce boards, nominal thickness 28.5 mm and width approximately equal to 200 mm. The layers were stapled to each other with six staples at each node of crosswise jointed layers of boards. The wall was fixed to the foundation with seven angular steel elements which work both by tension and shear loads. For an exhaustive description of this construction system see Pozza *et al.* (2015-b).

3.b – Light frame timber wall



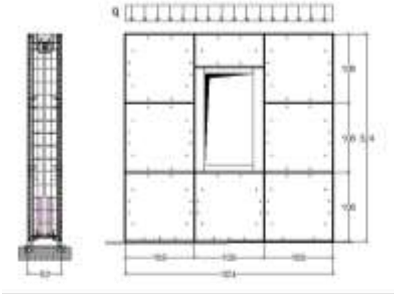
Test Protocol: EN12512 (CEN 2006)
Vertical Load: 8.0 kN/m – Global Mass 4.00 t
Wall dimension: b=4.88m; h=2.44m
The primary structure of the wall is a timber frame made by 50 mm x 100mm elements. The elements of the frame is connected by means of nails. The wall is braced by a double Oriented Strand Board panel nailed to the frame with ringed nails with diameters of 2,3mm and 50m spaced. The wall is fixed to the base using standard bolts (diameter = 12.5mm spacing = 400mm) and standard holdown at the side of frame. A more detailed description of the tested wall is reported in Karacabeyli *et al.* (1996).

3.c - Heavy frame timber wall



Test Protocol: EN12512 (CEN 2006)
Vertical Load: 18.5 kN/m – Global mass 5.90 t
Wall dimension: b=2.95m; h=2.95m
The wall is made of a glued laminated timber portal which is tied to the play wood panel. The connection between the upper beam and the pillars is represented by a mortise and tenon joint fixed with a screw HBS 10x400. The curb-side consist of a larch wood beam. The panel, that is tied to the structure by No. 28 6x120 screws. The connections to the ground basement are carried out by means of screwed steel angle brackets. More detail are reported in Terzi (2010).

3.d – Mixed wood-concrete frame wall



Test Protocol: EN12512 (CEN 2006) + EN 594 (CEN 2011)
Vertical Load: 20.0 kN/m – Global Mass 7.36 t
Wall dimension: b=3.40m; h=3.24m – window 0.82m x 1.60m
Wall characteristic: Heavy timber frame braced by special external concrete slab. Special homemade holdown and screws used to fixed the RC slab to the frame. For an exhaustive description of this innovative construction system see Pozza *et al*. (2015-c).

3.2. Bi-linearization of experimental curve

The hysteretic loop, the related Foschi envelope (Foschi *et al*., 1977) and the four different capacity curves obtained adopting the previously defined bi-linearization criteria are defined and summarized in Table 1. The specific force, displacement and stiffness values that characterized each bilinear capacity curve are also reported according the notation of Figure 5.

F_u: Force at the failure limit;
d_u: Displacement at the failure limit;
k_0: Initial stiffness of the Foschi envelope curve;
k_{pl}: Post elastic stiffness of the Foschi envelope curve;
F_0: Residual force;
F_y: Force at the yielding limit;
d_y: Displacement at the yielding limit;
α: Elastic stiffness of the bilinear capacity curve;
β: Hardening stiffness of the bilinear capacity curve;
$\mu=d_u/d_y$: Ductility ratio;
S.D.=Strength Degradation.

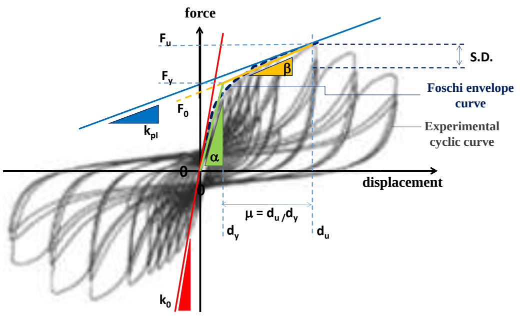
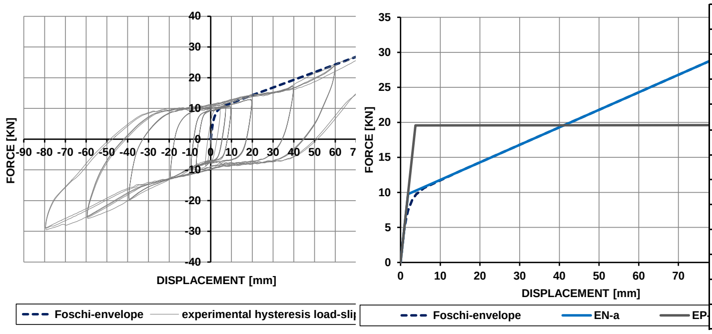


Figure 5. Definition of significant parameters characterizing experimental curve and analytical bi-linear approximation

In addition the ductility classes are evaluated according to the criteria based on the ductility ratio defined by Eurocode 8 (CEN, 2001) and reported in Table 2.

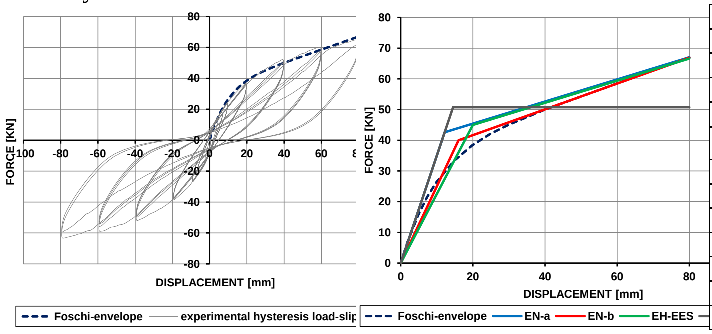
Table 2. Hysteresis loop and envelope curve (left), correspondent bilinear approximation (centre) and significant parameters for seismic characterization of the examined case study wall specimens

1.a – Blockbau wall



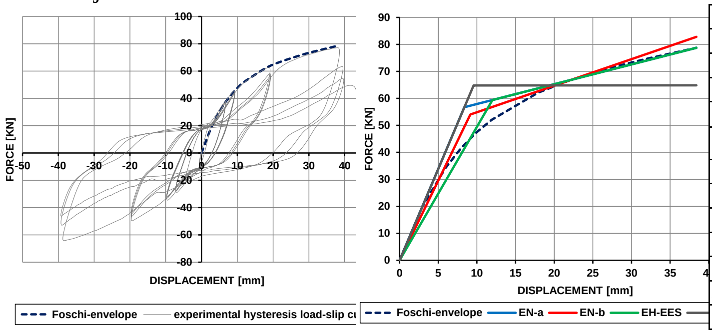
BI-LINEARIZ. METHODS	EN-a	EN-b	EH-EES	EP-EES
F_u [kN]	29.30	-	-	19.57
d_u [mm]	80.00			
K_0 [kN/mm]	5.21	-	-	5.21
K_{pl} [kN/mm]	0.25	-	-	0.00
F_0 [kN]	9.30	-	-	19.53
F_y [kN]	9.77	-	-	19.57
d_y [mm]	1.88	-	-	3.76
α [kN/mm]	5.21	-	-	5.21
β [kN/mm]	0.25	-	-	0.00
μ	42.64	-	-	21.34
Strengt Degradation	< 20%			
Ductility Class	H	-	-	H

1.b – Layered wall with dovetail insert



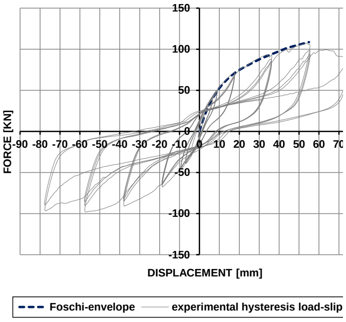
BI-LINEARIZ. METHODS	EN-a	EN-b	EH-EES	EP-EES
F_u [kN]	67.00	65.96	66.60	50.75
d_u [mm]	80.00			
K_0 [kN/mm]	3.50	2.50	3.50	3.50
K_{pl} [kN/mm]	0.36	0.42	0.36	0.00
F_0 [kN]	38.20	33.28	66.60	50.75
F_y [kN]	42.58	40.00	45.00	50.75
d_y [mm]	12.17	16.00	20.00	14.50
α [kN/mm]	3.50	2.50	2.25	3.50
β [kN/mm]	0.36	0.42	0.36	0.00
μ	6.58	5.00	4.00	5.52
Strengt Degradation	< 20%			
Ductility Class	H	M	M	M

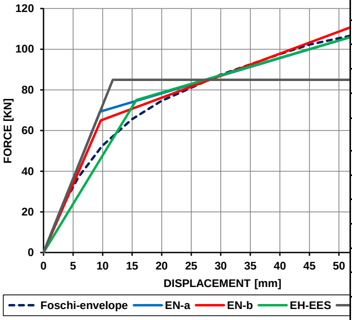
2.a – Un-jointed CLT wall



BI-LINEARIZ. METHODS	EN-a	EN-b	EH-EES	EP-EES
F_u [kN]	78.72	82.79	78.72	64.80
d_u [mm]	38.40			
K_0 [kN/mm]	6.77	5.90	6.77	6.77
K_{pl} [kN/mm]	0.73	0.98	0.73	0.00
F_0 [kN]	50.58	45.00	50.58	64.78
F_y [kN]	56.71	54.00	59.38	64.80
d_y [mm]	8.37	9.15	12.01	9.57
α [kN/mm]	6.77	5.90	4.94	6.77
β [kN/mm]	0.73	0.98	0.73	0.00
μ	4.58	4.19	3.19	4.00
Strengt Degradation	< 20%			
Ductility Class	M	M	L	M

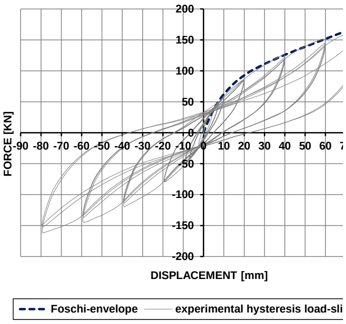
2.b - Jointed CLT wall

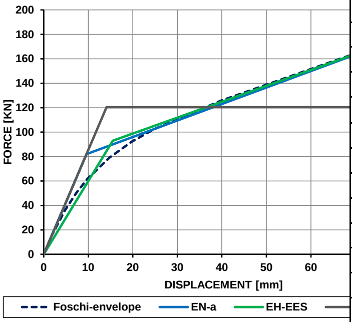




BI-LINEARIZ. METHODS	EN-a	EN-b	EH-EES	EP-EES
F_u [kN]	108.62	114.06	108.62	84.95
d_u [mm]	51.90			
K_0 [kN/mm]	7.25	7.41	7.25	7.25
K_{pl} [kN/mm]	0.93	1.23	0.93	0.00
F_0 [kN]	60.43	50.00	60.43	84.92
F_y [kN]	69.31	60.00	75.02	84.95
d_y [mm]	9.56	8.10	12.50	11.72
α [kN/mm]	7.25	7.41	6.01	7.25
β [kN/mm]	0.93	1.23	0.93	0.00
μ	5.42	6.41	4.08	4.43
Strengt Degradation	< 20%			
Ductility Class	M	H	M	M

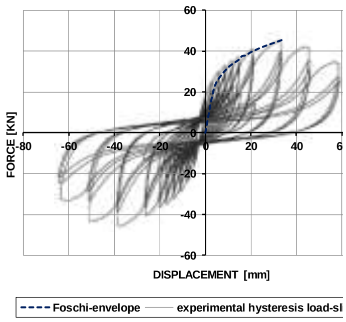
3.a – Stapled wall

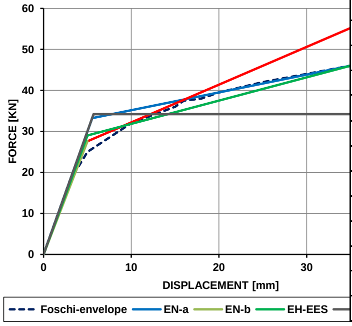




BI-LINEARIZ. METHODS	EN-a	EN-b	EH-EES	EP-EES
F_u [kN]	176.99	-	176.99	120.41
d_u [mm]	80.00			
K_0 [kN/mm]	8.54	-	8.54	8.54
K_{pl} [kN/mm]	1.35	-	1.35	0.00
F_0 [kN]	69.00	-	69.00	120.35
F_y [kN]	81.94	-	93.00	120.41
d_y [mm]	9.59	-	13.50	14.10
α [kN/mm]	8.54	-	6.89	8.54
β [kN/mm]	1.35	-	1.26	0.00
μ	8.35	-	5.98	5.70
Strengt Degradation	< 20%			
Ductility Class	H	-	M	M

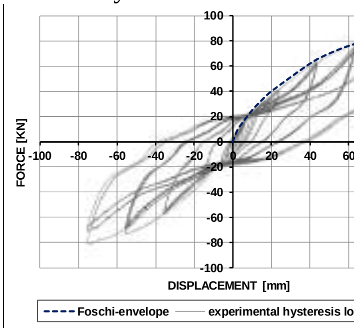
3.b – Light frame timber wall

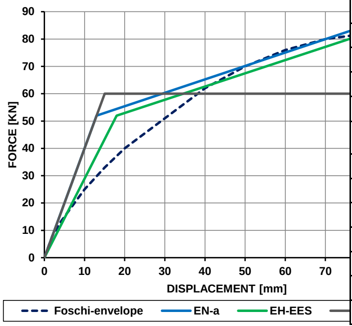




BI-LINEARIZ. METHODS	EN-a	EN-b	EH-EES	EP-EES
F_u [kN]	46.00	55.20	46.00	34.20
d_u [mm]	35.00			
K_0 [kN/mm]	6.00	5.52	6.00	6.00
K_{pl} [kN/mm]	0.43	0.92	0.40	0.00
F_0 [kN]	30.83	23.00	32.00	34.18
F_y [kN]	33.23	27.60	29.00	34.20
d_y [mm]	5.54	5.00	5.10	5.70
α [kN/mm]	6.00	5.52	5.80	6.00
β [kN/mm]	0.43	0.92	0.57	0.00
μ	6.32	7.00	6.86	6.14
Strengt Degradation	< 20%			
Ductility Class	H	H	H	H

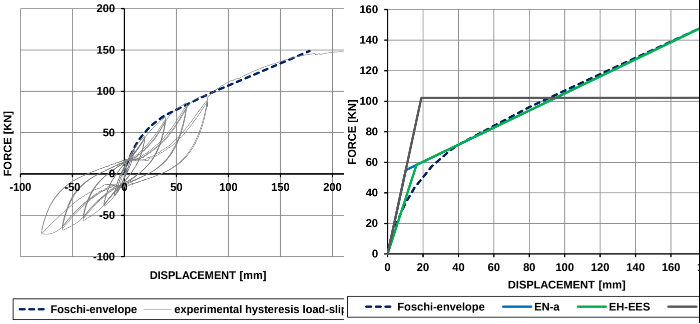
3.c - Heavy frame timber wall





BI-LINEARIZ. METHODS	EN-a	EN-b	EH-EES	EP-EES
F_u [kN]	84.88	-	82.00	60.00
d_u [mm]	80.00			
K_0 [kN/mm]	4.00	-	4.00	4.00
K_{pl} [kN/mm]	0.49	-	0.49	0.00
F_0 [kN]	45.62	-	42.80	60.00
F_y [kN]	52.00	-	52.00	60.00
d_y [mm]	13.00	-	18.00	15.00
α [kN/mm]	4.00	-	2.89	4.00
β [kN/mm]	0.49	-	0.49	0.00
μ	6.15	-	4.44	5.33
Strengt Degradation	< 20%			
Ductility Class	H	-	M	M

3.c – Hybrid wood-concrete wall



BI-LINEARIZ. METHODS	EN-a	EN-b	EH-EES	EP-EES
F_u [kN]	145.70	-	145.70	102.10
d_u [mm]	178.00			
K_0 [kN/mm]	4.60	-	4.60	4.60
K_{pl} [kN/mm]	1.10	-	1.10	0.00
F_0 [kN]	45.00	-	45.00	102.01
F_y [kN]	54.89	-	58.32	102.10
d_y [mm]	10.20	-	16.34	18.98
α [kN/mm]	4.60	-	3.50	4.60
β [kN/mm]	1.10	-	1.10	0.00
μ	17.37	-	10.89	9.40
Strengt Degradation	< 20%			
Ductility Class	H	-	H	H

The usage of different bi-linearization methods defines different yielding conditions for the examined wall configurations. Therefore, there is a differentiation in the elastic branch stiffness values (α) and therefore in the fundamental period of the structure (meant as an assembly of the considered wall-typology). Furthermore, since the ultimate displacement (d_u) is independent from the bi-linearization method, the static ductility values (μ) vary depending on the yielding condition defined for the wall and consequently the associated ductility class. Results show that the criteria based on the Elastic Plastic Strain Energy can be applied independent of the specific shape and nonlinear behaviour of the capacity curve, as well as the EN "a" approach. Otherwise the criteria based on the Elastic Hardening Strain Energy and the EN "b" approach could present some applicability limits due to the imposition of the elastic and hardening stiffness.

3.3. Q-factor estimation

In this section the estimation of the q-factor for the examined building configuration is presented referring to results from different calculation methods, obtained in this work or available in literature.

3.3.1. Effects of bi-linearization methods on the q-factor value

A first evaluation of the behavior factor for the examined wall systems can be done through the experimental method from Eurocode 8 (CEN, 2013), based on ductility classes. Moreover, using experimental results in line with hybrid method, it is possible to have an additional estimate of the q-factor.

Both these methods provide the estimation of the intrinsic q_0-factor because they refer to the yielding condition and disregard the design assumptions. Consequently the q-factor is coincident with the q_0-factor because no code or design overstrengths are accounted.

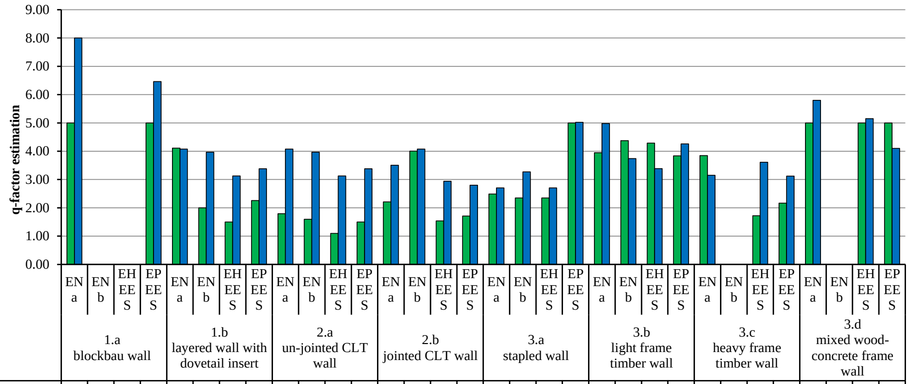
In addition these methods provide a q_0-factor estimation that relies on the type of bi-linearization curves used but represents the intrinsic dissipative capacity of the structure disregarding form design rules. Summarized in Figure 6 are different values for the q-factor, which were obtained with particular bi-linearization curves using the two above methods.

From the graph, it is important to note that the behavior factors obtained with the hybrid methods are higher than the ones estimated with the static ductility approach. Furthermore, there is a good correspondence between the q-factor values from the two methods when the wall-type is rated in a high ductility class.

Obtained results show that for all the case study wall specimens the criteria of the bi-linear curve energy balance (i.e. EH-EES and EP-EES) provides the lower estimation of the q-factor. In addition such criteria provides a more stable estimation of the behaviour q-factor.

Using the criteria based on the EN 12512 (CEN, 2006) the largest q-factor and the greatest variability are provided. Therefore their utilization is not conservative. It appears that the Elastic perfect Plastic bilinear capacity curves provide a more reliable estimation of the nonlinear response and ductility of the building constructive system and therefore of their respective q-factor.

It is finally possible to observe that for the specimens with global behaviour governed by mechanical connections (i.e. 2a, 2.b, 3.b, 3.d) the influence of the different bi-linearization criteria of the q-factor is less relevant than other typologies.



	1.a blockbau wall				1.b layered wall with dovetail insert				2.a un-jointed CLT wall				2.b jointed CLT wall				3.a stapled wall				3.b light frame timber wall				3.c heavy frame timber wall				3.d mixed wood-concrete frame wall			
	EN a	EN b	EH EES	EP EES	EN a	EN b	EH EES	EP EES	EN a	EN b	EH EES	EP EES	EN a	EN b	EH EES	EP EES	EN a	EN b	EH EES	EP EES	EN a	EN b	EH EES	EP EES	EN a	EN b	EH EES	EP EES	EN a	EN b	EH EES	EP EES
QUASI STATIC TEST	5.00	0.0	0.0	5.0	4.1	2.0	1.5	2.3	1.8	1.6	1.1	1.5	2.2	4.0	1.5	1.7	2.5	2.3	2.3	5.0	3.9	4.4	4.3	3.8	3.8	0.0	1.7	2.2	5.0	0.0	5.0	5.0
HYBRID METHOD	8	0.0	0.0	6.5	4.1	4.0	3.1	3.4	4.1	4.0	3.1	3.4	3.5	4.1	2.9	2.8	2.7	3.3	2.7	5.0	5.0	3.7	3.4	4.3	3.2	0.0	3.6	3.1	5.8	0.0	5.2	4.1

Figure 6. Influence of the bi-linearization criteria over the q-factor estimation performed using the method based on the ductility class and the so called "hybrid method"

3.3.2. Summary of q-factor value adopting different calculation methods

It is doable for some constructive systems to compare the q-factor values abovementioned (i.e. obtained with the ductility class-based and hybrid method) with those available in literature deducted by shaking-table tests or non-linear dynamic analysis (NLDA) in terms of minimum, maximum and average values. Clearly the provided literature values are homogeneous not among them, because in some cases they refer to the intrinsic part of the q-factor but in other cases they also consider the effects of code and design overstrengths. Therefore it will be provided for each case which value is reported.

To date, q-factor values for blockbau system type 1.a are not available either by shaking-table tests. Otherwise, the system was numerically investigated by Bedon *et al.* (2015), providing an estimation of the suitable q-factor values.

The values for the "special" system type 1.b and 3.a are only associated to numerical NLDAs results conducted by Pozza *et al.* (2015-b). These analyses refer to a specific design of the structure aimed to minimize the overstrength effects. Consequently the q-factor estimations are representative of the intrinsic dissipative behaviour of the structural system.

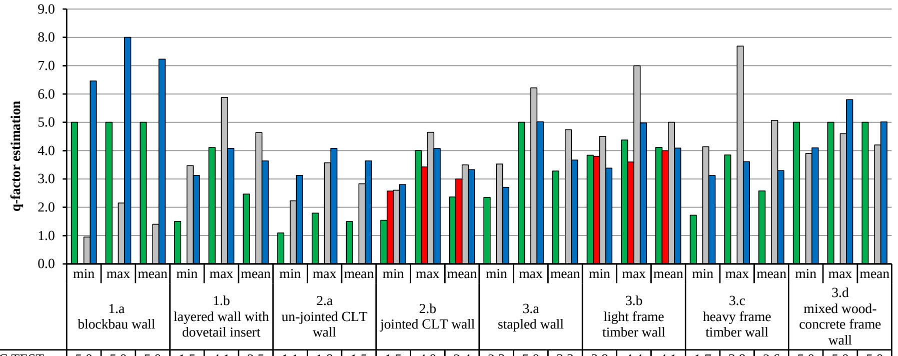
Test results of CLT wall type 2.a and 2.b are referred to a three-story building shaking-table test supervised by Ceccotti (2008) and numerical analysis reported by Pozza and Scotta (2014). Numerical simulations refer to the same three story building tested on shaking table using NLDAs performed with seven different earthquake signal. Two assembly configuration were numerically analyzed referring to the type 2.a and 2.b respectively. These estimations of q-factor are code dependent and provide the entire q-value accounting for both intrinsic structural dissipative capacity and overstrength.

The behavior factor for light-frame system type 3.b has been widely analyzed by Ceccotti and Karacabeyli (2002) through numerical analysis. In addition, shaking-table test results are available from the research conducted by Tomasi *et al*., (2015). These results are code dependent therefore the intrinsic behaviour factor and the overstrength are given together.

The behavior factor for heavy-frame system type 3.c has been analyzed by Terzi (2010) adopting numerical simulation and NLDAs using four different earthquake signal. Obtained results are code dependent accounting simultaneously for both the intrinsic and the overstrength quote of the q-factor.

Ultimately, the q-factor estimation for the hybrid wood-concrete system is appertained to numerical results carried out by Pozza *et al.* (2015-c). In this analyses the intrinsic q_0 factor is provided.

A summary for the q-factor values for the different construction systems is reported in Figure 7. The results are obtained for both numerical analysis method and shaking-table test in accordance with the accelerations approach.



	1.a blockbau wall			1.b layered wall with dovetail insert			2.a un-jointed CLT wall			2.b jointed CLT wall			3.a stapled wall			3.b light frame timber wall			3.c heavy frame timber wall			3.d mixed wood-concrete frame wall		
	min	max	mean	min	max	mean	min	max	mean	min	max	mean	min	max	mean	min	max	mean	min	max	mean	min	max	mean
QUASI STATIC TEST	5.0	5.0	5.0	1.5	4.1	2.5	1.1	1.8	1.5	1.5	4.0	2.4	2.3	5.0	3.3	3.8	4.4	4.1	1.7	3.8	2.6	5.0	5.0	5.0
SHAKING TABLE TEST										2.6	3.4	3.0				3.8	3.6	4.0						
NUMERICAL METHOD	1.0	2.2	1.4	3.5	5.9	4.6	2.2	3.6	2.8	2.6	4.7	3.5	3.5	6.2	4.7	4.5	7.0	5.0	4.1	7.7	5.1	3.9	4.6	4.2
HYBRID METHOD	6.5	8.0	7.2	3.1	4.1	3.6	3.1	4.1	3.6	2.8	4.1	3.3	2.7	5.0	3.7	3.4	5.0	4.1	3.1	3.6	3.3	4.1	5.8	5.0

Figure 7. q factor estimation for the examined wall systems obtained adopting the four calculation methods

Some specific comments and remarks about the q-factor estimations provided with the different methods for each examined wall specimens are listed below.

First of all it is possible to observe that the q-factors provided by numerical analyses show the higher variability, among the same wall specimens, in cause of the effect of earthquake frequency content in the global response of the building systems. Other methods for the q-factor estimation result more stable and with reduced variability.

The specimen 1.a (blockbau wall) is characterized by large displacement capacity with a specific rigid-plastic behavior due to the friction effects between overlapped crosspieces. Consequently the q-factors evaluated adopting the method based on the wall ductility result very high. On the contrary the q-factor estimation provided by numerical simulations are significantly lower because the adopted near collapse criterion does not exploit all the displacement capacity (i.e ductility) of the systems. Such numerical estimations are aligned with values provided by Eurocode 8 (CEN, 2013) where this system is classified being low dissipative system and must be design elastically (q=1.5). Otherwise the q-factor estimations provided by the ductility methods contrast with the code provision as results are very high because of the specific rigid-plastic behaviour and of large failure displacements. This discrepancy between the code provisions and the experimental evidence highlight that one hand the dissipative capacity due to the friction effects can be relevant but on the other hand, that the q-factor estimation based on the ductility ratio overestimates the actual behaviour factor for this construction type.

Relative to type 1.b, it is possible to observe an influence of the method to calculating the q-factor, in particular the numeric method gives higher values than the ductility class-based method. In spite of the limited number of mechanical connections, the obtained behavior factor is usually higher and in contrast with Eurocode 8 (CEN, 2013) design guidelines for the wood carpentry-joint system. The bi-linearization EN-a method appears to be the more consistent with the normative prescriptions to calculate the q-factor value.

For specimens 2.a and 2.b (CLT wall system) the influence of the methods used to define the q-factor is not relevant. The distinct methods are stable and provide consistent estimates of the behavior factor. Comparing the two wall typologies 2.a and 2.b (i.e. un-jointed and jointed CLT wall panel), it follows that the number of connections used to assemble the CLT panels is substantial. A q-factor of 2 results proper for the wall with un-jointed panels, while a q-factor of 3 for the jointed CLT wall panels. This result proves to be in accordance with the parametric analysis results related by Pozza (2013). In this case the normative guidelines turn out to be extremely precautionary and eligible just for structures composed by un-jointed panels.

The results obtained for the wall 3.a (i.e. stapled wall panel) show that this typology is characterized by a dissipative capacity value between a rigid panel system, as CLT, and a light-frame system. In this case, the dissipative capacity of the base-connections is compounded by the panel as compared to the frame system. Therefore it is possible, from the design perspective, to assimilate this typology with the light-frame system. A correspondent intrinsic behavior q_0-factor of 4 is associated in accordance with Pozza *et al.* (2015-b).

The q-factor values obtained for the light-frame system (wall 3.b) with different methods, result consistent, and in accordance with the normative design guidelines (CEN, 2013) which suggest a value up to 5. The maximum q-factor values are attained with numerical method while the other methods provide lower values because do not account for the design and code overstrengths. Based on available results it seem that a q-factor equal to 5 and an intrinsic q_0-factor equal to 4 sound adequate for this building system.

The heavy-frame system (wall 3.c) results less dissipative compared to the light-frame (wall 3.a) especially referring to experimental and hybrid methods that provide the intrinsic dissipative capacity of the specimens. Design of this system can be performed adopting a q-factor equal to 4 and an intrinsic q_0-factor equal to 3.

Finally the hybrid wood-concrete system (wall 3.d) presents a high ductility and dissipative capacity. The wood-concrete system is comparable to the light-frame, therefore a q-factor of 5 and a q_0-factor of 4 is applicable.

5. Conclusions

Heretofore, it is been given a preliminary definition of the behavior factor and its importance designing wood construction systems. The q-factor factorization into the quote representing the intrinsic dissipative capacity of the structure (q_0) and the overstrength due to design and code assumption (Ω) is given. Several procedures calculating the q-factor have been described and analyzed. These techniques are based on numerical and analytical approaches, and for each one advantages and limitations are highlighted.

Behaviour factor values applicable to different building systems are determined and several variables influencing construction are examined. The necessary experimental tests and analytical interpretation for estimating the behaviour factors for these wall systems, characterized as they are by several types of layout, were performed.

The q-factor variability for different systems, using several bi-linearization methods for the experimental curve, is been analyzed. Moreover, it is been noticed that not all bi-linearization methods are always appropriate, the most suitable procedure should be choose case by case.

A research of the values available in literature for the behavior factor is been conducted. Several constructive typologies has been considered and referred both to shaking table tests and NLDAs analysis results. Obtained results show that:

- Wood constructive systems can be classified in three categories: (1) linear elements systems assembled by carpentry joints, (2) rigid wall panel systems cobbled together with mechanical connections, (3) deformable wall panel system (i.e. frame and special type).
- Normative guidelines result to be quite precautionary for the wood linear elements systems, while in good agreement for the rigid wall panel and wood frame systems.

Results enlightened is this work, even though representative for the examined case studies, present limitations due to the use of not homogeneous data coming from both experimental tests and numerical analysis. A more detailed analysis should be carried out to investigate the effect of design assumptions on the final behavior factor value, and so defining the intrinsic q_0-factor value and the considered over-strength component in the design.

Either way the information given in this paper may be used to estimate the behavior factor of whole timber buildings realized with traditional and modern constructive technologies.

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