Capacity design of typical earthquake-resistant connections for CLT structures

Davide Trutalli[a], Luca Marchi[a], Luca Pozza[b], Roberto Scotta[a]

[a] *Dipartimento di Ingegneria Civile, Edile e Ambientale, Università degli Studi di Padova, Via Marzolo 9, 35131 Padova*

[b] *Dipartimento di Ingegneria Civile, Chimica, Ambientale e dei Materiali, Università di Bologna, Viale Risorgimento 2, 40136 Bologna*

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ABSTRACT

Earthquake-resistant CLT structures can be designed in accordance with the concept of non-dissipative or dissipative structural behaviour. In the second case, modern Italian and European seismic codes require the compliance with the capacity design, to assure the development of plastic deformations in the dissipative components before failure of the non-dissipative ones. For timber structures, this requirement is crucial, because no regions outside the defined non-linear zones, i.e., steel connections, exhibit dissipative capacity. The load-bearing capacity of connections with dowel-type fasteners is currently estimated according to conservative design methods for static actions, which do not take advantage of their entire strength in the seismic design. Such underestimation of their actual capacity, the high scattering of their mechanical parameters demonstrated by experimental tests and the use of different partial factors for material properties for each component of the same connection, introduce indecisions in the capacity design and in the overstrength factors to be used. Additionally, the recent diffusion of multi-storey CLT buildings in earthquake-prone areas like Italy, makes these issues particularly relevant. In this work, the concept of capacity design and its application in the seismic deign of CLT buildings are discussed, also in terms of suitable values of overstrength factors for this structural system. Moreover, an overview of capacity-design rules for timber structures introduced by the latest reviews of Italian and European Codes and Standards is given. Finally, an applicative example of capacity design of a typical connection for CLT buildings and main outcomes are analysed.

1 INTRODUCTION

1.1 Earthquake-resistant connections for CLT structures

The seismic response of cross-laminated timber (CLT) structures is mainly attributable to the non-linear behaviour of ductile connections between panels and to foundation, since CLT panels have a rigid and elastic behaviour in their plane.

Connections between adjacent panels to transfer in-plane shear (vertical construction joints and horizontal diaphragm joints) are usually realized with self-tapping screws (STS) in different ways: half-lap joints, spline joints with laminated veneer lumber (LVL) or steel splines and butt joints with crossed inclined screws (Gavric et al. 2015a; Loss et al. 2018; Sullivan et al. 2018). These connections normally show a ductile behaviour, provided that fastener slenderness assures the development of failure modes with plastic hinges, as now clearly required by the latest update of the Italian Building Code (MIT 2018).

Connections used at foundation and in-between storeys are generally angle brackets and hold-downs, manufactured to prevent respectively sliding and rocking of the shear walls. These connections are normally made of punched and cold-formed steel plates fastened to the panel with ring shank nails or STS. Ductility and energy dissipation capacities are entirely assigned to fasteners (ductile component of the connection), whereas the steel plates and concrete anchoring should be over-resistant (brittle components of the connection). The use of 4-mm diameter ring shank nails or 5-mm diameter STS has already demonstrated to confer good strength and ductility to the connection (Ceccotti 2008; Ceccotti et al. 2013; Gavric et al. 2015b, c; Izzi et al. 2016; Pozza et al. 2018a, b; O'Ceallaigh and Harte 2019). Nevertheless, many experimental evidences showed also events of brittle failures due to excessive nailing, leading to an unexpected exceedance of the actual strength of the steel plate or timber elements (Popovski et al. 2010; Gavric et al. 2013; Tomasi and Sartori 2013; Piazza et al. 2015; Izzi et al. 2018b, a). The main reasons that may explain such excessive fastenings are attributable to:

1. The underestimation of their actual strength and the consequent overdesign, applying in the seismic design conservative methods for static actions, which do not take advantage of their entire strength;
2. The high scattering of their mechanical properties and the consequent large gap between the lower and upper characteristic load-bearing capacity, i.e., 5th and 95th percentile of the strength;
3. The use of different partial factors for material properties for each component of the same connection, i.e., conservative values for fasteners and low values for steel elements, leading to another source of overdesign of the fastening.

Actually, hold-downs and angle brackets were derived from the light-frame system, for which energy dissipation capacity is assured by in-plane deformation of the wall and consequent shear deformation of the small-diameter fasteners, connecting the bracing panels to the light frame. Therefore, they were not originally conceived to be particularly strong and ductile. The use of these *first-generation connections* in mid-rise CLT structures in seismic-prone areas has required geometrical and mechanical improvements, leading to a *second generation of connections* with greater strength and robustness, increasing thickness, steel grade, number of nails and adding new stiffeners to avoid local deformations. Examples are in (Ceccotti et al. 2013; Tomasi and Smith 2015; Polastri and Pozza 2016; Izzi et al. 2018b; Polastri et al. 2019). Despite the substantial increase of strength and stiffness, the ductile mechanism has remained unchanged. *Third-generation connections* are therefore being developed with the aim to optimize also ductility and dissipative capacity, relocating the dissipative mechanism from the shear deformation of the fasteners to the plastic deformation of specifically-designed steel elements, which, in some cases, are conceived to be also replaceable after the earthquake. Some prototypes are available in the literature (Baird et al. 2014; Loo et al. 2014; Latour and Rizzano 2015; Scotta et al. 2016, 2019; Hashemi et al. 2017; Schmidt and Blass 2017; Polastri et al. 2017; Blomgren et al. 2018; van de Lindt et al. 2019). Most of them exploit the hysteretic behaviour of steel or friction, keeping elastic the fastening to the panel, with a resulting reduced wood embedment and pinching effect. Therefore, they work in an opposite way with respect to the *first-* and *second-generation connections*. The relocation of dissipation in an element (fuse) with a more reliable and predictable behaviour – besides the strong increase of dissipation – allows also a better control of the actual strength and failure of the connection, thanks to the well-predictable yielding and peak forces of steel and low scattering of strength. The excessive fastening, in this case, reduces the risk of brittle failures.

1.2 Capacity design of CLT structures

Independently from the type of connection, clear and comprehensive capacity-design rules are needed, which should take into account all the aforementioned features of connections for CLT structures. The higher complexity in estimating the shear strength of fasteners in CLT panels (Brandner et al. 2016; Ringhofer et al. 2018) than in glulam or solid wood, makes these issues even more relevant.

The capacity design approach, originally developed by Paulay and Priestley (1992) has been already defined and applied to CLT structures (Fragiacomo et al. 2011; Jorissen and Fragiacomo 2011; Gavric et al. 2013; Scotta et al. 2017; Casagrande et al. 2019; Trutalli et al. 2019). It can be applied at three levels: connection, wall and building level (Gavric et al. 2013). At the connection level, plasticization of the ductile element should be ensured; at the wall and building level, selected ductile connections should yield before others. Moreover, at the building level, a *box behaviour* of the building should be assured. The analytical approach for capacity design at wall and building level for CLT structures is available in (Casagrande et al. 2019), to ensure that yielding of all vertical joints occurs before yielding of the other connections. The consequent increase of global dissipative capacity obtained with the fragmentation of the façades into narrow modular CLT panels can be significant (Pozza and Trutalli 2017).

1.3 Aim of this work

This work deals with capacity design at connection level with close reference to *first-* and *second-generation connections* for CLT structures, discussing the simplified conceptual model and main theoretical formulations. Overstrength factors according to experimental evidences in the literature are listed. Then, the capacity-design provisions from the latest reviews of Italian and European Codes and Standards are summarized. Finally, a case study of capacity design of a typical connection for CLT structures is presented and results obtained from the application of different provisions are compared.

2 THEORETICAL MODEL OF CAPACITY DESIGN

In this Section, the simplified conceptual model of capacity design and the definition of overstrength factor to be used in the design of the brittle components of a connection, based on strength properties of its ductile part, are discussed.

2.1 Conceptual model

The application of capacity design is based on the fulfilment of inequality (1)

$$F^{-}_{B,\,code} \geq F^{+}_{D,\,peak} = \gamma_{\mathrm{Rd}} \cdot F^{-}_{D,\,code} \tag{1}$$

where F^{-}_{code} is the characteristic load-bearing capacity estimated according to a Code or design rule, F^{+}_{peak} is the 95th percentile of the peak strength obtained by experimentation, subscripts *B* and *D* identify brittle and ductile elements respectively, γ_{Rd} is the overstrength factor.

Inequality (1) assures for all the brittle components of the connection system a 5th-percentile load-bearing capacity higher or equal to the 95th-percentile peak strength of the ductile part. Since $F^{+}_{D,\,peak}$ is normally unknown, it can be estimated from the product between the 5th-percentile load-bearing capacity of the ductile element – which can be analytically evaluated according to a particular Code – and the overstrength factor given by the same Code.

Figure 1 shows the conceptual model of capacity design of the weakest brittle component, starting from the strength properties of the ductile element. Other parameters appear in the Figure:

- d_y is the yielding displacement;
- d_{peak} is the displacement corresponding to peak strength;
- F^{-}_{peak} is the 5th percentile of the maximum strength obtained by tests;
- F^{mean}_{peak} is the mean value of the maximum strength obtained by tests;
- F^{-}_{y} is the 5th percentile of the yielding strength obtained by tests;
- F^{mean}_{y} is the mean value of the yielding strength obtained by tests;
- F^{+}_{y} is the 95th percentile of the yielding strength obtained by tests.

In this approach, the definition of overstrength for the ductile element is based on two components: the experimental scattering of the peak strength and the underestimation of such strength applying conservative analytical formulations according to a particular Code or Standard (analytical overstrength). Therefore, the overstrength factor γ_{Rd} can be defined directly as a unique term, according to Equation (2), or split into two parts as in Equation (3):

$$\gamma_{\mathrm{Rd}} = \frac{F^{+}_{D,\,peak}}{F^{-}_{D,\,code}} \tag{2}$$

$$\gamma_{\mathrm{Rd}} = \gamma_{\mathrm{sc}} \cdot \gamma_{\mathrm{an}} = \frac{F^{+}_{D,\,peak}}{F^{-}_{D,\,peak}} \cdot \frac{F^{-}_{D,\,peak}}{F^{-}_{D,\,code}} \tag{3}$$

where γ_{sc} is the scattering of peak strength and γ_{an} is the analytical overstrength.

It is worth emphasizing that the example provided in Figure 1 refers to a connection characterized by hardening behaviour. The fulfilment of inequality (1) assures – with a fixed probability according to ultimate-limit-state (ULS) approach – that the full ductility of the connection is exploited and a ductile failure is reached.

However, when ductile elements allow for large displacement capacity but this does not comply with the admissible drifts of the structure, such ductility can be exploited only partially. In this case, the peak point (d_{peak}, F_{peak}) of the force-displacement curve in Figure 1 has to be replaced with a target point (d_{target}, F_{target}) positioned between d_y and d_{peak}. In this way, the connection will be able to attain the predefined target ductility before failure of the brittle element, reducing the overstrength value and improving the cost-effectiveness of the connection.

As a final remark, it should be noted that in inequality (1) the characteristic values of the load-bearing capacities have been compared. In an even more conservative approach, the design values of the bearing capacities could be compared, and the partial factors for material properties γ_m should be introduced: $\gamma_{B,m}$ reducing the strength of brittle components and $\gamma_{D,m}$ increasing the strength of ductile one. With such implementation, overstrength factor would be further increased by the factor $\gamma_{D,m} \cdot \gamma_{B,m}$.

It needs to be stressed that $\gamma_{D,m}$ should be greater when dowel-type fasteners behave as ductile component of the connection instead of the steel part.

The full conceptual model of capacity design – including also the target displacement and the partial factor for material properties – and its application to *third-generation connections* are available in (Trutalli et al. 2019).

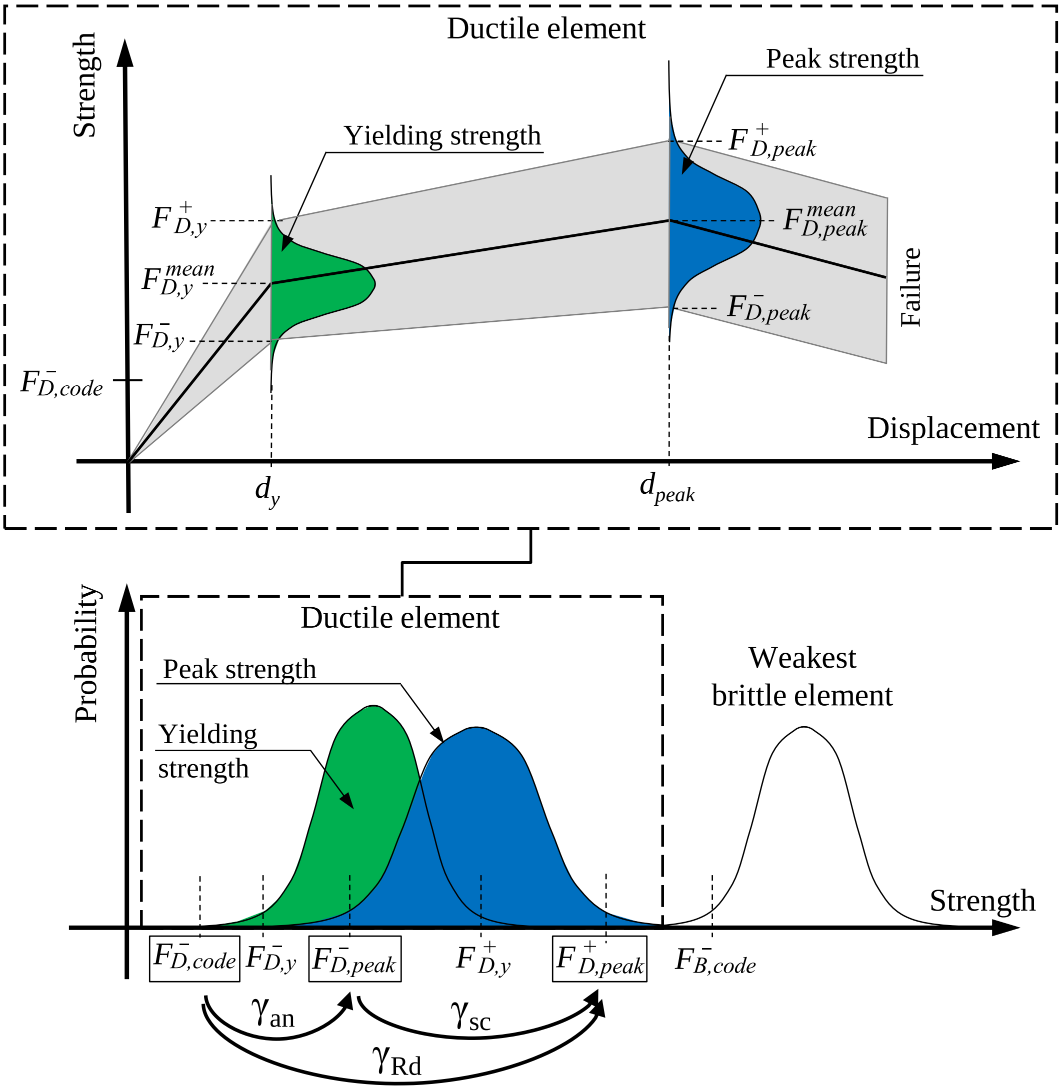

Figure 1. Conceptual model of capacity design applied to a ductile connection

Looking at Figure 1, other remarks can be added.

While the scattering of peak strength γ_{sc} is experimentally assessable, the determination of the analytical overstrength γ_{an} is code-dependent being strictly correlated to the analytical method used to compute $F^-_{D,\ code}$, which is actually the only value that is provided by national building Codes or European Technical Approvals (ETAs). This aspect is of utmost importance for connections for timber structures, and specifically for CLT, for which $F^-_{D,\ code}$ is currently not univocally defined, depending on the chosen values of parameters in the calculation model. For example, the shear resistance of a dowel-type fastener is normally computed according to Eurocode 5 (CEN 2014), applying the European Yield Model, but the resulting load-bearing capacity is not univocal, depending on the chosen values of parameters in the analytical formulations and on the specific rules provided by product approvals.

Therefore, the reliability of γ_{Rd} can be affected not only by the statistical variability of the strength of the ductile element but also by the analytical method to estimate its characteristic strength, according to a specific standard or ETA. From this, it is straightforward that γ_{Rd} values proposed in a Code has to be consistent with the analytical methods and parameters provided by the Code itself.

Furthermore, as anticipated in the introduction, considering that the load-bearing capacity of dowel-type fasteners is currently estimated according to conservative design methods for static actions, which do not take advantage of their entire strength in the seismic design, high γ_{an} are expected when they are used as ductile elements of the connections.

2.2 Overstrength factors

Table 1 summarizes overstrength factors of typical connections for CLT structures based on the conceptual model described in the previous Subsection and supported by experimental tests.

(Gavric et al. 2015b) evaluated the overstrength factors from tests in shear or tension of angle brackets and hold-downs anchored to CLT floors or to foundation. Values of γ_{sc} and γ_{an} are given and γ_{Rd} can be obtained from their product, resulting in values in the range between about 2.0 and 3.4. These values may be useful to apply a capacity design at wall level.

Another study conducted by (Gavric et al. 2015a) focused on the determination of the cyclic behaviour of typical screwed connections and γ_{sc} values in the range 1.22 – 1.95 were obtained (considering only specimens showing a ductile failure). Also in this case, the total γ_{Rd} can be extrapolated from $F^-_{D,\,code}$ according to Eurocode 5 (CEN 2014), resulting in a range 1.2 – 2.1.

An experimental research about steel-to-timber joints with ring shank nails for CLT is available in (Izzi et al., 2016). According to these tests and depending on the chosen parameters to compute $F^-_{D,\,code}$ and on the angle of the force to the face lamination of the panel, the obtained γ_{Rd} values are in the range between 1.6 and 2.6, thus demonstrating the strict correlation between γ_{Rd} and the analytical models and parameters to compute $F^-_{D,\,code}$. These values may be used to apply the capacity design at connection level, to design the steel plate of the connection or the anchoring to foundation or floor.

The overstrength factor was evaluated also for steel-to-timber connections with dowels (Ottenhaus et al. 2018). In this case, γ_{Rd} was firstly theoretically estimated and then experimentally verified and split into different sources of overstrength.

A recent comprehensive study evaluated the performance of steel-timber joints with 5-mm diameter STS (O'Ceallaigh and Harte 2019). Values of γ_{Rd} between 2.1 and 2.5 were recommended.

Table 1. Comparison of overstrength factors derived for some typical connections for CLT structures

Connector/Fastener	γ_{sc}	γ_{an}	$\boldsymbol{\gamma_{Rd}}$
Nails loaded parallel to face lamination[(a)]	1.27	1.61	**2.04**
Nails loaded perpendicular to face lamination[(a)]	1.53	1.69	**2.59**
5×50 STS loaded parallel to face lamination[(b)]	1.71	1.39	**2.38**
5×75 STS loaded parallel to face lamination[(b)]	1.39	1.80	**2.50**
5×50 STS loaded perpendicular to face lamination[(b)]	1.87	1.27	**2.37**
5×75 STS loaded perpendicular to face lamination[(b)]	1.60	1.67	**2.67**
Dowels[(c)]	1.51	1.29	**1.95**
Panel-to-panel joints with screws (half-lap joint) [(d)]	1.88	0.95	**1.79**
Panel-to-panel joints with screws (LVL joint) [(d)]	1.52	1.37	**2.08**
Hold-down in tension[(e)]	1.30	2.60	**3.38**
Hold-down in shear[(e)]	1.38	-	**-**
Angle bracket in tension[(e)]	1.23	2.80	**3.44**
Angle bracket in shear[(e)]	1.16	1.70	**1.97**

[(a)] From (Izzi et al. 2016)
[(b)] From (O'Ceallaigh and Harte 2019)
[(c)] From (Ottenhaus et al. 2018)
[(d)] From (Gavric et al. 2015a)
[(e)] From (Gavric et al. 2015b)

3 OVERVIEW ON ITALIAN AND EUROPEAN CODES AND STANDARDS

In this Section, main seismic design rules according to current and forthcoming reviews of Italian and European Codes and Standards are discussed, comparing the draft of the revision of chapter 8 of the European Seismic Code (EN 1998-1, Eurocode 8), the Italian Building Code and Commentary (MIT 2018, 2019) and the CNR-DT 206 R1/2018 (CNR 2018). It is worth noting that for all of them important updates and clarifications have been introduced, in particular with reference to capacity design and overstrength factors.

3.1 *Draft of chapter 8 of Eurocode 8*

In the ongoing revision of chapter 8 of Eurocode 8 (Follesa et al. 2018), CLT structures are allowed to be designed either in ductility class medium (DCM) or high (DCH), provided that connections *shall be able to deform plastically for at least three fully reversed cycles at a static ductility ratio* at least equal to 3 for DCM and 4 for DCH *without more than 20% reduction of their resistance between the first and the third cycle backbone curve.*

Alternatively, new specific requirements have been introduced about ductile failure modes of shear-loaded fasteners according to the Johansen's Theory, and considering axially-loaded fasteners as non-dissipative. Moreover, capacity design for CLT systems belonging to DCH requires that *vertical step joints between wall panels in segmented shear walls* and *shear-restrain and uplift-restrain connections* are designed as dissipative components, i.e., capacity design is performed both at connection and wall level.

This draft recommends an overstrength factor equal to 1.3 for CLT buildings, independently of their ductility class, except for vertical cantilever systems made with structurally continuous CLT wall elements, for which it is to be assumed equal to 1.6.

The draft also includes the possibility of locating the dissipative zones in *purposely developed energy dissipators (e.g., lead extruded or hydraulic dampers, dog-bone steel plates, etc.)*, introducing the concept of third-generation connections. In this case, no specific overstrength factors are given; it can be understood that ad-hoc experimentation is needed.

3.2 *Italian Building Code (NTC 2018) and Commentary 2019*

The revised version of the Italian Building Code NTC 2018 (MIT 2018) – issued in January 2018 – classifies CLT structures only into CD "B" (the same as DCM in the Eurocode), independently from the arrangement and dimensions of the panels and the application of capacity design at wall level. Such classification has been confirmed in the recently-issued Commentary to the NTC 2018 (MIT 2019).

Differently from the draft of the European Code, the overstrength factors for timber structures in NTC 2018 are not assigned to a specific structural type, but are assigned as 1.3 for CD "B" and 1.6 for CD "A" (the same as DCH in the Eurocode). As only CD "B" is considered for CLT buildings, it can be understood that only the overstrength factor 1.3 is to be used.

3.3 *CNR-DT 206 R1/2018*

The Standard CNR-DT 206 R1/2018 (CNR 2018) – issued in April 2018 – has been recognized as very useful guideline for designers of timber structures since its first issue in 2007.

In the latest review, CLT structures are classified into both CD "B" and CD "A". To belong to CD "A", requirements about ductile design of connections and vertical joints and geometry of wall panels in segmented walls in terms of width-to-height ratio are to be fulfilled.

An overstrength factor equal to 1.3 or 1.1 is suggested for CD "A" or CD "B" respectively, with the exception of vertical continuous cantilevers, for which it is equal respectively to 1.6 or 1.4.

As the European Code, this Standard considers the possibility that traditional connection systems are replaced by alternative connections, in which the dissipation is located in specific devices (generally metal devices). In this case, a more conservative value of γ_{Rd} equal to at least 1.5 is required and theoretical and experimental analyses are suggested to determine the actual local and global level of ductility.

3.4 *Comparison and discussion*

A brief comparison among the current and forthcoming reviews of European and Italian Codes and Standards is given in Table 2 and Table 3 in terms of behaviour factor values and overstrength factors for CLT buildings.

With reference to the behaviour factor value, it can be noted that the Italian Code (MIT 2018) assigned a single value, allowing only the design in CD "B". This value is intermediate between the values assigned for the two ductility classes in the draft of chapter 8 of Eurocode 8 (Follesa et al. 2018) and in the CNR-DT 206 R1 (CNR 2018). These two Standards provide same values of behaviour factor and similar requirements for the high-ductility class, with more detailed geometric limitations in the CNR Standard.

With reference to the overstrength factor, a single value is given in the Italian Code and in the draft of Eurocode 8, whereas CNR Standard diversifies it, depending on the chosen ductility class. Despite these small differences, they substantially conform to the same value of γ_{Rd} equal to 1.3 for the CLT building technology. This overstrength factor is anyway not consistent with values listed in Table 1, which have been obtained according to the conceptual model in Section 2 and experimental tests of different types of fasteners. The value of γ_{Rd} equal to 1.6 assigned by the Italian Code to timber structures in CD "A" seems a more appropriate value for CLT structures, even if considered only in CD "B".

Finally, the γ_{Rd} equal to 1.5 suggested by CNR Standard for connections with specific dissipative devices sounds controversial, since these elements are generally characterized by lower scattering and a more reliable definition of strength and failure than fasteners.

Table 2. Behaviour factors for CLT buildings

Ductility Class	Draft Eurocode 8*	NTC 2018	CNR-DT 206 R1
DCM (CD "B")	2.0	2.5	2.0
DCH (CD "A")	3.0	-	3.0

* *Other values for vertical continuous cantilevers*

Table 3. Overstrength factors for CLT buildings

Ductility Class	Draft Eurocode 8	NTC 2018	CNR-DT 206 R1
DCM (CD "B")	1.3 (1.6)	1.3	1.1 (1.4)
DCH (CD "A")	1.3 (1.6)	-	1.3 (1.6)
Dissipators	N/D	-	1.5

In brackets the values for vertical continuous cantilevers

4 CASE STUDY

In this Section, the capacity design has been applied to a typical connection for CLT structures. The results obtained applying the theoretical formulations in Section 2 or the overstrength values given by the design Codes in Section 3 have been compared. The connection is a hold-down working in tension, fastened to the CLT panel with 4×60-mm ring shank nails. The main mechanical and geometrical properties of fasteners and CLT have been taken from (Izzi et al. 2016), in which a minimum load-bearing capacity $F^{-}_{D,\,code}$ of a single nail equal to 2.16 kN was evaluated, in compliance with Eurocode 5. Assuming the use of a number of nails n equal to eighteen, adequately spaced in order to obtain:

$$n = n_{ef} \tag{4}$$

where n_{ef} is the effective number of fasteners, it is possible to determine the total load-bearing capacity of the fasteners, i.e., the ductile component of the hold-down:

$$F^{-}_{D,\,code} = 18 \cdot 2.16 = 38.9 \text{ kN} \tag{5}$$

This value must be multiplied by the proper overstrength factor in order to determine the minimum load-bearing capacity of any brittle element in the chain, see inequality (1).

Adopting the value γ_{Rd} = 2.04 evaluated experimentally by (Izzi et al. 2016) and reported in Table 1 for nails loaded parallel to face lamination, the minimum load-bearing capacity $F^{-}_{B,\,code}$ of the brittle element should be 79.3 kN. This value can be adopted for the verification of the steel plate in tension, evaluating the minimum cross-section and steel strength that verify Equation (6) according to Eurocode 3 (CEN 2015):

$$F^{-}_{B,\,code} = \min\left(A \cdot f_y;\, 0.9 \cdot A_{net} \cdot f_u\right) \tag{6}$$

where A and A_{net} are respectively the gross and net cross-section of the metal plate; f_y and f_u are the yielding and ultimate strength of steel. Assuming a typical pattern of holes in the plate with the alternation of two and three 5-mm holes per row, the resulting steel plate could have a gross cross-section equal to 60×4 mm with a steel class S355, obtaining $F^{-}_{B,\,code}$ = 82.6 kN > 79.3 kN, thus verifying the capacity design. It has to be recalled that the same check must be applied also to all the other brittle components of the connection, i.e., concrete anchor, punching of the base plate, etc.

Applying the γ_{Rd} values assigned by the Italian Code (MIT 2018) and the draft of revision of chapter 8 of Eurocode 8 (Follesa et al. 2018), the minimum required load-bearing capacity for the brittle elements decreases to 50.5 kN. In this case, a gross cross-section equal to 60×3 mm is sufficient for the steel plate. Finally, according to CD "B" class in CNR-DT 206 R1/2018 (CNR 2018) the minimum load-bearing capacity of the brittle elements could be further reduced to 42.8 kN. In this case, a cross-section of 60×3 mm and steel class S275 are suitable.

Table 4 summarizes the verification of the steel plate using 4×60-mm nails and lists also the results using 5×50-mm and 5×75-mm STS.

Table 4. Capacity design of the steel plate of a typical hold-down for different γ_{Rd} values

Fasteners	γ_{Rd} (-)	Steel class	Cross-section (mm)	Min $F^{-}_{B,\,code}$ (kN)	Actual $F^{-}_{B,\,code}$ (kN)
18	2.04	S355	60×4	79.3	82.6
nails	1.30	S355	60×3	50.5	62.0
4×60	1.10	S275	60×3	42.8	49.5
18	2.38	S355	70×4	90.0	99.4
screws	1.30	S275	60×3	49.1	49.5
5×50	1.10	S235	60×3	41.6	42.3
18	2.50	S355	80×4	109.4	113.6
screws	1.30	S355	60×3	56.9	62.0
5×75	1.10	S275	60×3	48.1	49.5

5 CONCLUSIONS

A simplified conceptual model of capacity design has been described and overstrength factors for typical connections for CLT structures, evaluated according to the theoretical model and current and forthcoming reviews of Italian and European Codes and Standards, have been compared. Finally, a case study of capacity design of the steel plate of a typical hold-down has been presented. The main outcome of this work is that the brittle components of a connection should be verified according to capacity design, applying overstrength factors consistent with the experimental scattering and the design strength of the ductile component. According to experimental evidences in the literature, overstrength values higher or equal than 2.0 could be considered suitable for the dowel-type fasteners generally used as ductile components of typical connections for CLT structures. This value theoretically would permit the ductile component to deform plastically up to reach its peak strength and therefore up to its failure, exploiting its full ductility, before the brittle component reaches its characteristic load-bearing capacity.

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