CAPACITY DESIGN APPROACH FOR MULTI-STOREY TIMBER-FRAME BUILDINGS

D. Casagrande, T. Sartori & R. Tomasi

Department of Civil, Environmental and Mechanical Engineering, University of Trento, Italy

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1 INTRODUCTION

The traditional seismic design of a structure adopts a force-based method. The capacity of the structure to support the seismic action is usually obtained by dissipating the seismic energy via its structural damaging and hence assuming a nonlinear structural behaviour. Seismic linear analyses are usually carried out, dividing the elastic seismic forces by the behaviour factor q, depending on the global structural ductility. The global behaviour of a structure, and in particular its ductility, strongly depends both on the mechanical properties of structural components and on the global failure mechanism. For this reason, in order to achieve high values of q, local and global brittle failure mechanisms should be prevented.

In timber structures the energy dissipation cannot be achieved in timber elements because, as known, they are characterized by a brittle failure. Hence the capability of the structure to dissipate the seismic energy is obtained from the yielding of the mechanical connection devices, which should be designed so that their local ductility is consistent with the global ductility of the structure assumed in the design phase, and represented by the behaviour factor q.

Regarding timber frame buildings a high upper limit value of the behaviour factor q is usually suggested by Standards (i.e. equal to 5 for European Standards for seismic design [1]), setting this structural type in the high ductility class (DCH). The main reason is that the a great amount of fasteners (to connect the sheathing panels to timber frames) and connection devices (hold-downs, angle brackets) are used, hence a high and well-distributed energy dissipation is expected.

Nevertheless, unlike what for other material structural types (concrete or steel), no well-defined criteria about the global failure mechanism to guarantee a consistent global ductility of the structure are reported in Standard. Moreover, it is not clear which connection type should be chosen (fasteners, hold-down or angle brackets) as the weakest element (where the ductility capacity of the structure is concentrated) and no analytical expression is suggested for the application of the capacity design approach for timber-frame buildings.

In this paper, a proposal for the application of the capacity design applied to timber-frame building is reported. Two main approaches are presented: the first one consistent with a medium ductility class, the second one to achieve a higher global energy dissipation.

2 CAPACITY DESIGN APPROACH TO TIMBER BUILDING

The capacity design (CD) for structures under seismic loads, can be illustrated through the chain model, originally developed in [2]. As the strength of a chain is the strength of its weakest link, one ductile link may be used to achieve ductility for the entire chain. According to the Eurocode 8 provisions [1], in timber structures the ductile link must be only concentrated in joint (e.g. dowel type joint) whereas the timber elements behave elastically (figure 1).

Therefore a reliable strength prediction of the joint and its components is essential for applying the CD and ensuring the required ductility. Potentially brittle mechanisms have in fact to be prevented by checking that their actual strength exceeds the strength demand. To this aim, an over-strength coefficient can be adopted in the design of the timber element:

$$R_{b,Rd} \geq \gamma_{Rd} \cdot R_{d,Rd} \tag{1}$$

where $R_{(b,Rd)}$ is the design strength of the brittle (timber) element, $R_{(d,Rd)}$ is the design strength of ductile element (mechanical connection) and γ_{Rd} is a suitable over strength factor (see [3] and [4]).

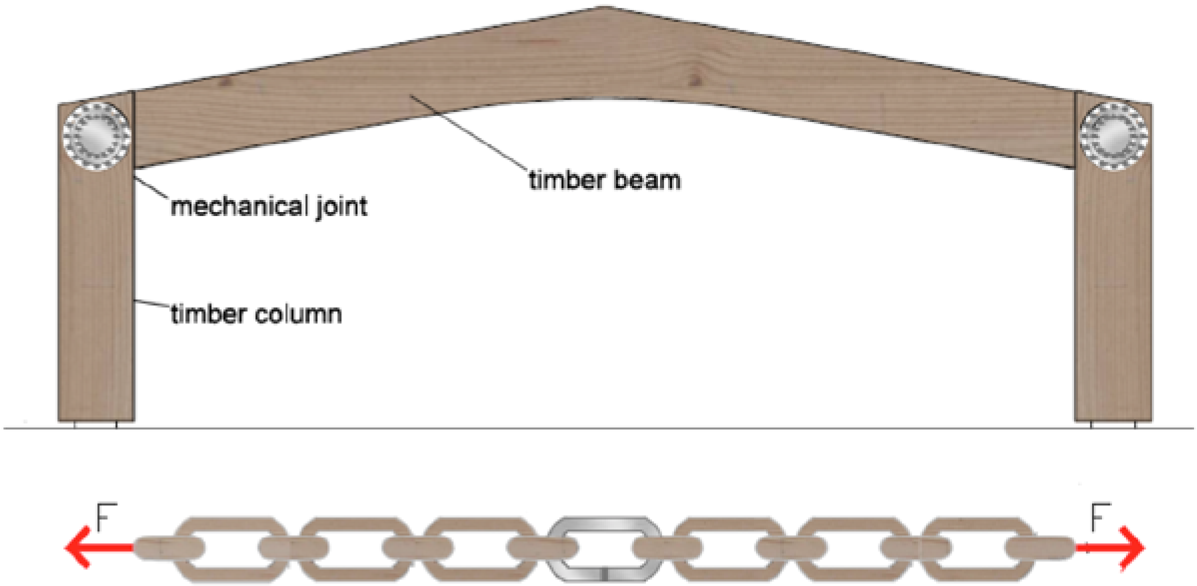

Figure 1: *The chain model applied to hyperstatic timber frames [5]*

The application of this concept is quite simple for most structural types (hyperstatic timber frames, timber hyperstatic beams) as presented in [4] an [5].

Concerning timber frame buildings the global failure mechanism of the structure is not described in Standards and hence it is not evident how to apply the CD concept correctly to this structural type. In the common practice all connection types (sheathing to framing connections, hold-downs, angle brackets) are usually considered as dissipative zones and are designed for the design load from the analysis (taking account of the behaviour factor *q*). Few prescriptive provisions are reported regarding the geometrical parameters in sheathing to framing connections (minimum thickness of the sheathing material and maximum nail diameter), without explicitly indicating this as the weakest component. On other hand was experimentally demonstrated [6] that the ductility of most commercial hold-downs and angle brackets looks pretty lower than the ductility obtained from sheathing-to-framing fasteners.

3 LINEAR ELASTIC RHEOLOGICAL MODEL OF A MULTI-STOREY TIMBER FRAMED SHEAR WALL

A timber frame shear wall subjected to a distributed vertical load and a horizontal force can be assumed as a statically determinate system, and can be represented by an elastic model represented in figure 2a, presented in [7] and in [8]. An equivalent representation is given by a rheological model composed by four in series spring (figure 3a), each of which is related to a single deformation contribution, namely: sheathing-to-framing connection (SH); sheathing panel shear (P); rigid body translation (A); rigid body rotation (H).

Assuming the hypothesis of an elastic-perfectly plastic behaviour for the mechanical connections, according the chain model, the global ductile failure occurs when the failure mechanism is related to the first one of the following structural components: sheathing-to-framing (SH), angle brackets (A), or hold-down (H). When the component associated of the weak mechanism is individuated, the other elements remain in the elastic range and can be defined as stronger components, according to CD approach (eq. 1). The global ductility of the wall depends only on the ductility of the weakest component. Hence, the provisions about ductility should be applied only to the weakest connection component whereas an elastic design can be used for other devices. It can be easily demonstrated that generally the wall ductility is lower than the weakest component ductility: they are equal only if the stiffness of the stronger components can be assumed infinity [8].

The case of a *M*-storey timber frame wall can be analysed in a similar way of 1-storey wall. A simple model for a *M*-storey wall can be obtained by means of the superimposition of *M* one-storey walls (figure 2b). It is not difficult to show that also this model is a statically determinate system. The rheological model used to represent its behaviour under a series of horizontal forces is in fact obtained linking in series *M* one-storey rheological models and adding all horizontal forces F_j (figure 3b).

As in the previous case, assuming an elastic-perfectly plastic behaviour for the mechanical connections, the structure yielding occurs when at least one component yields. In this case the weakest element of the chain should be determined in relation to the ratio between its strength and the tensile force. Hence, only one spring in the model described in figure 3b should be assumed as dissipative zone (for example the SH component at the i^{th} floor) whereas all other components should be considered as stronger components.

However, the described mechanism failure does not assure the participation of all the stories to the dissipation of the earthquake input energy, and for this reason a "soft-storey" global failure may occur. For this reason, according to the CD philosophy, aimed to obtain a better distributed ductile mechanism along the height of the walls,

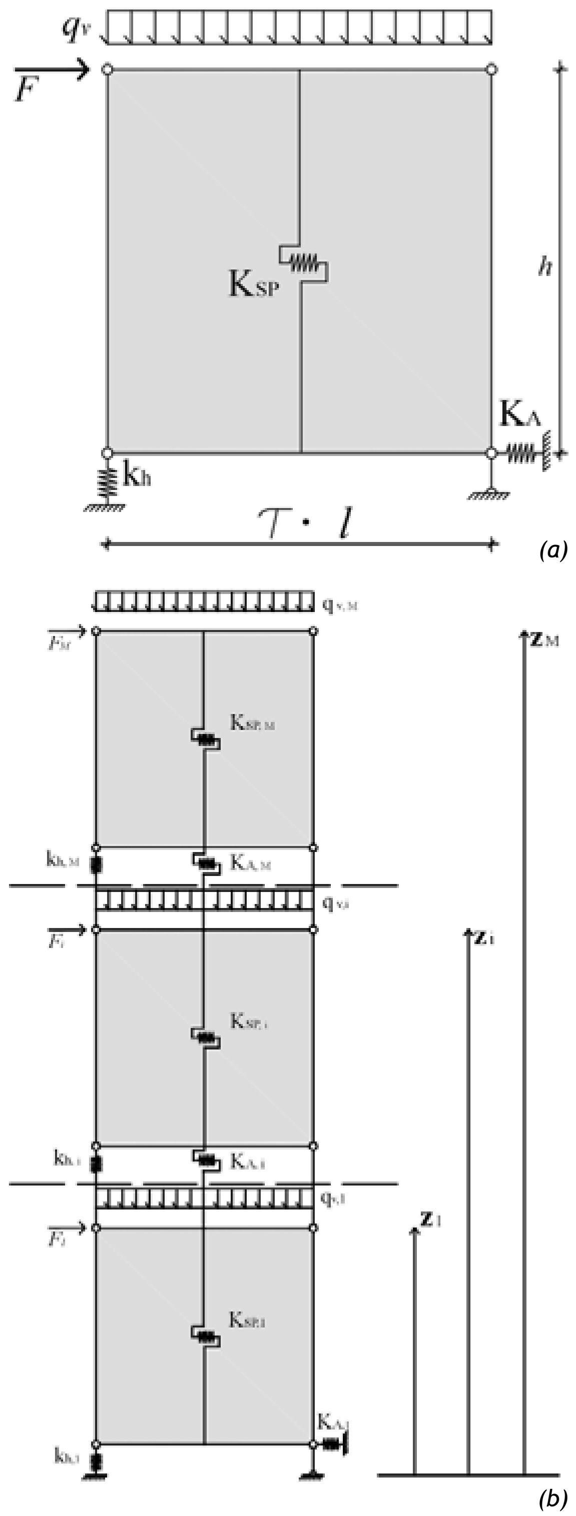

Figure 2: *Elastic model for one-storey and multi-storey timber frame wall ([7] and [8])*

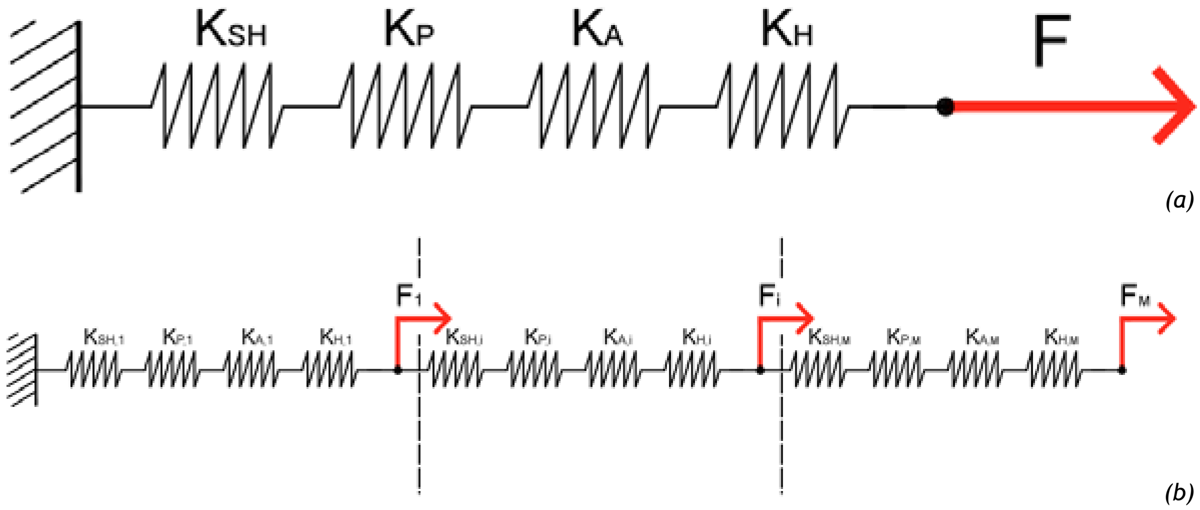

Figure 3: *Rheological model for one-storey and multi-storey timber frame wall ([7] and [8])*

in section 4 a new design method is proposed.

4 CAPACITY DESIGN APPROACH FOR A HIGH DUCTILITY CLASS

As reported in section 2 Standards assume that timber frame buildings are characterized by a high capacity to dissipate energy because a great amount of small diameter fasteners are used. When fasteners used to connect the sheathing panels to the timber frame are characterized by a local ductility consistent with the high value of the behaviour factor[1], it is reasonable assuming as dissipative zones the sheathing-to framing connections (SH). Since the proposed expressions are rigorously based on the CD concepts and only the sheathing-to framing connection is considered as dissipative zone (usually characterized by high ductility, as experimentally demonstrated in [9]), the method seems suitable for a high ductility class (HDC) of buildings.

4.1 One-storey shear wall

Assuming the sheathing-to-framing connection (SH) as the weakest ductile component, therefore it has to be designed taking account of the external design analysis force acting on the shear wall F_{Ed}. Once the single fastener design capacity $F_{f,Rd}$ is known, the maximum fastener spacing $s_{f,max}$ between the connectors along the edge of the sheathing panel can be calculated. According to [10] we get:

$$s_{f,max} = \frac{\sum_{j=1}^{N} 1{,}2 \cdot F_{f,Rd} \cdot b_j \cdot c_j \cdot n_{bs}}{F_{Ed}} \qquad (2)$$

where N is the number of sheathing panels, b_j is the width of the j^{th} panel, c_j is a reduction factor that takes account of the shape of the panel and n_{bs} is the number of the braced sides of the wall (1 or 2).

Adopting the real fastener spacing $s_f \leq s_{f,max}$ (but lower than 150 mm, according to [10]) it is possible to carry out the shear wall capacity $F^{SH}_{sw,Rd}$ related to the sheathing-to-framing (SH) failure mechanism.

$$F^{SH}_{sw,Rd} = \sum_{j=1}^{N} \frac{1{,}2 \cdot F_{f,Rd} \cdot b_j \cdot c_j \cdot n_{bs}}{s_f} \geq F_{Ed} \qquad (3)$$

On the contrary the design of angle brackets (A) and hold-downs (H) should be carried out according to the CD approach. Therefore the design loads are not obtained from the analysis, but they are related to the design capacity of the wall $F^{SH}_{sw,Rd}$.

Regarding the angle brackets the minimum number $n_{a,min}$ to permit a strong mechanism can be calculated as:

$$n_{a,min} = \frac{F^{SH}_{sw,Rd} \cdot \gamma_{Rd}}{F_{a,Rd}} \qquad (4)$$

where $F_{a,Rd}$ is the strength of an angle bracket and γ_{Rd} is a suitable over-strength factor.

Adopting the real number of angle brackets $n_a \geq n_{a,min}$ is possible to carry out the shear wall capacity related to the angle brackets failure mechanism $F^{A}_{sw,Rd}$ as:

$$F^{A}_{sw,Rd} = n_a \cdot F_{a,Rd} \geq F^{SH}_{sw,Rd} \qquad (5)$$

The tensile design force acting on the hold-down can be carried out by means the following

[1] According to the Eurocode 8 the dissipative zones shall be able to deform plastically for at least three fully reversed cycles at a static ductility ratio of 6 for ductility class, without more than a 20% reduction of their resistance.

equation derived from the rotational equilibrium considerations (see figure 4):

$$F_{h,Ed} = \frac{F^{SH}_{sw,Rd} \cdot \gamma_{Rd} \cdot h}{\tau \cdot l} - \frac{q_v \cdot l}{2 \cdot \gamma_{LOAD}} \leq F_{h,Rd} \qquad (6)$$

where l and h are length and height of the wall, τ is the coefficient that takes into account that the internal level arm may be lower than l (from 0.95 to 1), q_v is the uniform vertical load. In the load analysis the 95th percentile for the value q_v of the vertical load is adopted. Since q_v has a positive effect of in reducing the action on the hold-down, it has to be reduced by means of the coefficient $\gamma_{LOAD}=q_v/q_{v,k,0,05}$ to get the 5th percentile value.

The shear wall capacity related to the hold-down failure mechanism $F^{H}_{sw,Rd}$ is therefore:

$$F^{H}_{sw,Rd} = \left(F_{h,Rd} + \frac{q_v \cdot l}{2 \cdot \gamma_{LOAD}}\right) \cdot \frac{\tau \cdot l}{h} \geq F^{SH}_{sw,Rd} \qquad (7)$$

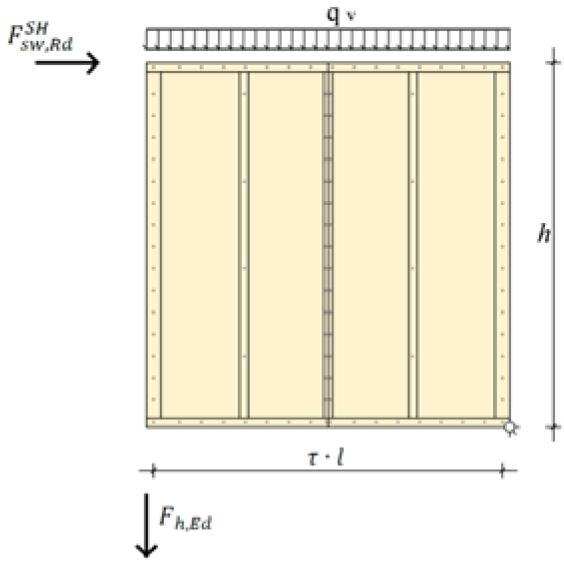

Figure 4: *Tensile force of the hold-down applying capacity design*

The brittle failure mechanism of compression (8) and tension (9) on the timber studs, of compression perpendicular to the grain in the bottom beam (10) and of shear stress on the sheathing panel (11) are represented by the following design equations, according to Eurocode 5 [10]:

$$\sigma_{c,0,d} = \frac{\frac{F^{SH}_{sw,Rd} \cdot \gamma_{Rd} \cdot h}{\tau \cdot l} + \frac{q_v \cdot l}{2}}{A_{stud}} \leq k_{c,y} \cdot f_{c,0,d} \qquad (8)$$

$$\sigma_{t,0,d} = \frac{\frac{F^{SH}_{sw,Rd} \cdot \gamma_{Rd} \cdot h}{\tau \cdot l} - \frac{q_v \cdot l}{2 \cdot \gamma_{LOAD}}}{A_{stud}} \leq f_{t,0,d} \qquad (9)$$

$$\sigma_{c,90,d} = \frac{\frac{F^{SH}_{sw,Rd} \cdot \gamma_{Rd} \cdot h}{\tau \cdot l} + \frac{q_v \cdot l}{2}}{A_{eff}} \leq k_{c,90} \cdot f_{c,90,d} \qquad (10)$$

$$\tau_d = \frac{F^{SH}_{sw,Rd} \cdot \gamma_{Rd}}{l \cdot t} \leq f_{v,d} \qquad (11)$$

Where A_{stud} is the area of the external stud of the wall, A_{eff} is effective area under compression in the bottom beam, t is the sheathing panel thickness. The other symbols are in accordance with the Eurocode 5 [10]. The previous equations can be easily obtained by the simple equilibrium considerations substituting the acting force F_{hEd} with the shear wall capacity $F^{SH}_{sw,Rd}$ multiplied by the over-strength factor γ_{Rd} (see figure 4).

4.2 Multi-storey shear wall

Similar expressions can be written in case of multi-storey walls. In according to the model presented in the section 3, the statically determinate multi-storey walls are connected at each level with mechanical devices counteracting translation and rotation of the single storey walls.

Consistently with the method previously introduced for one-storey wall, the energy dissipation can be assumed concentrated in the sheathing-to-framing connections. In order to guarantee a well-distributed energy dissipation along the height of the building, an approach similar to the proposed one for designing steel X-braced frames could be adopted according Eurocode 8 [1], where the global mechanism failure is characterized by the yielding of the diagonal elements at each storey level. The diagonals in fact should be designed so that the ratio between their strength and the tensile design force is uniform along the height of the same braced frame. In this way a soft-storey floor failure mechanism is prevented.

For a multi-storey timber frame wall, once the seismic design shear action $V_{Ed,i}$ is known from the seismic analysis (see figure 5), at each level i_{th} (i=1…..M, where M is the number of storeys) the maximum fastener spacing $s_{f,max,i}$ along the edge of the sheathing panel can be calculated.

$$s_{f,max,i} = \frac{\sum_{j=1}^{N} 1{,}2 \cdot F_{f,Rd} \cdot b_j \cdot c_j \cdot n_{bs}}{V_{Ed,i}} \qquad (12)$$

Adopting the real spacing $s_{f,i} \leq s_{f,max,i}$ (but lower than 150 mm) is possible to carry out the shear wall capacity at each floor $F^{SH}_{sw,Rd,i}$ related to the sheathing-to-framing failure mechanism (SH$_i$).

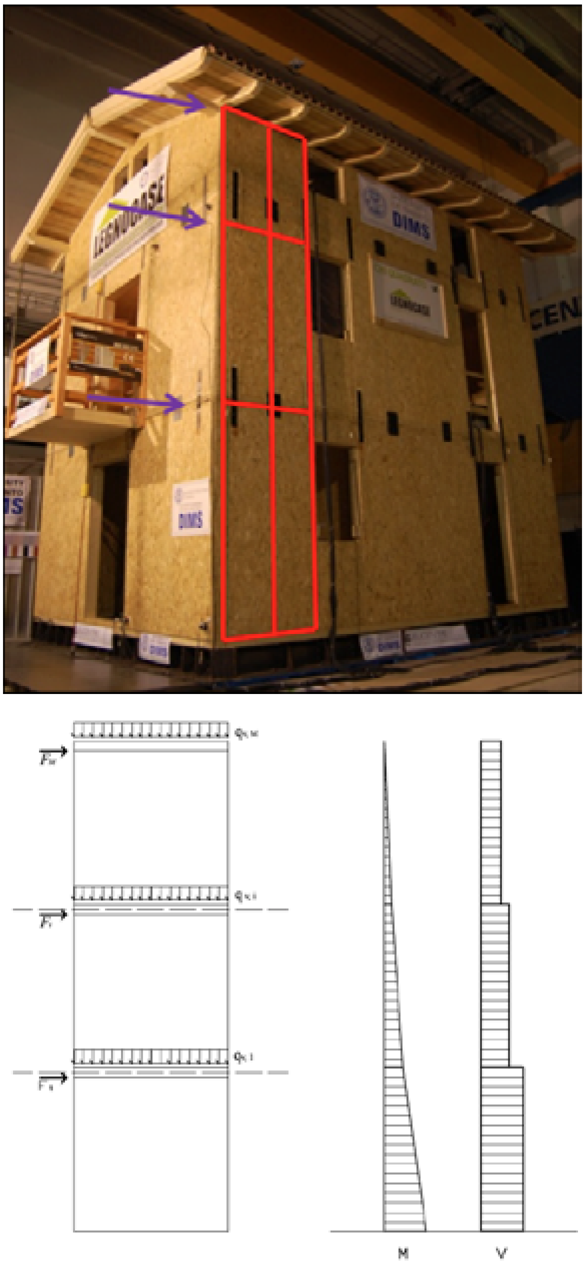

Figure 5: *Multi storey timber frame building: the cantilever model*

$$V^{SH}_{sw,Rd,i} = \frac{\sum_{j=1}^{N} 1{,}2 \cdot F_{f,Rd} \cdot b_j \cdot c_j \cdot n_{bs}}{s_{f,i}} \tag{13}$$

The over strength factor a_i at the i^{th} storey can be calculated as:

$$\alpha_i = \gamma_{Rd} \cdot \frac{V^{SH}_{sw,Rd,i}}{V_{Ed,i}} \tag{14}$$

Assuming the wall as a statically determinate system the global over strength factor for the wall is therefore:

$$\alpha = min(\alpha_i) \tag{15}$$

In order to guarantee a well-distributed energy dissipation along the height of the wall uniform values of a_i should be obtained. This could be guaranteed by the following expression:

$$\alpha_{max} \leq \varphi \cdot \alpha \leq q \tag{16}$$

where φ is a suitable parameter that should be proposed for timber frame walls (e.g. for X-braced steel frames in [1] it is equal to 1.25).

The CD approach for the design of the stronger components (H_i and A_i) can be adopted at each level in relation to the shear wall over-strength factor *a*. Hence, the design shear force on the angle brackets $V_{d,A,i}$ should satisfy the following expression:

$$V_{d,A,i} = \alpha \cdot V_{Ed,i} \leq V_{Rd,A,i} \tag{17}$$

where $V_{Rd,A,i}$ is the shear wall design strength at the i^{th} storey related to angle brackets.

With the same criterion, the tensile force acting on the hold-down at the level i^{th} can be calculated as:

$$F_{h,Ed,i} = \alpha \cdot F_{h,Ed,E,i} - \sum_{j=i}^{M} \frac{q_{v,j} \cdot l}{2 \cdot \gamma_{LOAD}} \leq F_{h,Rd,i} \tag{18}$$

where $F_{h,Ed,E,i}$ is the design tensile force on the hold-down obtained from the seismic analysis at the i^{th} storey and $F_{h,Rd,i}$ is the design hold-down strength at the same storey. Also in this case a load coefficient γ_{LOAD} is introduced to reduce the vertical loads to the 5th percentile.

Similar expression to equations from (8) to (11) can be written for the failure mechanism, related to timber elements at each level:

$$\sigma_{c,0,d} = \frac{\alpha \cdot F_{h,Ed,E,i} + \sum_{j=1}^{N} \frac{q_{v,i} \cdot l}{2}}{A_{stud}} \leq k_{c,y} \cdot f_{c,0,d} \tag{19}$$

$$\sigma_{c,90,d} = \frac{\alpha \cdot F_{h,Ed,E,i} + \sum_{j=1}^{N} \frac{q_{v,i} \cdot l}{2}}{A_{eff}} \leq k_{c,90} \cdot f_{c,90,d} \tag{20}$$

$$\sigma_{t,0,d} = \frac{\alpha \cdot F_{h,Ed,E,i} - \sum_{j=1}^{N} \frac{q_{v,i} \cdot l}{2 \gamma_{LOAD}}}{A_{stud}} \leq f_{c,0,d} \tag{21}$$

$$\tau_d = \frac{\alpha \cdot V_{Ed,i}}{l \cdot t} \leq f_{v,d} \tag{22}$$

It has to be underlined a proper design need to keep the values of the *a* over-strength coefficient as uniform as possible according to the limitation reported in eq. 16, with the aim to promote the sheathing to framing connection yielding of more than one storey. However this approach needs that fastener spacing and number of braced side (1 or 2) may vary at each level and at each shear wall. Moreover, since the maximum spacing of the sheathing to framing fastener is limited according to the code provisions, this method could present some difficulties to be applied, especially in the case of low seismic design action. For this reason, a possible strategy for building characterized by at least three storeys, could be not to apply the limitation of eq. 16 for the top storey, considering that its seismic design shear force is usually much lower than the bottom storey ones.

5 CAPACITY DESIGN APPROACH FOR A MEDIUM DUCTILITY CLASS

In this section a second approach for the application of the CD method to multi-storey timber frame walls is presented. Differently from the method for HDC reported in section 4, this method is not rigorously based on the CD concepts. Coherently with the common practice all sheathing-to-framing fasteners and connection devices (hold down and angle brackets) are considered as dissipative zones. Their design force is obtained from the seismic analysis and the CD approach is applied only to timber elements (studs, bottom beams and sheathing panels). Since angle brackets and hold-downs are considered in this case dissipative zones, the provisions regarding their local ductility reported in Eurocode 8 [1] should be satisfied (the dissipative zones shall be able to deform plastically for at least three fully reversed cycles at a static ductility ratio of 4 for ductility class, without more than a 20% reduction of their resistance). However, many studies demonstrated that most commercial devices are not characterized by a high local ductility and are usually designed in relation to their strength. For this reason the approach proposed in this section should be applied in case of a medium ductility class (MDC) and hence a reduced behavior factor should be adopted. Further studies are necessary in the future to calibrate this value.

5.1 Multi storey shear wall

Since SH, H and A components are assumed as dissipative zones, the loads from the seismic analysis can be used in the design phase and the CD approach is not applied. Hence the following expression should be satisfied at each level i^{th} (i=1…..M, where M is the number of storeys):

$$V_{sw,Rd,i}^{SH} = \frac{\sum_{j=1}^{N} 1{,}2 \cdot F_{f,Rd} \cdot b_j \cdot c_j \cdot n_{bs}}{s_{f,i}} \geq V_{sw,Ed,i} \tag{23}$$

$$V_{sw,Rd,i}^{A} = n_a \cdot F_{a,Rd} \geq V_{sw,Ed,i} \tag{24}$$

$$F_{h,Rd,i} \geq F_{h,Ed,i} = F_{h,Ed,E,i}^{\square} - \sum_{j=1}^{M} \frac{q_{v,i} \cdot l}{2} \tag{25}$$

Since the CD should be applied to timber elements, the over strength factor of the entire wall β should be calculated. This is defined as the minimum of the over strength factor β_i at the i^{th} level:

$$\beta = min(\beta_i) \tag{26}$$

$$\beta_i = min\left(\frac{\gamma_{Rd} \cdot V_{sw,Rd,i}^{SH}}{V_{Ed,i}}; \frac{\gamma_{Rd} \cdot V_{sw,Rd,i}^{A}}{V_{Ed,i}}; \frac{\gamma_{Rd} \cdot F_{h,Rd,i} + \sum_{j=i}^{M} \frac{q_{v,j} \cdot l}{2}}{F_{h,Ed,E,i}}\right) \tag{27}$$

The brittle failure mechanisms for timber elements can be verified at each level applying the CD as in the approach for HDC buildings, using the factor β.

$$\sigma_{c,0,d} = \frac{\beta \cdot F_{h,Ed,E,i} + \sum_{j=1}^{N} \frac{q_{v,i} \cdot l}{2}}{A_{stud}} \leq k_{c,y} \cdot f_{c,0,d} \tag{28}$$

$$\sigma_{c,90,d} = \frac{\beta \cdot F_{h,Ed,E,i} + \sum_{j=1}^{N} \frac{q_i \cdot l}{2}}{A_{eff}} \leq k_{c,90} \cdot f_{c,90,d} \tag{29}$$

$$\sigma_{t,0,d} = \frac{\beta \cdot F_{h,Ed,E,i} - \sum_{j=1}^{N} \frac{q_i \cdot l}{2 \cdot \gamma_{LOAD}}}{A_{stud}} \leq f_{c,0,d} \tag{30}$$

$$\tau_d = \frac{\beta \cdot V_{Ed,i}}{l \cdot t} \leq f_{v,d} \tag{31}$$

6 CONCLUSION

In this paper two analytical approaches for the application of the capacity design to timber frame buildings are presented. The first one (method HDC) assumes that only the sheathing-to-framing connection components can dissipate energy since, if well-designed, may be characterized by a high ductility. Assuming a timber frame wall as a statically determinate system, other connection devices (angle brackets and hold-downs) should be designed in relation to the strength of the weakest component according to the design capacity approach. The provisions regarding the local ductility should be satisfied only to the sheathing-to-framing connections and it seems correct to adopt a q factor related to a high ductility class of the structures. The second method (approach MDC) is simplified and considers both fasteners and connection devices able to dissipate energy. All of them are designed in relation to the analysis loads and the capacity design approach is applied only to timber elements. There is not a global control on the mechanism failure (the weakest component may be a fastener, an angle bracket or a hold-down) and the provisions about the local ductility are to be applied to all components (SH, A and H). In this case, since the ductility of connection devices is usually lower the fastener one, a reduced value of the behaviour factor *q* should be adopted.

Some practical rules and expressions are proposed to apply the capacity design approach as requested by Standards. However, further studies should be carried on, in order to select the value of the behaviour factor *q* for the two approaches (with particular regard to the second one for medium ductility class), and to calibrate the value of the parameter φ, so to have a sufficient distributed energy dissipation.

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