DIRECT DISPLACEMENT-BASED SEISMIC DESIGN OF TIMBER STRUCTURES WITH DOWEL-TYPE FASTENER CONNECTIONS

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SUMMARY:
The applicability of the Direct Displacement-Based Design (DBD) procedure is strictly related to *a priori* evaluation of the design displacement and the matching Equivalent Viscous Damping (EVD) of the structure. In this paper we propose analytical models of these design parameters, at the ultimate limit state, for wooden structures built with engineered joints. Experimental results show that the plastic resources and dissipative capabilities of timber structures under earthquake conditions are ensured by the connections between the members. Therefore, the formulation of the design DBD parameters is based on the mechanical model of the single connector and assumes the inelastic deformation of the structure to be concentrated at the joints. The expected non-linear response of the connections can be either ductile or brittle. However, through an appropriate choice of the geometry and strength characteristics of the materials, in the design process we can control the expected ductile behavior of joints.

Keywords: Displacement-Based Design, Equivalent Viscous Damping, Timber structures, Timber connections, Dowel-type fasteners

1. INTRODUCTION

The Direct Displacement-Based Design (Direct-DBD) approach was developed to design structures in seismic zones, using displacement as input parameter. The Direct-DBD methodology was first codified by Priestley (1998) and subsequently developed by several other researchers. A key reference on displacement-based methods applied to various materials and structural types is Priestley et al. (2007), while a first attempt to incorporate the Direct-DBD methodology into a design code was among the objectives of a research project which has recently ended [(RELUIS, 2009), (Calvi and Sullivan, 2009)].

Overviews of Direct Displacement-Based seismic Design methods (Direct-DBD) for timber structures can be found in Priestley et al. (2007), Newcombe (2010) and Loss (2011). The current development state of the Direct-DBD methods shows fully available procedures for a few structural types, such as wood portal frames, while for other structure types we are still far from full applicability. In this paper we propose an approach to formulate a Direct-DBD procedure for any type of structural system built with timber elements assembled using dowel-type fastener connections. The design of such structures involves joint optimization and an engineering solution to ensure their required static and dynamic behavior. Joints with dowel-type mechanical fasteners (nails, staples, screws, dowels, bolts and other pin-like fastener systems) are recognized today as the most important connection systems in timber engineering. These connections are used alone or in combination with other devices, such as formed steel parts or “system-fasteners” (Augustin, 2008).

Independently of the structural material, the Direct-DBD procedure codified by Priestley (2003) requires evaluation of a target design displacement (Δ_d) and a matching Equivalent Viscous Damping (ξ_{eq}) for the selected performance level.

Here we provide a rational Direct-DBD procedure, consistent with the state-of-the-art of timber structures and in particular of timber connections. The model presented allows simple and direct assessment of design parameters, based on the mechanical description of dowel-type fastener connections. This enables an extension of the formulation to several types of structure with timber components. In this paper we show analytical expressions for design parameters tailored to glulam portal frame systems and to wood frame systems.

2. EARTHQUAKE-RESISTANT TIMBER CONSTRUCTIONS

It is well known that timber buildings can be designed with adequate strength and reliability to withstand earthquake loads: to protect life, an appropriate design procedure must be followed to reduce damage and prevent their possible collapse (Karacabeyli and Popovsky, 2003). Today there are many different building techniques and systems, 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. In particular we note that joints are often of the dowel type, with or without the use of formed steel components.

This work focuses on the category of timber structures designed to withstand earthquakes; and in particular on structures normally used in Europe. The seismic behavior of wood structures is strongly influenced by the behavior of the connections. Therefore, the selection of construction typologies must meet the requirements for dissipation capacity and ductility of joints. Some typical European wooden structures that ensure ductile behavior are shown in Figure 1. Figure 1(a) is a simplified drawing of the system used to build commercial, industrial and other open-space buildings.

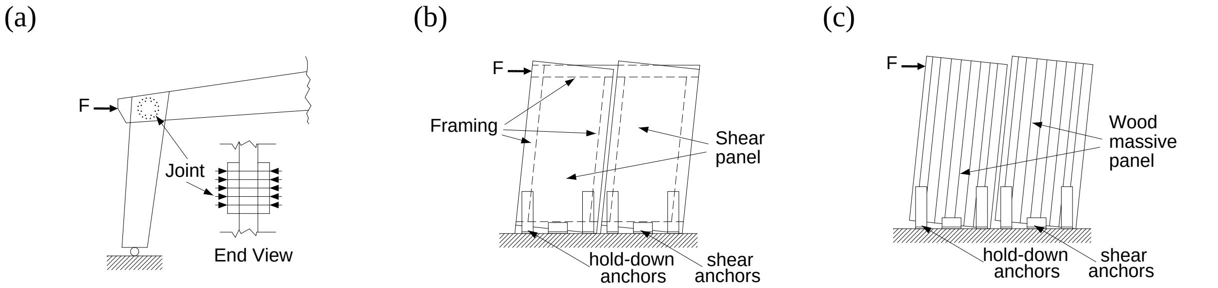

Figure 1. Examples of ductile timber structural systems; (a) Moment-Resisting timber portal frame; (b) wood frame system; (c) Cross-laminated timber panel system (modified from Priestley et al. 2007).

The systems commonly used in multi-storey timber buildings are: the wood frame system, or timber frame panel system (Figure 1(b)), and the cross-laminated solid wood panel system, also known as the XLAM (Figure 1(c)). Wood frame systems (Figure 1(b)) are usually built with the platform frame manufacturing technique. In the platform frame system each floor offers a work surface for the next, so that erection proceeds easily. The cross-laminated solid wood panel system (Figure 1(c)) has recently been introduced into Europe and is a valid alternative to the traditional wood frame system. The concept of global stability of the structure is very similar (erection using the platform frame technique) to that of traditional wood frame system construction; except that the timber frame panels are replaced by cross-laminated massive wood panels. Wind and seismic loads are transmitted to the ground by similar methods. Other types of earthquake-resistant construction systems can be seen in Europe, but are less frequently used for housing.

3. DESIGN OF TIMBER CONNECTIONS

3.1. Behavior of timber joints under cyclic loading

Under seismic loads the structure must offer strength, stiffness, ductility and, in particular, energy dissipation capacity. This capacity in a timber structure is essentially the result of the cyclic behavior of the connections and can reduce during a seismic event. The hysteretic behavior of connections is influenced by the capacity of the mechanical fasteners and the strength properties of wood, as a function of the load direction relative to the direction of the grain (Piazza et al., 2005). The required ductility of the system is reached by adequate selection and design of the connections (Dolan, 1994) to avoid brittle failure mechanisms. To ensure a ductile structure response, the design of the connections must respect the *Capacity Design* rules (CEN, 2004b).

The connections normally used in modern timber structures are steel devices that ensure transmission of forces between the various wood structural elements. In this research we refer to modern connections that use dowel-type mechanical fasteners and metallic devices. Experiments have shown that the shape of the hysteresis loops in connections with dowel-type mechanical fasteners is sensitive to the amplitude of the imposed displacement. In Karacabeyli and Popovski (2003) some recent cyclic tests on modern connections, including the dowel-type, are discussed. The tests showed that the failure mechanism involving yield of the dowel and the embedment strength of wood can provide high ductility and excellent energy dissipation under repeated load cycles. Such performance is ensured, in particular, by well-spaced slender dowels.

The typical expected cyclic load-slip curve (F-ν) of a ductile connection with steel dowels is shown in detail in Figure 2(a) (Ceccotti, 1995).
Connections in wood are characterized by two important phenomena: the *pinching effect* and the *memory of the material*. The *pinching effect* modifies the hysteresis cycles in the transition from first to subsequent cycles for the same displacement. The typical pinched hysteretic cycle is characterized by a thinner loop in the middle compared to the ends. This phenomenon is caused by the cavity formed around the fasteners during plastic deformation. The *memory of the material* implies that the load-slip curve of the connection at a given time is a function of the instantaneous displacement and of the loading history (Dolan, 1994). Both these phenomena lead to a reduction in the energy dissipated in hysteresis loops. Available numerical models capable of accurately describing the cyclic load-slip curve for a single dowel-type connector, e.g. Allotey (1999) and Lo (2002), appear very sophisticated and allow a complete investigation of the whole dynamic response up to failure.

(a) (b)

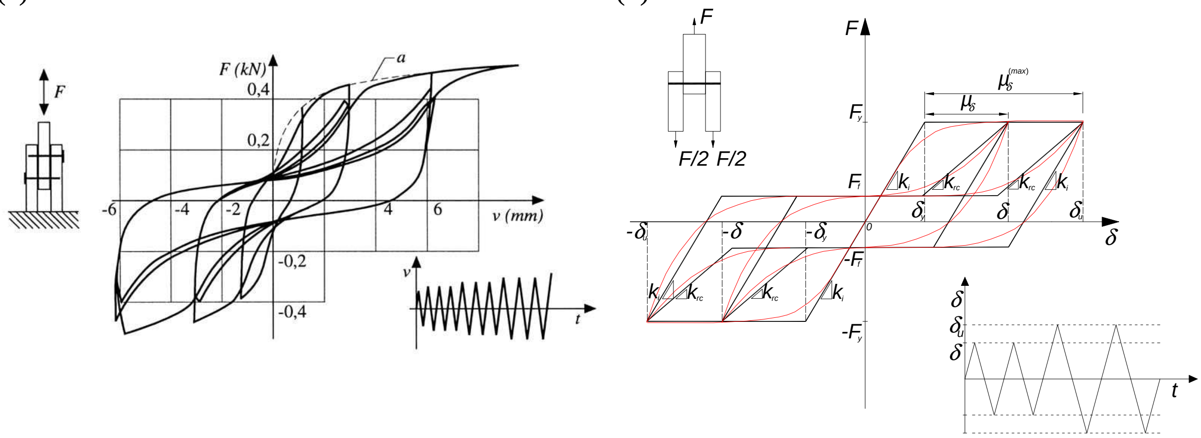

Figure 2. (a) Typical cyclical behavior of a nailed connection with nail slenderness of 8.5. The dashed line shows the Envelope curve (Ceccotti, 1995); (b) Expected load-slip curve of a dowel-type fastener connection (red line); Analytical load-slip curve of a dowel-type fastener connection (black line).

This research, on the other hand, focuses on the study of the dynamic response of a structure at the ultimate limit state, a condition that can be studied with limit analysis. In ductile connections, for displacement close to the ultimate state level, the state of deformation involves the development, at least in part, of plastic hinges in the dowels. The maximum energy dissipation capacity of the connection is reached when all the available plastic hinges are developed. For ductile connections with dowel-type fasteners in double shear plane, timber-to-timber, neglecting the strength and stiffness losses under consecutive cycles of loading, the expected load-slip curves are shown in red in Figure 2(b). This assumption leads to negligible error under conditions of high ductility, since the energy dissipated at the ends of the hysteresis curve becomes small compared with that dissipated in the central part of the cycle.

The analytical load-slip cyclic model formulated for the single dowel is that shown in Figure 2(b) as a simple linearization of the expected curve. The analytical model shown in Figure 2(b) in bold is acceptable for displacement amplitudes close to the ultimate displacement of the connection. The generic hysteresis curve consists of an elastic - perfectly plastic branch in the loading phase, unloading with a slope equal to the elastic stiffness, and residual plastic deformation restored by a force equal to that required to plasticize the dowel and overcome friction generated between the wood and steel surfaces of the elements. After the first cycle, the response is based on a curve where the plateau branch is set to the restoring force F_f, while in the final part the stiffness is equal to k_{rc}, whereby the bearing capacity F_y is restored. The parameters required to define the cyclic load-slip curve are: F_y, δ_u (or μ_δ), F_f, k_i and k_{rc}.

In the next Section we describe the evaluation method for the load capacity of the single dowel F_y and the restoring force to the undeformed situation, here noted as F_f (restoring force). Parameters μ_δ, k_i and k_{rc} can be evaluated immediately with the data available in the literature.

3.1. Extended European Yield Model for ductile connections

The so called European Yield Model (EYM) is the most commonly recognized analytical model for evaluation of the load-bearing capacity (F_y) of dowel-type mechanical fasteners, laterally loaded, based on the failure mode expected. The EYM is currently adopted in several design codes for wooden buildings, such as Eurocode 5 (CEN, 2004a), and represents the natural extension of the model presented by Johansen (1949). The EYM model foresees different failure modes (named I, II and III), each accompanied by a series of restrictions on the relative distance between fasteners and the edges of the elements connected, to avoid unexpected failure mechanisms.

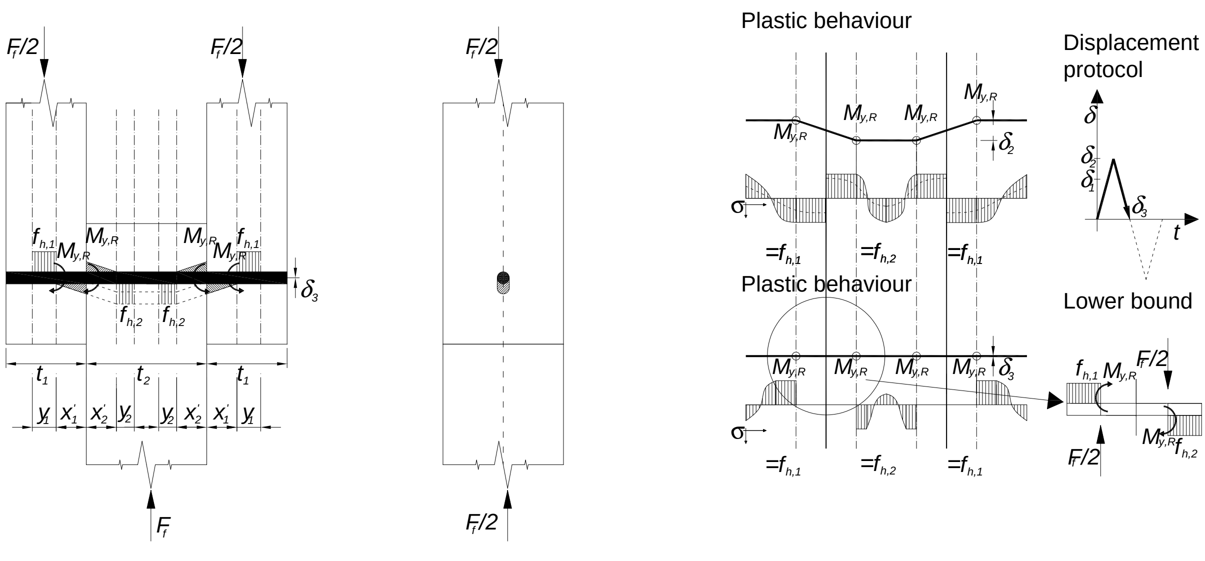

Figure 3. Limit equilibrium for Failure Mode III, for a dowel-type fastener connection in double shear, on return to the undeformed situation

If appropriately designed against seismic forces, timber joints are expected to fail according to a ductile mode: in accordance with the EYM, the ductile failure modes of dowel-type joints are types II and III. For fasteners in double timber-to-timber shear planes, according to the EYM, the only failure mode that ensures high ductility and energy dissipation capacity is failure mode III. Indeed, the ductile mode with the maximum number of plastic hinges in the connector is failure mode III. The load capacity (F_y) that is generated as a result of dowel yield and wood embedment is estimated using equilibrium of the forces acting on the wooden connection in the ultimate situation (Figure 3). F_y can be estimated as a function of the mechanical properties of the materials (wood and steel) and from the joint configuration using the analytical formula in Eurocode 5 (CEN, 2004a).

By analogy, F_f can be evaluated by simply imposing the equilibrium in a new ultimate deformed situation, consistent with the limit analysis (lower bound theory). In a new dowel equilibrium, no longer with wood support in the zone between two successive plastic hinges, we can calculate the restoring force F_f from the force F_y (Figure 3). The restoring force F_f is the sum of the forces due to plastic deformation of the dowel and to friction between dowel and wood. Neglecting the latter, we can write the following equations (Eqns. 3.1 and 3.2):

$$-M_{y,Rk} - M_{y,Rk} + F_f\left(\frac{y_1}{2} + x_1' + x_2' + \frac{y_2}{2}\right) = 0 \tag{3.1}$$

$$F_f = f_{h,1,k} \cdot d \cdot y_1 = f_{h,2,k} \cdot d \cdot y_2 \tag{3.2}$$

Where $M_{y,Rk}$ is the characteristic fastener yield moment, $f_{h,i,k}$ is the characteristic embedment strength in timber member i, while x_1’, x_2’, y_1 and y_2 are the lengths required for stress equilibrium on the connector. Equation 3.1 follows from the rotation equilibrium of a half connection, while Eqn. 3.2 follows from the translational equilibrium on each i^{th} wood element. The lengths x_1’ and x_2’, required for stress equilibrium on the dowel, are analytically evaluated according to EYM [(CEN, 2004a); see also Piazza et al. (2005)]. With some substitution and remembering how each parameter is defined, we can find a second order equation in variable y_1. In conclusion, the restoring force to the undeformed state is obtained with Eqn. 3.3.

$$F_f \cong 0.33 F_y = \sigma_F F_y \tag{3.3}$$

Where σ_F is defined as the ratio between the restoring force (F_f) and the bearing capacity of the dowel (F_y). In the same manner we can evaluate F_y and F_f for connections in panel-to-timber shear configuration. In Loss PhD thesis (2011) these analytical expressions are provided.

3.2. Recommended provisions for ductile connections

The current Eurocode 8 [EC8 (CEN, 2004b)] includes a series of design provisions developed to build systems with an expected high ductility behavior (DCH). For construction systems assembled with fasteners in timber-to-timber shear plane mode, EC8 sets the minimum slenderness value (= the ratio between the thickness of the wood members and fastener diameter) of 10, while the maximum diameter of the dowels is 12 mm. The use of these geometrical rules proposed in EC8 (CEN, 2004b) guarantees type III failure mechanism in the fastener connections, regardless of the materials used. Möller's chart (Möller, 1951) can be used to show that type III failure mode is always ensured independently of the wood and steel strength classes. Figure 5 shows Möller's point diagrams, for a diameter of dowel of 12 mm, where the solid lines represent the boundaries of each of the three failure modes defined for the connection. The cloud of points in each diagram represents the variability of the materials (wood and steel) for the same geometry.

The failure mode III (EYM) involves the maximum static slip ductility and the maximum energy dissipation capacity of the connections. In compliance with EC8 (CEN, 2004b), we can expect that a connection designed for the DCH ductile class will ensure a static ductility ratio of 6, with no more than 20% strength loss.

4. DIRECT DISPLACEMENT-BASED DESIGN METHOD FOR TIMBER STRUCTURES

This Section describes the analytical formulae for the design displacement and for Equivalent Viscous Damping (EVD), based on the local behavior of the single dowel-type connector. The design displacement (Δ_d) corresponds to the maximum deformation capacity of the structure at the ultimate limit state. The EVD represents the energy dissipation capacity of the structure at the design displacement level. In the inelastic phase, Δ_d and EVD are functions of the element geometry, of connection details and of the properties of the materials used. However, for the ultimate limit state condition, we propose a simple analytical model.

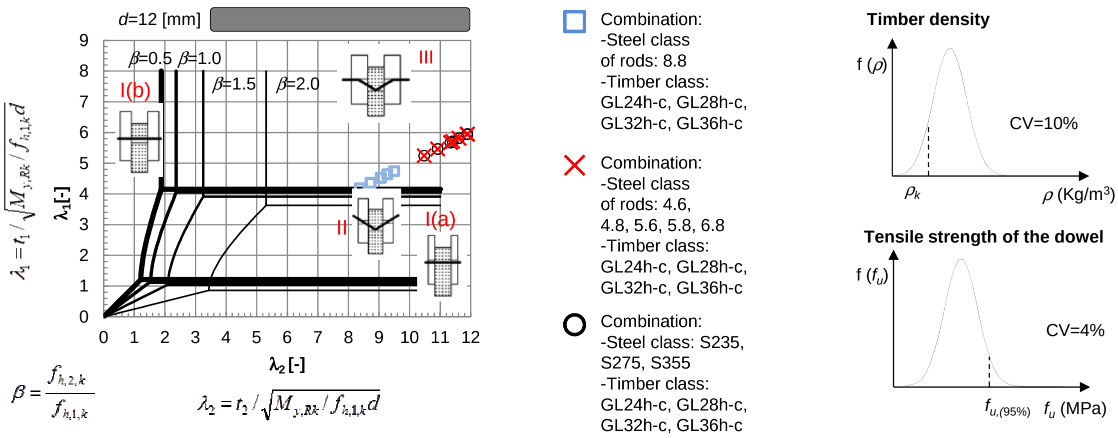

Figure 4. Möller's chart for dowel-type mechanical fasteners with various mechanical configurations

4.1. Formulation of the design displacement (Δ_d)

The analytical model of the design displacement assumes that the inelastic deformation is concentrated at the joints. Therefore, the timber elements must be designed with appropriate over-strength factors with respect to the connections, according to the Capacity Design philosophy, while the connectors must ensure a ductile failure mode. In order to reduce the effect of an earthquake on the structure, we consider the timber building designed for high dissipative behavior (High Ductility Class, DCH), in accordance with Eurocode 8 (CEN, 2004b).

For structures assembled with engineered joints, the design displacement (Δ_d) can be estimated as the sum of the inelastic displacement due to rigid rotation of elements, following connector yield (Δ_j), and the elastic deformation of the timber members (Δ_s). Indeed, the design assumptions in which wood elements are considered more rigid than the joints, usually accepted in the elastic range, can be extended to the plastic range, as in the post-yield phase there is always a stiffness reduction generally due to structural damage. The rotation in the joints is then given by a rigid deformation with inelastic slip of fasteners.

At the ultimate limit state, we can find the instantaneous centre of joint rotation from the geometrical centre of the connections, considering the resulting direction of internal forces (shear stress, axial force, bending moment). The criterion that defines the ultimate capacity of the joint is that for which the maximum slip is reached in the most stressed dowel. The inelastic displacement is simply derived from the ultimate joint rotation, given the ultimate slip of the most stressed dowel. The elastic displacement component can be evaluated by the theory of elastic beams and is independent of the configuration of connections.

4.2. Formulation of the Equivalent Viscous Damping (EVD)

The evaluation of the Equivalent Viscous Damping is based on Jacobsen's energy approach (Jacobsen, 1930). The damping is calculated as the ratio between hysteresis energy dissipated in a mid-cycle and the potential energy stored by an equivalent simple oscillator for the same displacement amplitude.
For dowel-type connections the hysteretic dissipation is due partly to the steel dowels that embed in the wood during the loading phase and partly to the plastic hinges in the fastener, as explained previously. Because this mechanism implies reduction of energy dissipation after each cycle, the total amount of energy dissipated depends largely on the load protocol. As a function of the load history applied to the connection, we can define different damping-to-ductility curves. The connection load history due to seismic loads is expected to be irregular, related to events that are unpredictable by nature.

In Europe, estimation of the dissipative connection properties is allowed by using the cyclic quasi-static test protocol provided by EN 12512 (CEN, 2001). According to EN 12512, the damping must be evaluated in the third cycle of each level of displacement imposed by the standardized loading protocol.

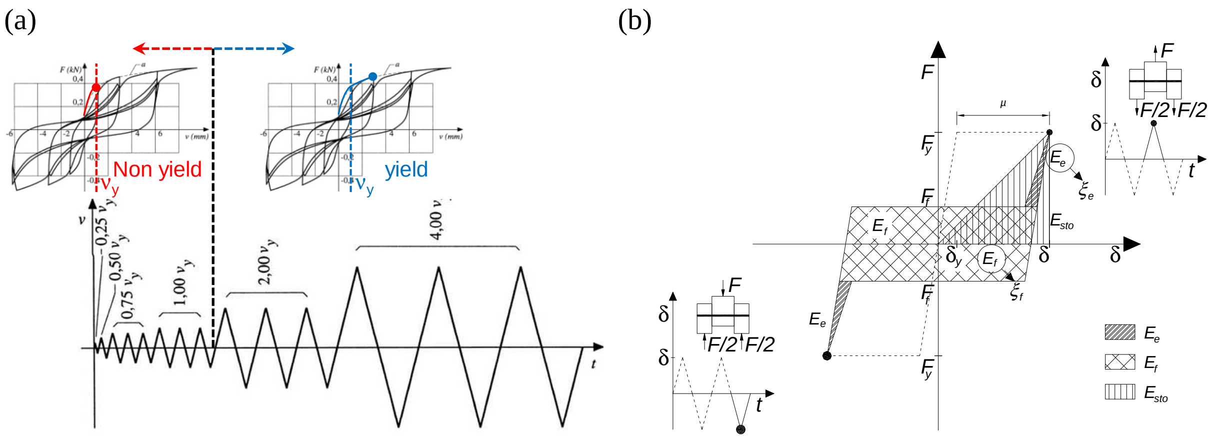

Figure 5. (a) Procedure for cyclic testing following EN 12512 (CEN, 2001); (b) Definition of equivalent viscous damping ratio for a dowel-type fastener connection based on analytical hysteretic curve

As demonstrated in Section 3, the hysteretic model selected to describe the behavior of dowel-type fastener connections is that of Figure 2(b). At this point, based on Jacobsen's energy approach, through simple mathematical operations (Figure 5(b)) we can calculate the final value of the damping of dowels ($\xi_{eq,dowel}$), at the third cycle, as the direct sum of two components, ξ_e and ξ_p (Eqn. 4.1).

$$\xi_{eq,dowel} = \xi_e + \xi_p = \frac{(\beta_k - 1)}{2\pi\mu_\delta}(1-\sigma_F)^2 + \frac{2\sigma_F}{\pi}\left(1 - \frac{1}{\mu_\delta}\right) \tag{4.1}$$

ξ_e estimates the energy dissipated between the time when the fastener regains stiffness from contact with the wooden surface and the time when the previous displacement is reached. With ξ_p we estimate the effects of friction and plasticity of the fastener during the loading and unloading phases. In Eqn. 4.1 the parameters $\sigma_F(=F_f/F_y)$, $\beta_k(=k_i/k_{rc})$ and $\mu_\delta(=\delta_u/\delta_y)$ are numerically evaluated using the parameters defined for the hysteretic curve. When the equivalent viscous damping value of one connector is known ($\xi_{eq,dowel}$), we can estimate the final value of the joint damping. Joints can be seen as a set of non-linear springs that work in parallel and are subject to the same slip if they have the same distance from the centre of instant rotation. The damping value is calculated as a function of the slip (or ductility μ) reached in each dowel. From the equivalent viscous damping of the connections we can evaluate the structure EVD via simple direct assessment, assuming the same inelastic deformed shape of the system used in design displacement evaluation.

5. PROPOSED DESIGN PARAMETERS FOR DIRECT-DBD PROCEDURE

5.1. Portal frame system

The portal structure hinged at the base, presented in Figure 1(a), has semi-rigid ductile joints arranged with dowels on concentric rings. The design displacement (Δ_d) can be estimated as the sum of the inelastic displacement for rigid rotation of the columns, following yield of the connectors (Δ_j), and the elastic deformation of the portal (Δ_s) calculated assuming rigid beam-to-column joints (Eqn. 5.1):

$$\Delta_d = \Delta_j + \Delta_s = \delta_u \frac{\theta\gamma\beta}{1+1/\theta\gamma\beta} + \frac{H_c}{2000}(\theta+1)\gamma \tag{5.1}$$

In Eqn. 5.1 some geometrical and mechanical parameters of the structure are replaced with non-dimensional coefficients, while the others are known at the start of the design process. Parameters that control the design displacement formula of the portal (Δ_d) are basically the ultimate slip of the dowel (δ_u), the height of the portal frame (H_c), the portal aspect ratio ($\theta = H_c/L$), the ratio between the length of the portal and the height of the beam cross-section ($\gamma = L/h$), and the ratio between the cross-section of the beam and the external radius of the joint ($\beta = h/r_{ext}$). The dimensionless parameter β ($= h/r_{ext}$) can be estimated in the design process, while H_c and θ are obviously known at the design stage. Lastly, the non-dimensional geometric parameter under normal design assumptions, γ($=L/h$), is expected to be between 10 and 15. A complete discussion of Eqn. 5.1 implementation can be found in Zonta et al. (2011) and in Loss et al. (2012).

The Equivalent Viscous Damping of the structure (EVD) is estimated via Eqn. 5.2, assuming that all the dissipation capacity is concentrated in the beam-to-column joints of the portal frame:

$$\xi_{eq} = \frac{E_{hyst}^{(1/2cycle)}}{2\pi E_{sto}} = \frac{2E_{dis,J}^{(1/2cycle)}}{2\pi(F\Delta_d/2)} = \frac{2(\xi_{eq,J}\, 2\pi M_u \Phi_u)^{(1/2cycle)}}{2\pi 2(V_J \Delta_d)/2} = \xi_{eq,J}\,/(1+\Delta_s/\Delta_j) \tag{5.2}$$

where $\xi_{eq,J}$ is the equivalent viscous damping of the moment-resisting connection at the ultimate limit state [More details are reported in Loss (2011)]. Known the equivalent viscous damping of the single dowel, the $\xi_{eq,J}$ of a joint with two concentric rings of fasteners can be evaluated with Eqn. 5.3:

$$\xi_{eq,J} = \frac{(E_{hyst,J})^{1/2cycle}}{2\pi E_{sto,J}} = \frac{(E_{dis,J})^{1/2cycle}}{2\pi M_u \Phi_u/2} = \frac{2\pi F_y \delta_u/2(n_{ext}\xi_{eq,dowel,ext} + n_{\text{int}} r_{\text{int}}/r_{ext}\xi_{eq,dowel,\text{int}})}{2\pi M_u \Phi_u/2} \tag{5.3}$$

The terms M_u and Φ_u, ultimate bending moment and ultimate rotation of the connection, can be calculated analytically with models proposed in Loss (2011), n_{ext}, n_{int}, r_{ext} and r_{int} are the numbers of connectors and the radii of the external and internal dowel rings respectively, while F_y and δ_u are as previously defined.

5.3. Wood frame system

Wood frame structures with platform frame technology as in Figure 1(b) are common in residential buildings, in which the seismic response is directly related to the shear wall behavior. From the capacity of the single shear wall panel we can obtain the capacity of the whole structure. This procedure is known as "segmented shear wall design approach" (Prion, 2003). The formulation of the design displacement as proposed here is consistent with the expected response of the shear wall observed experimentally (Prion, 2003), in the inelastic deformation range, known as "shear type mechanism". The shear mechanism can be developed when anchoring devices are provided at the ends of each wall to prevent overturning and sliding on the floors at each level. Assuming the shear wall panel to be a cantilever element, the design displacement (Δ_d) of the shear wall is the sum of the elastic bending displacement (Δ_b) of members, of the panel shear displacement (Δ_v) and of non-linear displacement induced by the panel-to-frame connections (Δ_n). The contributions of deformation due to bending and shear stresses are obtained using the virtual unit load method (Eqns. 5.4(a) and (b)).

$$\Delta_b = 2F_w h_w^{\ 3} /(3EA_s b_w^{\ 2}), \quad \Delta_v = F_w h_w /(G_p b_w t_p) \qquad (5.4\text{ (a), (b)})$$

where E is the modulus of elasticity of the framed timber elements, A_s is the cross-section area of studs, h_w is the wall panel height; b_w is the wall panel width and F_w is the force acting at the top of wall (the maximum resisting force $F_{V,w}$), G_p is the shear modulus for the sheathing panels and t_p is the effective thickness of the sheathing. In the elastic range the contribution of the displacement due to nail slip, Δ_n, can be calculated by analytical models proposed by these authors: Judd and Fonseca (2006), Yasumura (2000) or, more simply, by applying the relation provided in Chapter 5 of NZS 3603 (New Zealand Standard, 1993). We propose here the extension of the Judd and Fonseca model to the plastic phase, by simply replacing the elastic slip of the dowel with its ultimate slip δ_u (Eqn. 5.5). This is true when the ductile failure mode of the dowel is ensured as explained previously, in which case the hysteretic curves of Figure 2(b) can be assumed for the dowels.

$$\Delta_n = \delta_u \left(h_w / l_w\right) \cdot \left(2\sqrt{2}\cos\phi\right) / \left(\cos 2\phi\right) \qquad (5.5)$$

Where l_w is the diagonal length of the panel, h_w is the wall height, $\phi=\pi/4\text{-}arctan\ (b_w/h_w)$, b_w is the wall panel width and δ_u is the ultimate slip of the dowels. The shear wall displacement capacity (Δ_d) is strictly related to the wall arrangement and to the type of nails in use (strength of the steel wire, diameter and length), as well as to the aspect ratio of the sheathing panels h_w/b_w.

According to the strain model assumed for estimation of the design displacement, we can calculate the equivalent viscous damping as a simple linear combination of all the connectors that define the capacity of the single framed panel. From the analytical model employed for each shear wall we have a uniform slip and a force that are equally divided over every nail. In the case of a shear wall the expression of the equivalent viscous damping specializes in Eqn. 5.6, as a weighted average of the dissipation capacity of each structural element at the ultimate limit state:

$$\xi_{eq,wall} = \left(\sum\nolimits_{i=1}^{N_{nails}} \xi_{eq,i,dowel}(\mu_{\delta i}) F_i \delta_{u,i}\right) / \left(\sum\nolimits_{i=1}^{N_{nails}} F_i \delta_{u,i}\right) = \xi_{eq,dowel}(\mu_\delta) \qquad (5.6)$$

where F_i, $\delta_{u,i}$, $\mu_{\delta i}$ and $\xi_{eq,i,dowel}$ are respectively the design force, the ultimate slip, the ultimate slip ductility and the equivalent viscous damping of the i^{th} nail in the shear wall, while N_{nails} is the number of “resisting” connectors. Finally, the equivalent viscous damping of shear walls (ξ_{eq}) is equal to the equivalent viscous damping of a single nail ($\xi_{eq,dowel}$), evaluated at ultimate slip δ_u.

6. CONCLUSIONS

We introduce a general Displacement-Based Design methodology for wooden structures with dowelled connections. The baseline idea is that in timber structures, ductile resources and dissipative capacity is typically concentrated at the connections, regardless of the specific structural system. Thus, we can ensure the desired seismic behavior of the structure by appropriate design of the joints. Using limit equilibrium methods, we derive analytical expressions of target displacement and equivalent viscous damping, the design parameters required by DBD, for wood frame buildings and for portal frames with moment-resisting joints. Extending this general method to other structural timber systems is an attractive possibility. However, its practical implementation in design codes requires further investigation and should be properly supported by experimental validation.

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