DIRECT DISPLACEMENT-BASED DESIGN OF GLULAM TIMBER FRAME BUILDINGS

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ABSTRACT:
The work we present here aims at defining a direct Displacement Based Design (DBD) methodology that specifically applies to warehouses or commercial buildings, based on glued laminated timber portal frames. The case study investigated is an industrial wood-framed warehouse with two-hinged frames where the post-beam connections are semi-rigid moment-resisting joints with dowel-type fasteners. A necessary condition for applying DBD is that it be possible to estimate a priori (i) the target displacement of the portal and (ii) the equivalent damping ratio of the structure at the ultimate capacity. The general assumption is that the displacement capacity of the building mainly depends on single joint behavior and only to a smaller extent on the size of structural members. This observation lets us define a practical expression for calculation of the target displacement with only the dimensions of members and connections. Using pushover non-linear analyses, we demonstrated that the expression provides prior values of target displacement that are close to those obtained *a posteriori* using a much more refined model that takes account of the exact geometry of members and connections. The comparison with the results of Eurocode 8 shows that the DBD method potentially can overcome some of the simplifications that a Force Based Design (FBD) method necessarily leads to.

KEYWORDS:
Displacement Based Design, Timber Structures, Portal Frame, Moment Resisting Joint, Target Displacement, Equivalent Damping

1. INTRODUCTION

The work presented in this paper aims at defining a direct Displacement Based Design (DBD) method that specifically applies to heavy timber structures, including warehouses or commercial buildings, based on glued laminated timber portal frames. We will refer to a specific case study, a warehouse described in detail in Section 2, although the general concept can be easily extended to most hyperstatic portal frame buildings. We will also refer to dowel connections, as these possibly represent the type of connection most extensively used in glulam construction technology. The load-carrying capacity of bolted wood connections can be easily estimated using the well-established Johansen theory (Johansen, 1949), sometimes referred to as *European Yield Theory*. However, Johansen's model provides only the strength of the connection, but no information on load-slip relation, ductile capacity or energy dissipation of the fastener. More generally, no commonly accepted method for predicting precisely the ductile capacity of a fastened connection can be found in the scientific literature, although there are many works that deserve to be considered. The problem is closely connected to the large number of parameters necessary to characterize the general behavior of the fastening, including the angle between the load direction and the grain, the direction of the load (tension or compression), the ratio between the length and the diameter of the fasteners and obviously the number of shear planes involved. Moreover, fastener behavior is usually represented by a bi-linear law, which implies univocal definition of the yield point and consequently the ductility. However, the concepts of yield point and ductility are not well-defined (see a review on this topic in appendix B of Dolan (1994). Under cyclic loads, bolted connections exhibit load-slip hysteresis loops characterized by a pinch effect. This behavior is due to the steel fasteners that embed in the wood during the load action, hence slackening the joint. The same mechanism implies lateral stiffness degradation and reduction of energy dissipation after each cycle. According to Dolan and Gutshall (1997), the load history affects the behavior only if the load magnitude is below 38 % of capacity for bolts loaded parallel to the grain, and below 75 % for nails. In the case of reversed cyclic load, the yield load, the stiffness and the ductility are lower than in the monotonic case. However, the load capacity of the connection increases or remains the same. Daneff et al. (1996) conducted extensive tests on bolted connections, comparing the monotonic load-slip response to the envelope response in a reversed cyclic test. They observed that the slip at the maximum load is consistently higher in the case of cyclic tests and that the energy dissipated by a connection subjected to monotonic load is 5 to 17 times smaller than the cumulative energy dissipation in cyclic regime.

2. DESCRIPTION OF THE CASE STUDY

The structural concept selected as a case study is a typical glulam warehouse built completely, as to structural members, of glulam timber type GL24h (CEN, 2000). The building is a single-story structure, where all the masses are placed approximately at the same height. The bearing structure is regular and has 5 portal frames, equally spaced at pitch 6.5m. The main geometrical dimensions of the portal are shown in Figure 1. Each portal frame consists of a continuous curved beam with two 4.79 m high columns, hinged at the base to the foundation. The columns have spaced elements connected with packs. The beam is an apparent double pitched cambered beam 15.5 m long, with a slope α=10.2°. The secondary structure of the roof consists of six 0.18 by 0.46 m beams and a ridge beam of identical cross-section, all in glulam timber. Plywood panels are inserted between the columns serving as shear walls, while the roof in-plane bracing is provided by the timber boarding. The building was dimensioned according to Eurocodes: in detail we assumed a service class 1, according to Eurocode 5. We also assumed the building to be located in a seismic zone with a design ground acceleration a_g=0.35g, on ground type C. The total seismic mass of a portal is m=19775 kg and is considered as concentrated at the roof level. In this paper the analysis will address only the in-frame direction.

The beam to column connection is a moment resisting joint fastened with dowels located on two concentric circles. The dowels work as double shear planes timber to timber connections. Assuming the members to be rigid, the elastic rotational stiffness of the joint K_φ, i.e. the constant that relates moment M to rotation $\Delta\varphi$, can be calculated as

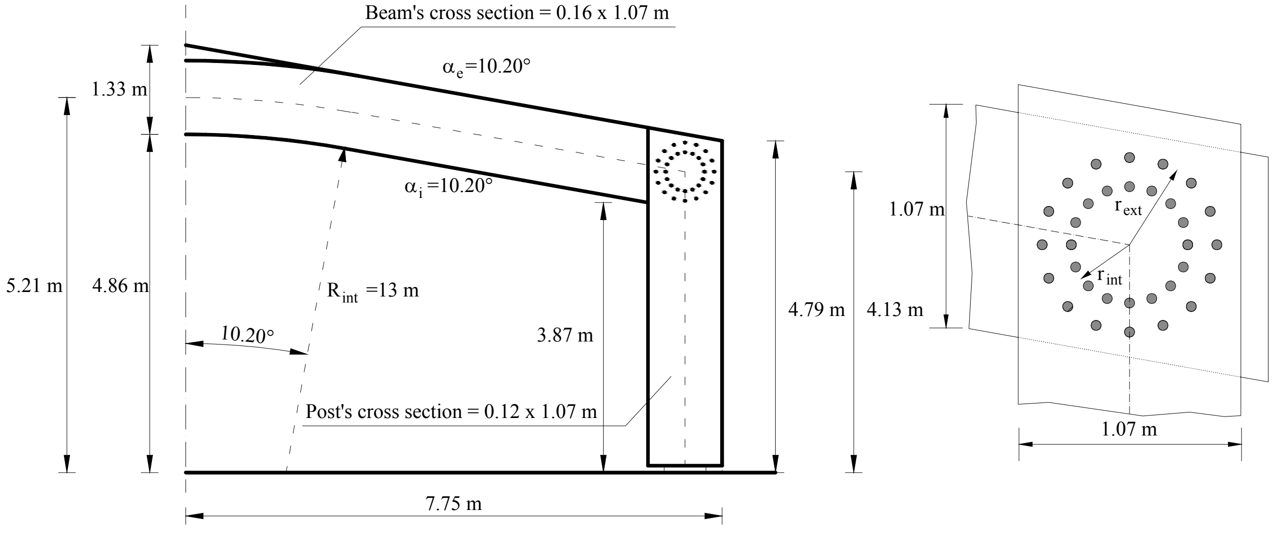

Figure 1 Geometric features of the case study and detail of the connection (Piazza et al., 2005)

Table 1 Geometry and performances of moment resisting joint

		Design I	*Design II*
Fastener diameter	d [mm]	12	16
Internal radius	r_{int} [mm]	425	385
Number of internal fasteners	n_{int} [Adim]	44	30
External radius	r_{ext} [mm]	485	465
Number of external fasteners	n_{ext} [Adim]	50	36
Rotational stiffness (elastic)	$K_{\varphi es}$ [kNm]	175194	144961
Rotational stiffness (ultimate state)	$K_{\varphi u}$ [kNm]	116796	96641
Resisting moment	M_{Rd} [kNm]	730	820

$$K_{\varphi} = \sum_{i}^{n} K_i r_i^{2} \tag{2.1}$$

where K_i is the elastic slip modulus of the i-th single dowel and r_i is its distance from the geometrical centre of the fasteners. Within the scope of this work, the slip modulus of the single fastener can be conventionally assumed equal to that given by Eurocode 5. Table 1 illustrates how we can obtain almost identical static performance of the connection by arranging in similar configuration either dowels with diameter d=12 mm (*Design I*) or of diameter d=16 mm (*Design II*): indeed, both the designs are equally well assessed with respect to the maximum shear action at the most critical dowel, under static loads. However, the base shear calculated according to Eurocode 8 is very sensitive to dowel diameter. Eurocode 8 states specifications for defining the most appropriate ductility design class, L M or H, based on the joint detailing.

Design in ductility class H is allowed when the fastener diameter d does not exceed 12 mm and the thickness t of the connected members is not smaller than 10 times d: in this case a value of q=4 can be assumed. By contrast, when d is greater than 12 mm and t is less than 8 times d, the structure should be classified in ductility class L, and a value as low as q=1.5 must be taken. It is immediately verified that in our case, being t=120 mm, the first condition is fulfilled using dowels with diameter d=12 mm as in *Design I*; the latter when d=16 mm as in *Design II*. This results in a completely different base shear force calculated in the two scenarios, equal to 53kN and 142 kN for *Design I* and *Design II* respectively.

3. FORMULATION OF A DBD METHOD FOR TIMBER PORTAL STRUCTURES

In the following sections we introduce a Direct DBD formulation that specifically applies to timber portal frame-structures; in essence, this is an extension to glulam frames of the general methodology developed by Priestley (Priestley, 2000) for concrete and steel structures.

3.1 Target displacement

A necessary condition for applying the DBD method is that it be somehow possible to estimate *a priori* the target displacement Δ_d of the portal, regardless of the geometrical dimensions of members and connections. In the case of serviceability limit states, this assumption is most likely true, as the limit value of displacement is typically expressed in terms of maximum acceptable drift of the building. However, for a collapse limit state, this estimate might not be so obvious, because the ultimate capacity generally depends on member and joint characteristics, which are unknown at the design phase. Nevertheless, below we will show that, in the case of a two-hinged timber portal building, it is possible to obtain a sufficiently accurate estimate of Δ_d using a fairly simple formulation.

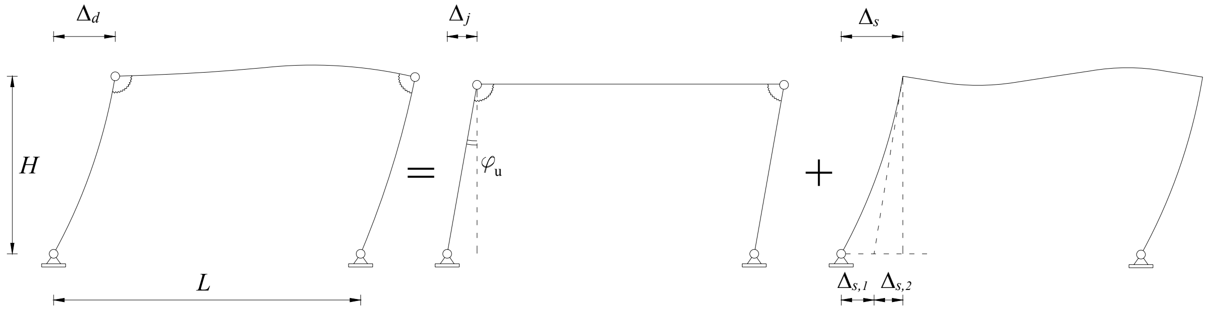

Figure 2 Model for estimating the target displacement

First of all we can assume that, if the members are appropriately dimensioned, at ultimate limit state all of the inelastic displacement is due to the rotation of joints, while the timber member behavior is considered elastic. Hence, we can estimate the total target capacity Δ_d as the sum of the displacement Δ_j due to rigid rotation of columns, as a consequence of joint yield, and the elastic deformation Δ_s of the portal, calculated assuming the joints to be rigid (Figure 2).

Evidently we have $\Delta_j=\varphi_u H$, where φ_u is the ultimate rotation of a joint and H the column height. The assumption of rigid members basically means that the rotation φ will produce, in each fastener, slips δ_i proportional to their distance from the joint centre of rotation C. Therefore, the ultimate limit state equation reads $\delta_u=r_{max}\varphi_u$, being r_{max} the maximum distance between the rotation centre C and the most critical dowel. For simplicity, we will assume here that all of the fasteners are located in a single circular pattern, of radius r. As the connection is subject to both shear V and moment M, the rotation centre C does not coincide with the geometrical center O of this circle. In general, the direction of shear stress depends on the ratio between seismic loads and gravitational loads: if we further assume that the gravitational loads can be neglected in this calculation, then the relation $M=VH$ must hold. With some approximation we can further assume the ultimate distribution of forces on dowels to be proportional to that in the elastic state. Hence, moment and shear components of dowel reaction can be calculated by: $F_M=M/nr$ and $F_V=V/n$, where n is the total number of dowels in the connection. Using simple geometrical relations, with reference to Figure 3, and keeping in mind that $M=VH,$ we obtain the following estimate of r_{max}:

$$\Delta r = r\frac{F_V}{F_M} = r^2\frac{V}{M} = \frac{r^2}{H};\; r_{\max} = r + \Delta r = r\left(1+\frac{r}{H}\right) \qquad (3.1a,b)$$

After some formal steps, which we skip here for sake of brevity, the displacement Δ_j can be written as:

$$\Delta_j = \delta_u \frac{\theta\gamma\beta}{1+1/\theta\gamma\beta} \qquad (3.2)$$

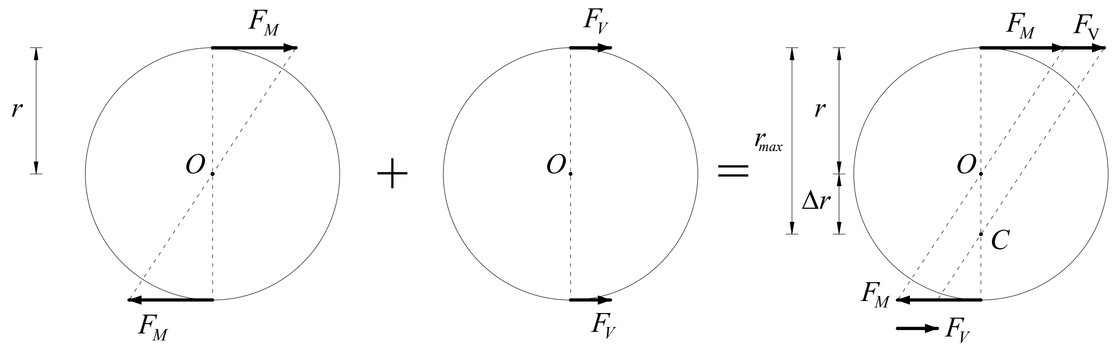

Figure 3 Moment and shear components of dowel stresses

where h is the geometrical height of the section of the column; $\theta = H/L$ is the aspect ratio of the building; $\gamma = L/h$ and $\beta = h/r$. Displacement Δ_s, is simply given by the elastic relation:

$$\Delta_s = \Delta_{s,1} + \Delta_{s,2} = \frac{MH^2}{3EJ_c} + \frac{M}{6EJ_b} LH \tag{3.3}$$

where E is the timber elastic modulus, and J_b and J_c are the moments of inertia of the beam and the column, respectively. Equation 3.3 depends on the member characteristics, but can be rearranged assuming some simplifications. If the portal is appropriately designed, the beam yielding moment $M_{R,c}$ should be greater than the joint ultimate resisting moment M_u. In other words, it should be $M_u = \alpha M_{R,c}$, where α is an over-strength factor. Thus, Equation 3.3 can be rewritten as:

$$\Delta_s = \frac{2}{3\alpha}\left(\theta \frac{J_b}{J_c} + \frac{1}{2}\right) H\gamma\varepsilon_y \tag{3.4}$$

where ε_y is the yield strain of timber and $\theta = H/L$ is the aspect ratio of the portal. Reasonable values for yield strain and over-strength factor are $\varepsilon_y = 0.002$ and $\alpha = 1.2$-1.3, respectively. Also for a typical warehouse building we might expect that $J_b/J_c \cong b_b/b_c = 1.0$-$0.5$. Using the conservative value of $J_b/J_c = 0.5$, the expression of Δ_s reduces to:

$$\Delta_s \cong \frac{H}{2000}\gamma(\theta + 1) \tag{3.5}$$

In summary, the ultimate target displacement can be estimated using the following equation:

$$\Delta_d = \delta_u \frac{\theta\gamma\beta}{1 + 1/\theta\gamma\beta} + \frac{H}{2000}(\theta + 1)\gamma \tag{3.6}$$

From this formula we can note that the parameters that control the displacement capacity of the portal are basically the ultimate slip of the dowel δ_u, the height of the building H, the portal aspect ratio $\theta = H/L$ and the ratio $\gamma = L/h$. Also the ratio $\beta = h/r$ influences the displacement capacity, but only to a minor extent. H and θ are obviously known at the design stage, while reasonable assumptions can be made for prior estimation of the other geometrical parameters. In fact, $\beta = h/r$ is necessarily greater than 2 and most typically somewhere between 2.5 and 3; also, under normal design assumptions, $\gamma = L/h$ is expected to be between 10 and 15.

3.2 Equivalent damping ratio

The next step is to define the design damping ξ_d, i.e. the equivalent viscous damping of the structure at the design displacement. In general, it is commonly accepted to see ξ_d as the sum of a constant viscous component ξ_0 and a hysteretic component ξ_{hyst}, which increases with displacement capacity Δ_d. It is also customary to express the hysteretic component of damping as a function of structural ductility μ. An often used general formulation of this kind is that suggested by Priestley (2003):

$$\xi_d = \xi_0 + \xi_{hyst} = \xi_0 + \frac{a}{\pi}\left(1 - \frac{1}{\mu^{0.5}}\right) \tag{3.7}$$

where a and ξ_0 are constants that depend on the structural material and technology. This expression is commonly used for steel and concrete structures, while almost no application is found in the technical literature in the case of timber. Overall, we must remark how for wooden structures the phenomenon of stiffness degradation in cyclic loads makes the application of this equation unavoidably approximate.

In his Master Thesis, Sartori (2008) proposes values for parameter a and ξ_0 calibrated on the experimental outcomes of tests on full-scale fastened glulam joints, carried out according to UNI EN 12512 (2006). This load protocol includes a sequence of reversed cycles of increasing amplitude and is the standard used in Europe to characterize the response of fastened joints. The general assumption is that the energy dissipation of a glulam frame is mostly due to plastic deformation of its connections. We must remember at this point that, for fastened connections, the hysteretic dissipation is mostly due to the steel dowels that embed in the wood during the load action. Because this mechanism implies reduction of energy dissipation after each cycle, the total amount of energy dissipated is strictly dependent on the load protocol: therefore, the relationship between equivalent damping and ductility is not unique. Taking account of this issue, Sartori defined three different types of damping-to-ductility curves. The first curve is that obtained under the same load protocol used in the experimental tests. The other two represent the upper- and lower-bond curves, theoretical expected in the case of monotonically increasing cyclic load and constant amplitude cyclic load, respectively. For the purpose of this work, the use of the first type of curve seems the most appropriate to reproduce the effect of a real earthquake. In this case, the numerical values of the dissipation parameters are: a = 60 and ξ_0 = 10% for *Design I*; a = 54 and ξ_0 = 8% for *Design II*.

3.3 Design base shear force

Once defined the target displacement Δ_d and the corresponding equivalent damping ξ_d, the equivalent period of the structure T_{eq} at the ultimate displacement is obtained by solving the following limit state equation:

$$\Delta_d = S_d\left(T_{eq}, \xi_d\right) \tag{3.8}$$

where S_d represents the expression of the elastic displacement spectra as a function of period and damping rate. Equation 3.8 can be solved either numerically or graphically (Figure 4b), on the basis of the representation of the response spectra provided, for example, by Eurocode 8. The equivalent stiffness k_{eq} and the design base shear force of the structure F_b can be obtained by the following expressions:

$$k_{eq} = \frac{4\pi^2 m_{eff}}{T_{eq}^2} \tag{3.9}$$

$$F_b = k_{eq}\Delta_d \tag{3.10}$$

where m_{eff} is the effective mass associated to the first mode shape.

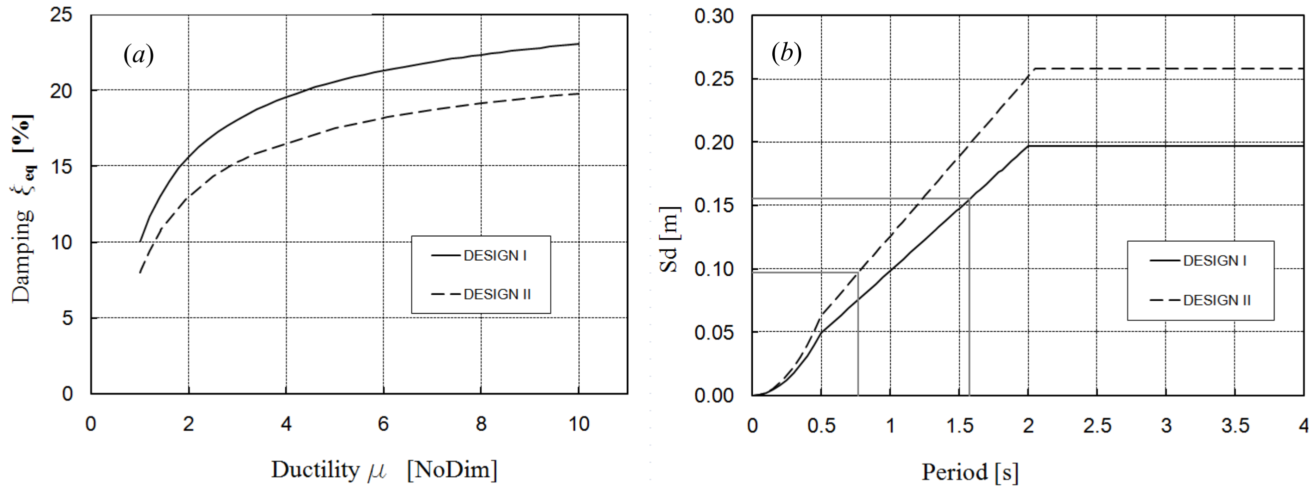

Figure 4 Equivalent damping vs. ductility relationship (*a*) and design displacement spectra (*b*)

Table 2 Structure displacement performance and Base Shear Forces obtained with different methods

		Design I	***Design II***
Dowel load-carrying capacity	F_y [kN]	17	29
Dowel elastic slip modulus	K_ϕ [kN m^{-1}]	5926	7902
Dowel ultimate slip	δ_u [mm]	17.4	7.8
Design displacement (Eqn. 3.6)	Δ_d [mm]	182	109
Structure ductility capacity	μ	6	2.10
Equivalent viscous damping (Eqn. 3.7)	ξ_d	21.2%	13.2%
Ultimate displacement (via pushover analysis)	Δ_d [mm]	205	132
Ultimate rotation	Φ_u [rad]	0.0317	0.0149
Yielding rotation	Φ_y [rad]	0.0053	0.0071
Base Shear Force by Eurocode 8	$F_{b,EC8}$ [kN]	53	142
Base Shear Force by Direct DBD	$F_{b,DDBD}$ [kN]	56.4	130.2

4. SAMPLE APPLICATION

In this section we will apply the Direct DBD method described above to the case study, in order to compare the results with those obtained using Eurocode 8. To do so, we must verify that the parameters chosen in the DBD approach are consistent with the implicit assumptions of Eurocode 8. As seen before, Eurocode 8 assigns ductility class H and a behavior factor $q=4$ to the frame structure when the connections are fastened with d=12 mm dowels, as in *Design I*. The same code assumes that in this case the dissipative zones shall be able to deform plastically for at least three fully reversed cycles, at a static ductility ratio of 6, with less than a 20% reduction of their strength. Therefore we may conservatively assume that the ultimate slip of the dowel δ_u is at least 6 times the conventional yield slip δ_y. In turn, δ_y is calculated as the ratio between the yield strength F_y and the slip modulus K_ϕ, both of these given by *Eurocode 5*.

Within the scope of this analysis, average values of material strengths appear to be more appropriate than design values for calculating F_y. Mean values have been derived from characteristic values assuming normal distribution and coefficients of variation of 3% and 18% for steel and timber respectively. The resulting values of target displacements have been verified through a pushover analysis on a more refined non-linear model of the portal. The model accounts for the exact geometry of members and connections. To define an appropriate value of equivalent damping the ratio of ductile capacity of the structure μ was used (ratio of ultimate rotation and yielding rotation) and the equivalent viscous damping. Based on the resulting values of equivalent damping, Eurocode 8 provides displacement response spectra of the type shown in Figure 4b. The equivalent elastic period of the structure T_{eq}, based on the target displacement considered, can be read directly from this set of design displacement spectra. The design base shear forces F_b are then obtained using Equations 3.10. Table 2 summarizes the values of base shear obtained with different methods: as expected, the design base shear is very sensitive to the dissipation capacity of the structure.

5. CONCLUSIONS

An application of Priestley's DBD method to a two-hinged glulam timber frame structure has been presented. A necessary condition for applying this method is that it be possible to estimate *a priori* (i) the target displacement Δ_d of the portal and (ii) the equivalent damping ratio of the structure at the ultimate capacity. Both of these parameters can be derived from the geometrical dimensions of members and connections. We proposed a practical expression that allows us to calculate the ultimate target displacement and the equivalent viscous damping, simply and reliably. The expressions are very easy to use, and can be extended to a broader range of structural schemes. Using pushover non-linear analyses, we demonstrated that the expression provides prior values of target displacement that are close to those obtained *a posteriori* using a much more refined model, which takes account of the exact geometry of members and connections. On the other hand, the simulations show that the design base shear is very sensitive to the dissipation capacity of the joints represented by equivalent viscous damping. This parameter is investigated through laboratory experiments and there is a proposal for a simple expression to determine ξ_d. The comparison with the results of Eurocode 8 shows that the DBD method potentially can overcome some of the simplifications that a Force Based Design (FBD) method necessarily leads to.

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