16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 2019
Registration Code: S-W1464371725

A NEW SELF-CENTERING DISSIPATIVE CONECTION FOR GLUED LAMINATED TIMBER FRAMES

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Abstract

Chile is one of the most important timber producers in the world; however, the use of timber as a structural material represents only 19% of 2015 nationwide. Within this percentage, the number of projects in glued laminated timber (Glulam) is even lower. Although in the last decade, the number of projects using glulam is increased, it does not represent a significant share in the Chilean market yet. The relatively small use of timber in Chile is explained by the limited knowledge and experience in large scale projects, its potential, and the lack of connection systems with a verified efficiency and efficacy under seismic actions.

The research presented here proposes an innovative type of connection for Glulam timber structures that is capable of dissipating energy, reducing stresses during an earthquake, and that is also capable of recovering its initial configuration after earthquakes. The connection proposed herein, links the beam with the column and the column with the foundation, behaving as rigid connections under axial and shear loading, and providing stiffness and energy dissipation under bending moment. The connection gap remains rigidly closed while the magnitude of the internal bending moment remains below a characteristic bending threshold. When this bending threshold is exceeded, the connection gab opens, dissipating energy with a minimum of increment of internal stress in the Glulam elements, due to the relatively low plastic stiffness provided by the connection. In this article, the geometry of the proposed connection is shown and the constitutive law that describes their behavior is defined. The results of a parametric push-over analysis and time-history analysis are shown, comparing the performance on Glulam structures with and without the connection.

Keywords: self-centering connection, energy dissipation, glued laminated timber.

1. Introduction

Chile is a highly seismic country, it has had three major earthquakes: 1960, Valdivia (Mw=9.5); 1868, Arica (Mw=9.0) and 2010, Maule (Mw=8.8). They are the first, sixth and eighth earthquakes of highest magnitude recorded worldwide [1]. Due to the above, the seismic philosophy in Chile is based on the inherent ductility of the constituent materials and structural system of a building. The latter is directly related to the capacity or energy dissipation of the structure and the seismic behavior of the materials and structural system used [2]. In the Chilean seismic code, the above is characterized by a response reduction factor "R", which is related to the capacity of the structure to dissipate energy under a condition of maximum resistance. In the Chilean seismic design code, NCh 433 Of.96, the factor R used for reinforced concrete and steel frames is R=7, while for wood structures, the factor used is R=5.5. In the case of concrete and steel, the value of the R factor is related to the ductility of the steel in tension, because in both cases the failure is required to be controlled by this condition. In wood frames the R factor is related to the dissipation of energy in the joints, where damage is expected to occur. However, there are no predictive models that quantify the energy dissipated in the commercially available joints, because the mechanisms are diverse and complex.

Chile is one of the biggest timber producers worldwide, however, the use of wood as a structural material only reaches 19% in the country. Within this percentage, the number of projects in Glulam is even lower [3]. Although in the last decade the number of projects using glulam has been increasing, it does not yet represent a significant share in the Chilean market. The problem of using wood in Chile, lies in the limited knowledge of the material and the lack of commercially available joint systems with a verified efficiency in this type of wood designs [4].

The proposal presented in this article of a type of joint for this material comes up when considering the current problems regarding the use of wood in earthquake-resistant designs. This joint is capable of dissipating energy in a seismic event, reducing the stresses in the structure and it also has the capacity to recover its initial configuration after the earthquake. Additionally, the energy dissipated and the ductility developed for the structure is easy to be quantified in the proposed joint.

The joint proposed allows linking beams with columns –both made of glulam– and columns with the base or foundation, behaving as rigid joints for axial and shear loadings, and providing stiffness and dissipation of energy in bending. In this research the architecture of the proposed structural joint is shown, defining the constitutive law governing their behavior, depending on its design parameters. The results of a parametric push-over analysis performed in wood frames are presented. Two structural frames were studied, one with rigid joints and the other with the proposed joints in the junction of beams and columns and in the base of the columns. From the results of this analysis optimal combinations of the design parameters of the joints were obtained, considering an objective function for minimizing the internal stresses in the wood. The optimal parameters determined for the joints were finally used to define the geometric and mechanical properties corresponding to a real design of the joint. A five stories frame with optimal parameters for their plastic hinges is finally used in a time-history analysis. The above results were compared with the response of the same frame but with rigid connections, showing the efficiency of proposed joints.

2. Description of the structure studied

The types of structure used in this study are frames of one to five stories high, with plastic hinges in the beam-column joints and column-base. One of these frames is shown in Fig. 1, highlighting in red the hinged connections at the base of the columns and at the beam-column joints.

The frame geometry corresponds to that typically used in Chile, and the dimensions are shown in Fig.2. Each frame holds a slab of reinforced concrete of 5m x 5m x 0.15m with a specific weight γ=23.5 kN/m^3.

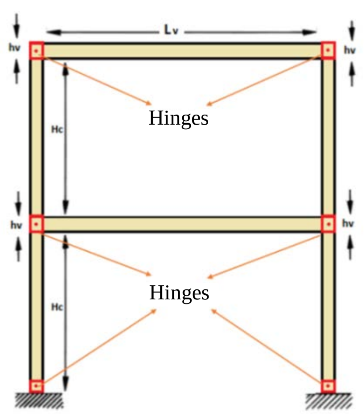

Fig. 1 – Frame with frictional self-centering hinges at the beam-column and column-base joints.

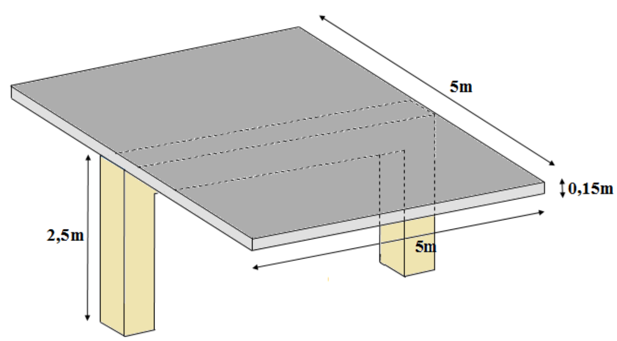

Fig. 2 – Geometry of the basic constituent of the frame studied.

The frame is made of glulam with horizontal lamination of pine (*Pinus radiata*) with structural grade B, according to standard NCh 2165. The admissible stresses and the modulus of elasticity for glulam are presented in table 1.

Table 1 – Properties of the glulam *pinus radiata*, structural grade B, according NCh 2165.

Admissible stress in Flexion (MPa)	Admissible stress in Parallel Compression (MPa)	Young Modulus (MPa)
19	13	8000

2.1 Determination of scantlings Glulam by number of stories of the frame.

To find the scantlings that allow an efficient use of laminated wood elements according to the number of floors of the building, it is necessary to conduct a seismic design of the elements, according to the Chilean code NCh433, using static analysis.

To calculate the shear stress produced by seismic action in the basal level of the building, Eq.(1) of the NCh433 code is used:

$$Q_0 = CIP \tag{1}$$

Where:

C: Seismic coefficient –Defined in 6.2.3.1 and 6.2.7 in NCh433–.

I: Coefficient related to the importance of the building –Table 6.1, NCh433–.

P: Total weight of the building above the baseline.

To determine the parameters related to the calculation of Q_0, it is necessary to define certain characteristics of the structure and its location. For this investigation it is considered that the structure is of category II (housing) and that it is located in the city of Concepcion, Chile. In table 2 the factors used to determine the base shear load are shown. The shear load for a structural frame of one floor is 20500 N.

Table 2 – Parameters involved for determining Q_0.

Coefficient	Parameters involved	Code	Value
C	Reduction factor (R)	NCh 433	5.5
	Type of Soil (S)	NCh 433 & D.S. 61	1
	Effective Acceleration (A_0)	NCh 433	3
I	Category of the building	NCh 433	1
P	Concrete slab	-	88125 N
	Overload non structural elements	NCh 1537	25000 N
	Overload of use reduced (30%)	NCh 1537 & NCh 433	0.3·50000 N

With the shear force at the base, the maximum bending moments are obtained in the elements, with which the internal stresses in the beams and columns can be determined, from the Navier equation.

$$\sigma_t = \frac{M_{max}}{W_x} \tag{2}$$

Where:

M_{max}: Maximum bending moment in the element.
W_x: Bending resistant modulus of the cross section.

Taking the admissible stress of the material as a known data (Table 1), a demand factor of the cross section, f, is defined, seeking that f be close to 1 to ensure a more efficient use of the wood elements. In Table 3, the cross section of the elements are presented for both, beams and columns, that fulfill the maximum roof displacements according to NCh433, without overcoming the admissible stresses of the glulam. Carried out the above, each frame is defined as the number of stories and cross sections of its constituent elements.

Table 3 – Sections for columns and beams by number of stories.

			Columns			Beams		
N° of floors	**Q_0 (N)**	**u (mm)**	**f (%)**	**Width (mm)**	**High (mm)**	**f (%)**	**Width (mm)**	**High (mm)**
1	20500	4.9	30	305	305	21	155	245
2	41000	10.0	29	415	415	23	215	335
3	61500	14.8	30	455	455	26	265	425
4	82000	20.0	31	495	495	27	275	475
5	102500	25.0	30	565	565	28	295	535

2.2 Model of the self-centering plastic hinge

The joining device, which is described schematically in Fig.3, can rotate around two axes located at opposite edges of the element to be joined. Inside this element there is a pre-stressed cable that provides a centering bending moment that is able to close the joint after the external loads stop acting. Energy dissipation occurs by friction due to sliding between the parts joined at the opposite edge to the instant rotation axis. All these elements are within a housing, bolted to the column or beam.

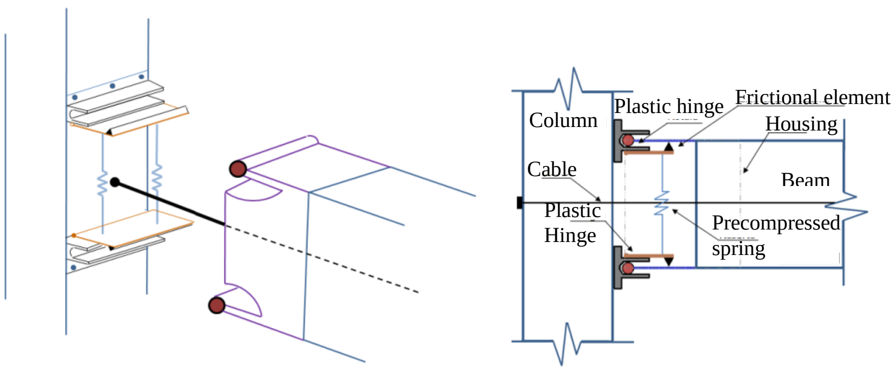

Fig. 3 – Conceptual scheme of the plastic hinge.

The normal force responsible for the friction force present in the model is provided by a flexible element, previously deformed according to the indications of design.

The pre-stressed cable that provides the self-centering capability is located inside the beam or column, passing along its longitudinal axis in its full extent. In beam-column joints, this is located from the outer edge of one column, passing completely through the beam, until the outside edge of the opposite column. In base-column connections, the cable is fixed on the base and passes through the hinge and the column in full height.

Cable tension and friction force parameters were included from the equilibrium of the plastic hinge, according to Eqs. (3) and (4).

$$M_{f,c}^{(S)} = (T_{0c}/2 + F_{rc}) \cdot b_c \tag{3}$$

$$M_{f,v}^{(S)} = (T_{0v}/2 + F_{rv}) \cdot h_v \tag{4}$$

Where:

$M_{f,c}^{(S)}$: Plastic bending moment of the hinges, at the base of the columns.
$M_{f,v}^{(S)}$: Plastic bending moment of the hinges at the end of the beams.
T_{0c} and $T_{0v:}$ Tensile loads previously applied to the cables within columns and beams respectively.
F_{rc} and F_{rv}: Friction forces within hinges at base of columns and ends of beams respectively.
b_c: Width of the square cross section of columns.
h_v: Height of cross section of beams.

The addition of plastic hinges seeks to protect the wooden structural elements, ensuring that they remain undamaged. Due to the above, the plastic bending moment M_{pc} and M_{pb} in column-base and beam-column respectively, must be lower than the bending moment at the limit of elastic behavior of the elements to be joined. Considering the above, the plastic moments of the hinges were defined as the admissible bending moment of the elements to be joined, reduced by a safety factor (SF_c or SF_b), greater than one.

3. Push-over analysis and computational implementation

To assess the seismic risk of structures there are various methodologies that are based on the incremental nonlinear static analysis, commonly known as push-over, and the nonlinear dynamic analysis [7]. The method called "capacity on demand" states that the capacity of the structure obtained from the push-over analysis, is an approximation of the nonlinear dynamic response when it is subjected to seismic loads.

The use of push-over analysis has become important because it has been incorporated into the recommendations of some recognized agencies for seismic assessment of structures [8,9]. The original idea for this evaluation method comes from works of Freeman et al [10].

The theoretical implementation of the push-over analysis for this research is to apply a known pattern of lateral loads to a structure of known geometric and mechanical characteristics. The intensity of this load pattern increases progressively, whereby the behavior of the structure changes from linear to nonlinear (Fig.4 and 5), with progressive degradation of stiffness when the plastic hinges open (Fig.5). The intensity of the load increases achieving the gradual opening of the plastic hinges until all of them are open and, therefore, working at full capacity. After achieving this, the structure has reached its ultimate capacity, allowing only lateral displacement increases without increasing the load magnitude. After reaching the maximum capacity, the lateral displacement limit is defined by the maximum opening capacity of the hinges, which corresponds to a design parameter of them, which in this study was considered virtually unlimited.

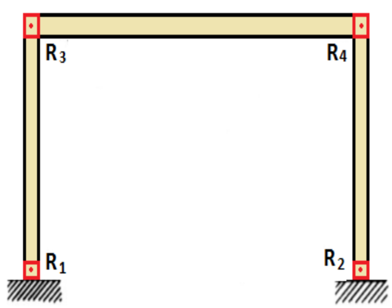

Fig. 4 – Frame working in linear range, all hinges close.

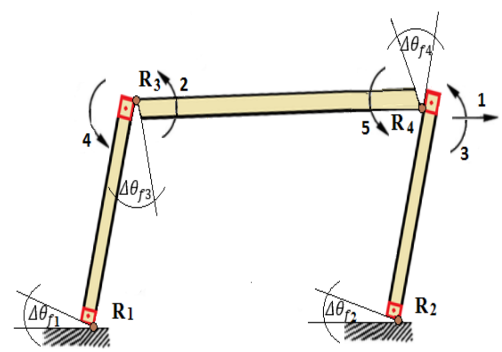

Fig. 5 – Frame with all hinges open.

Once the response of the analyzed system in terms of base shear and roof displacement is obtained, the relationship between these variables is plotted.

The safety factors mentioned, are directly related to the plastic capacity of the plastic hinge. These equations establish a relationship between the pre-stressed force (T_o) of the cable, that provides the linear component of the hinge, and the friction force (F_r), which provides the energy dissipation. The safety factors mentioned above were considered equal for all beam-column and column-base joints. For each frame analyzed, the theoretical reduction factor response, R_t, for different values of safety factors, SF was obtained.

Using the tools provided by the MATLAB software, a programmable algorithm was developed (Fig.6). This algorithm is used to model a framework of one to five floors, incorporating the plastic hinge proposed in this research. By entering input data and mechanical parameters of materials, geometry and pattern loads of push-over already defined, the structural response of a set of frames from one to five floors was obtained, in terms of strength of base shear load and roof lateral displacement.

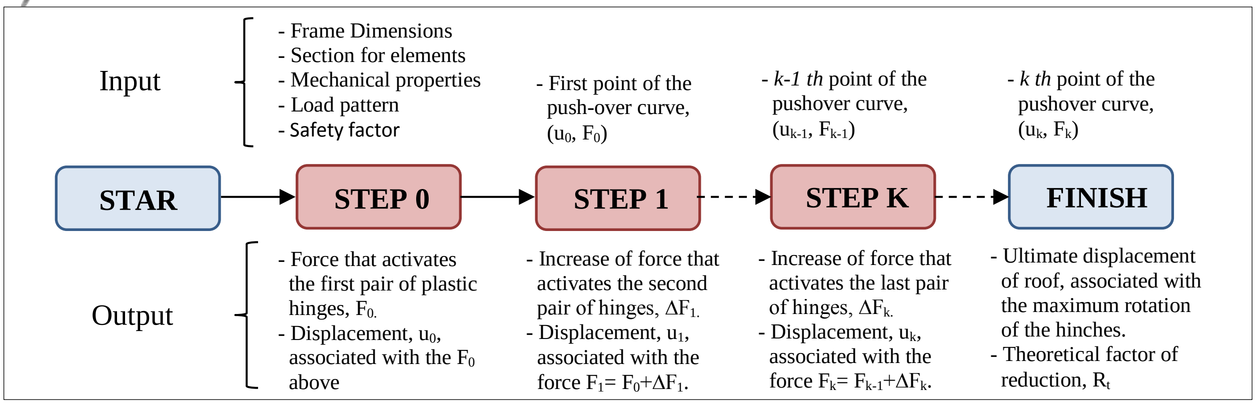

Fig. 6 – Computational implementation of push-over analysis.

4. Results of the Analysis

For each one of the frames studied, a series of parametric push-over analysis were performed. The parameter involved was the structural safety factor regarding flexural failure in glulam, which defines the plastic hinge capacity. In the parametric analysis, values of the safety factor SF from 1.0 to 4.0 were considered. Finally, for each of the analysis carried out, curves of plastic and linear elastic capacity for the frames with and without plastic hinges respectively were obtained. From these curves, the theoretical reduction factor of the response, R_t, was determined.

Didactic examples of how to implement the plastic hinge design, according to the proposed mathematical model are presented. According to the above examples, the section area of the pre-stressed cable, A, and the normal force, N, are determined by knowing the initial deformation of the pre-stressed cable, and the friction coefficient in frictional elements.

Finally, the optimal parameters of the joints determined for a frame of 5 floors with SF=1.5, are used in a time-history analysis. The results of the analysis of the frame with and without plastic hinges are compared, showing the efficiency of plastic hinges on reducing the base shear load and internal bending moment in the glulam elements.

4.1 Push-Over Analysis

To present the results, four plots for each value of the SF considered are included in Fig.7. These graphs show the push-over curve associated to each frame studied, from one to five floors. Jointly, the red line drawn represents the linear behavior of a two- stories frame (reference example).

The theoretical reduction factor R_t was obtained by using different values of the safety factor SF from 1 to 4 in the push-over analysis of frames from 1 to 5 stories high. From these results, it is observed that the values of R_t obtained for frames from 1 to 3 stories high are almost equal. The same occurs for the frames of 4 and 5 stories. Using these results, the curves of Fig. 8 were drawn. From these plots it is evident that the theoretical reduction factor, R_t, is directly proportional to the safety factor SF, referred to beams and columns of the elastic model of the frame.

The theoretical reduction factors of linear response, R_t, seen in Fig. 8, show values that can be even higher than those indicated in the Chilean seismic code, NCh433, for wooden frames (5.5). The above suggests the possibility of designing structures with seismic forces lower than those indicated by the NCh433 code, with security levels and serviceability higher than in current designs and potentially without damage. This would have a positive impact on reducing costs due to the reduction of the cross section of the beams and columns required by analysis, because of a further reduction of the seismic shear load.

In classic designs of wooden frames considering R = 5.5, according to the seismic design code NCh433, the energy dissipation is concentrated in the joints. The junctions currently used for wood structures correspond to metallic elements bolted, glued or mechanically locked. For these joints dissipation of energy is required for developing permanent damage in the materials involved, either by yielding of steel, crushing or cracking of the wood. In designs using the plastic hinge proposed in this research, the non-linear deformations are reversible and damage is nonexistent.

From the calculated results, for security factors lower than 3, values of R_t, that are below that indicated by the NCh433 code are obtained. That does not necessarily mean that in these conditions the design using the proposed hinge is unfavorable, because one of the advantages of the dissipative junction proposed is to be able to return to its original configuration without damage in its own elements when achieves the plastic capacity.

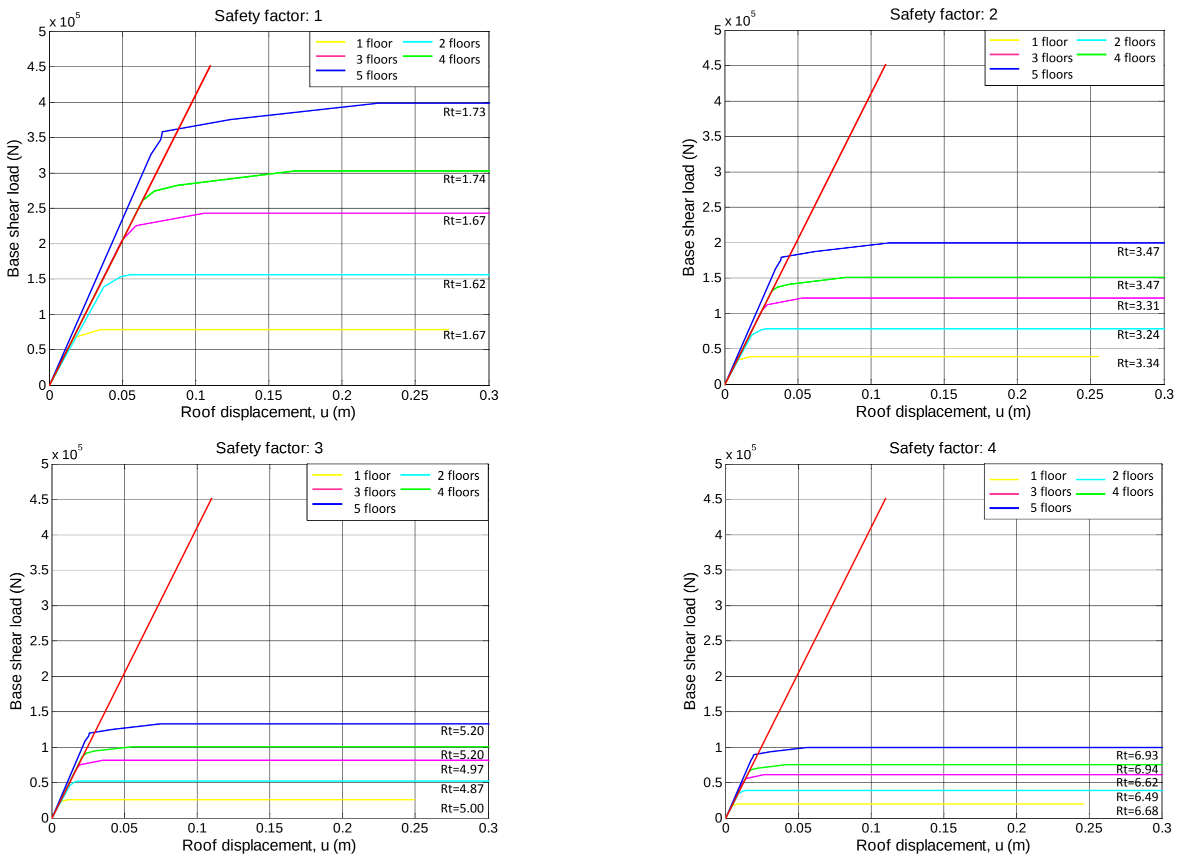

Fig. 7 – Curves of roof displacement u (m) v/s base shear load (N), for different values of SF.

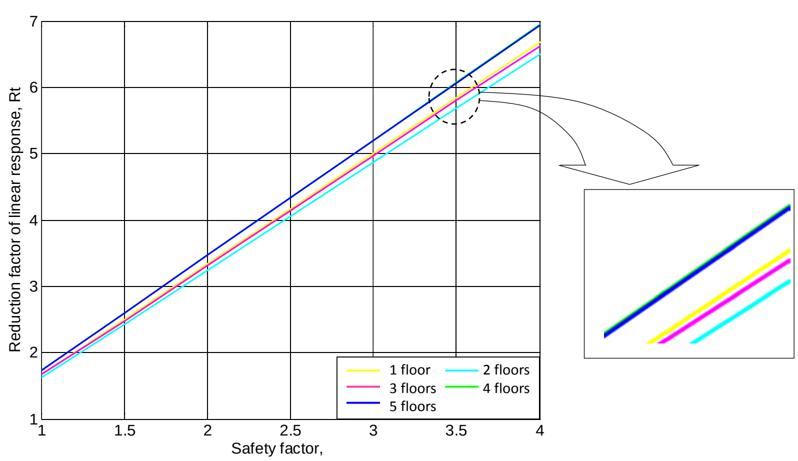

Fig. 8 – Safety factor, SF, versus theoretical reduction factor, R_t, for all 1 to 5 stories frames.

4.2 Example of design of the plastic hinge

According to the mathematical model defined in section 2.2, the design parameters "tension in cable, T_o" and "friction force, F_r" are determined from the plastic moment of the plastic hinge. The latter is related to the bending moment in the elastic limit of the wooden elements to be joined, and with the design safety factor, SF. To establish the design methodology of the hinge, the "upper yielding moment, $M_f^{(S)}$" and the "lower yielding moment, $M_f^{(I)}$" are defined. The first one is the bending moment necessary to achieve the opening of the hinge due to external loads. The second one is the bending moment developed by the hinge when it closes by itself once the external loads are removed.

In Fig. 9a, the plastic hinge with the internal forces of design is shown when the hinge is opening. In this case, both forces, T_o and F_r, act in the same direction. In Fig. 9b, the plastic hinge is shown when it is closing, including the same forces on it. Here, the friction force begins to act contrary to the tension T_o.

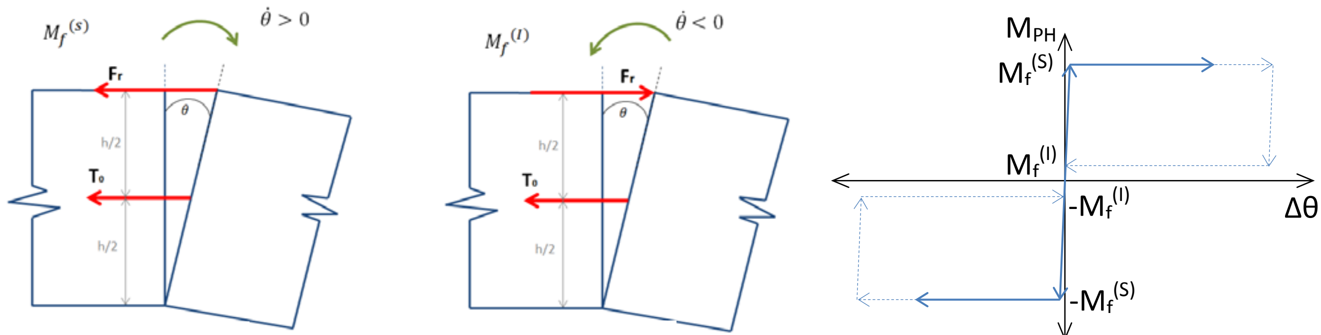

Fig. 9 – Design forces, T_o and F_r, in opening and closing, and Moment-Rotation curve of the hinge.

From Fig.9, and by imposing static equilibrium of bending moment around the center of rotation, it is possible to obtain the equations that define the upper and lower yielding bending moment.

$$M_f^{(S)} = F_r \cdot h + T_0 \cdot \frac{h}{2} = \frac{M_{adm}}{FS} \tag{5}$$

$$M_f^{(I)} = -F_r \cdot h + T_0 \cdot \frac{h}{2} \geq 0 \tag{6}$$

Where the friction force, F_r, is proportional to the normal force applied, and the proportional constant is the kinematic friction coefficient, μ.

$$F_r = \mu \cdot N \tag{7}$$

The coefficient μ depends on the rugosity and materials in contact, and it is an input parameter. For purposes of design of the plastic hinge, it is considered that the materials in contact are both steel, in which case the kinematic friction coefficient is μ=0.57 [11].

The cable is considered made of steel, and its tension T_o is defined by the following equation:

$$T_0 = \Delta l_0 \cdot K \tag{8}$$

Where K is the axial stiffness and Δl_0 corresponds to the elongation of the cable, which is expressed in terms of the strain and the cable length l_0.

$$\Delta l_0 = \varepsilon_0 \cdot l_0 \tag{9}$$

The strain of yielding of the steel was considered as 0.0021, divided by a safety factor, which was considered equal to 1.5. Thus the value of ε_0 used in the design is 0.0014. The length of the cable, l_0, corresponds to the length of the beam plus the width of both columns.

The tension in cable, T_o, is related to the cross section area, A, by equations (8), (9). Thus, the area A, is the direct parameter of design of the hinge, and not the tension in the cable, T_o, which is an indirect parameter.

In order to maximize the energy dissipation, the condition $M_f^{(I)} \ll M_f^{(s)}$, must be imposed. The above condition implies that in a loading and unloading hysteresis loop, it encloses a larger area and, therefore, maximizes the energy dissipated. By imposing the above condition plus Eq. (6), the following is obtained:

$$T_0 \gtrsim 2 \cdot F_r \tag{10}$$

From all the above and by using Eq. (5) and inequation (6), the design parameters N (normal force) and A (area of cross section of the wire), were determined and they are presented in Table 4. In this table, the parameters of plastic hinges used in the push-over analysis that resulted in the factors R_t=5.5 and R_t=6.5 are shown. These values of R correspond to the values indicated by the Chilean standard (NCh433) for wood and steel frames respectively.

Table 4 –Parameters of desing of the plastic hinges used in the push-over analysis, for R_t =5.5 y R_t =6.5.

	R_t = 5.5				**R_t = 6.5**			
	Beam - Column		**Column – Base**		**Beam – Column**		**Column – Base**	
N° of Floors	**A (cm^2)**	**N (kN)**	**A (cm^2)**	**N (kN)**	**A (cm^2)**	**N (kN)**	**A (cm^2)**	**N (kN)**
1	1.24	32.16	2.08	53.77	1.04	26.82	1.76	45.38
2	2.28	58.80	3.72	95.93	1.94	50.03	3.17	81.75
3	3.65	94.13	4.59	118.37	3.08	79.43	3.88	100.06
4	4.41	113.73	5.67	146.22	3.75	96.71	4.82	124.30
5	5.24	135.13	7.13	183.88	4.44	114.50	6.05	156.02

It is necessary to verify that the normal stresses in the wood due to the bending moment transmitted by the plastic hinge, and due to the axial tensile load on the cable, do not exceed the admissible stress of wood. The free-body diagram of a beam in the frame (Fig.10) shows the previous situation.

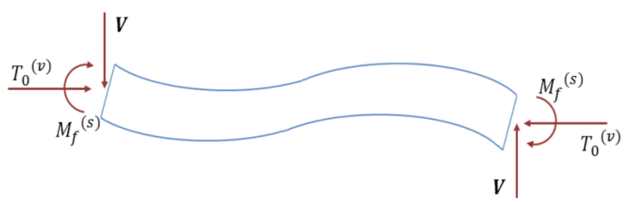

Fig. 10 – Diagram of free body of a beam in one of the wood frame analyzed.

Using Eq. (11), tensions verification is performed. In this case, as an example of verification, the maximum stress of the glulam for the beams of each wood frame analyzed, are presented in table 5, which must be lower than the admissible stresses shown in table 1. The results of stresses verification are presented in table 5.

$$\sigma_{máx} = \frac{M_f^{(S)} \cdot (h/2)}{I} + \frac{T_0^{(v)}}{b \cdot h} < \sigma_{adm} \tag{11}$$

The analyses show that it is possible to implement the plastic hinge using the conceptual model of this research, obtaining designs that are feasible to be implemented.

Table 5 –Maximum tensions at the end of the beam.

	Junction Beam-Column	
N° of floors	**σ_{max} (N/m^2)· 10^6 (R=5.5)**	**σ_{max} (N/m^2)· 10^6 (R=6.5)**
1	6.70	5.81
2	6.53	5.66
3	6.66	5.76
4	6.97	6.04
5	6.72	5.84

4.3 Time-History Analysis

In order to illustrate in a more reliable manner the benefit of implementing the proposed plastic hinge, an example of a time history analysis is shown below. The seismic input considered corresponds to the component N0ºE of the Sylmar, 1971 seismic register, at the coast of California, USA. The last register is an impulsive one, with a short duration of the intense phase, and a high peak ground acceleration, PGA=0.84g. The analysis was performed to a frame of 5 stories, with cross sections and mechanical properties of the elements indicated in Tables 1 and 3. Two similar frames, one with rigid joints and another with plastic hinges were analyzed to demonstrate the benefits obtained by incorporating the proposed junction. The mechanical properties of the plastic hinges were calculated according Eqs. 5 to 10, considering a safety factor SF=1.5.

The equation that rules the behavior of a plastic hinge in a time history analysis arises from Eqs. 5 and 6:

$$M_{P.H.} = \left(\frac{1}{2}\cdot\left(T_0 + \frac{1}{2}\cdot k_c\cdot|\Delta\theta|\cdot h\right) + F_r\cdot sign\left(\Delta\theta\cdot\dot{\Delta\theta}\right)\right)\cdot h\cdot sign(\Delta\theta) \qquad (12)$$

Where:

T_0 : Pre-stress tension in the cable that provides the auto-centering capacity of the plastic hinge.
k_c : Axial stiffness of the cable, which can include a flexible spring in series to reduce their stiffness.
$\Delta\theta$: Relative rotation of the opposite faces of the plastic hinge.
F_r : Frictional force (F_r=μN) at the edges far of the neutral axis of the plastic hinge
h : Height of the element joined or distance between the two hinges in the plastic hinge

In the push-over analysis, the stiffness kc is neglected, then, $T_0 + \frac{1}{2}\cdot k_c\cdot|\Delta\theta|\cdot h \approx T_0$. However, in the time history analysis, this stiffness is considered. Additionally, in order to consider the energy dissipation in the frames needed to tend vibrations to zero before the earthquake has finished, a viscous damping matrix was defined for each one of the two frames analyzed. For the frame with rigid connections, a constant 5% of the critical damping is used for all modes, and for the frame with plastic hinges, only a 2% for all modes was considered. The above is because in the frame with plastic hinges, the wood elements remained elastic and therefore the damping developed in the glulam should be lower than that in the frame with rigid connections.

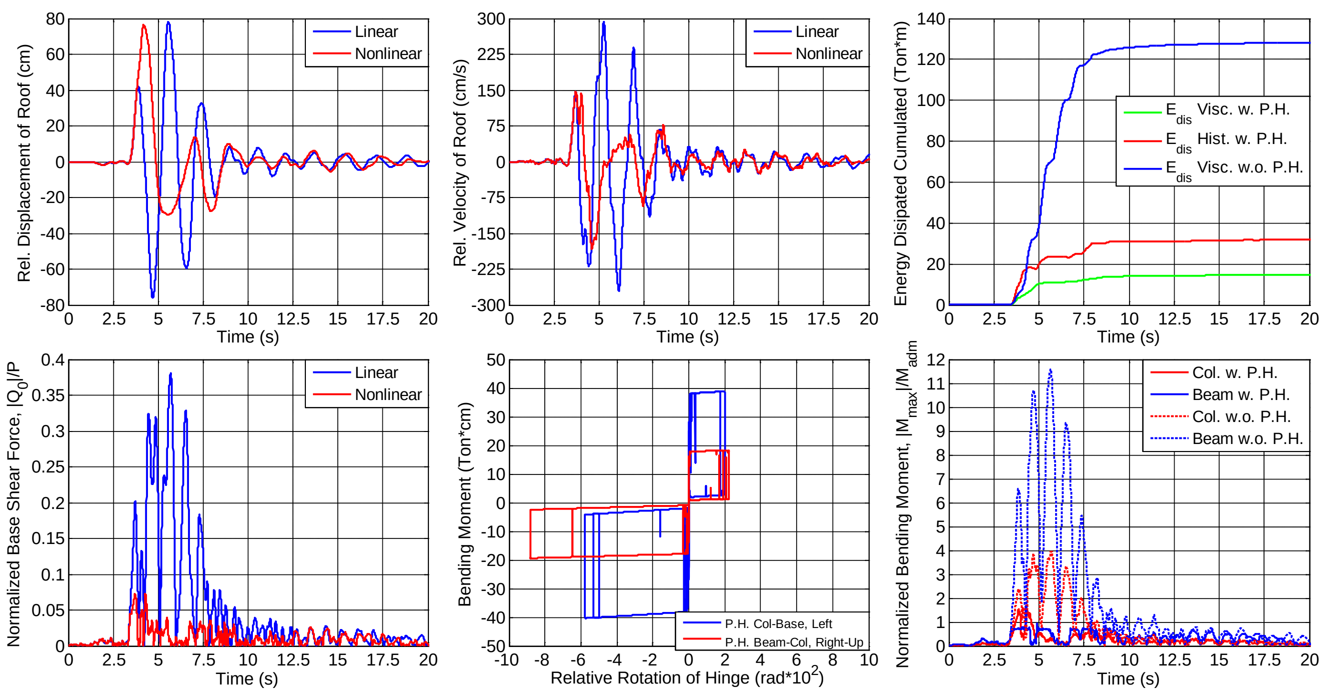

Fig. 11 – Time history analysis of a 5 stories frame, with (w.) and without (w.o.) plastic hinges

At the beginning of the analysis, the frame with plastic hinges shows no differences in its behavior compared to the frame with rigid connections, as long as the plastic moment is not reached. The last occurs in the time history analysis at t≈3.8s. After this time, the frame with plastic hinges starts to dissipate energy by hysteretic behavior and the stiffness of the frame begins to reduce. Because of this, the earthquake has a lessened effect in the frame with plastic hinges compared to the frame with rigid joints.

By observing the above results, the benefit of using the plastic hinges in the reduction of the base shear load elements (Fig. 11, first column, second row of plots) and maximum internal bending moment in glulam elements (Fig. 11, third column, second row of plots), is evident. In the first case, the shear load reached a 37% of the weight of the structure in the elastic frame and only a 7.5% in the frame with plastic hinges. In the case of the internal bending moment, the linear frame exceeded 11.5 times the admissible bending moment in beams (1050% more than the admissible), and 4 times in columns (300% more than the admissible). However, in the frame with plastic hinges, beams do not exceeded the admissible bending moment and columns exceeded it in 50%, but only for less than 2 seconds during the earthquake.

Despite the above, the maximum lateral displacement of the roof (Fig. 11, first column, first row of plots) was not so different between the two frames analyzed. However, the frame with plastic hinges has the capacity of reducing the lateral displacement more quickly than the frame with rigid connections, making these tend to zero, returning the structure to its undeformed configuration.

The maximum velocity of displacement of the roof (Fig. 11, second column, first row of plots) in the frame with plastic hinge reaches approximately 50% of that reached in the frame with rigid connections. This is explained because of the efficiency of the energy dissipation in the plastic hinges in comparison with the linear system. In fact, by observing Fig. 11, third column, first row of plots, the linear elastic structure dissipated more energy in total than the structure with plastic hinges. But in the second case, the dissipation achieved a faster reduction of the response.

The hysteretic curves of one of the column-base and beam-column plastic hinges (Fig. 11, second column, second row of plots) shows evidence of the auto-centering behavior of the proposed junction. In this figure, if the bending moment is cero, the only coordinate of the relative rotation between opposite faces of the plastic hinge is cero. The last indicated that after the earthquake is over, and the loads on the structures are retired, the plastic hinge has the capacity to close itself and return the structure to its undeformed configuration, without damage in the glulam element and a low damage in the hinges, due to chafing in the fictional elements. The above is not a problem, because the frictional elements can be designed with a relatively soft element between them, which can be replaced when they are worn out.

5. Conclusions

Through a push-over analysis, a methodology for calculating the theoretical reduction factor of the linear response, R_t, was defined. In some of the cases analyzed, values of R_t higher than those specified in NCh433code were obtained. This represents a benefit from the economic point of view, by allowing smaller cross sections of structural elements. Additionally, the designs including this plastic hinge focus the dissipated energy in the hinge, keeping the rest of the structure in linear range without damage. The above suggests the possibility of designs that are safer than the current designs by using the proposed plastic hinge.

The design features give the proposed hinge an auto-centering capacity, allowing the structure to recover its original configuration after an earthquake, without permanent deformation. This implies higher standards of serviceability as well as lower or no costs associated with repairs after a severe earthquake has occurred.

It was shown by calculations that the plastic hinge presented herein is possible to be implemented in a real design. This can be done for the example presented above, by using a pre-stressed cable with a diameter in the range of 12-25mm, and normal forces between the frictional elements in contact in the range of 32-135 KN. These are reasonable design parameters that are possible to be utilized.

It was demonstrated that the internal normal stresses induced by the pre-tensioned cable of the plastic hinge, plus the stresses induced in glulam elements due to the plastic bending moment, do not exceed the admissible stress

of the material. The above confirms the fact that the proposed plastic hinge can be used in a seismic design of a glulam frame, generating a safe design by keeping wood elements in a linear elastic range without any damage.
From the time-history analysis performed, it is possible to observe that the real reduction factor of the linear response, was greater than the value predicted by the push-over analysis. In fact, this reduction factor is given by the rate between maximum base shear load in the linear and plastic frame. In the case of the push-over analysis of the 5 story high frame with SF=1.5, the analysis gives $R_t \approx 2.5$. However, from the time history analysis, this factor was found to be $R_t \approx 37/7.5 \approx 4.9$, much higher than the one obtained from the push-over analysis. The above is explained because the push-over analysis does not reproduce the real dynamic behavior of the system. Therefore, considering what was observed above, it is possible to suggest that structural designs that incorporate the proposed plastic hinge, may be safer than predicted by the push-over analysis of.

6. References

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