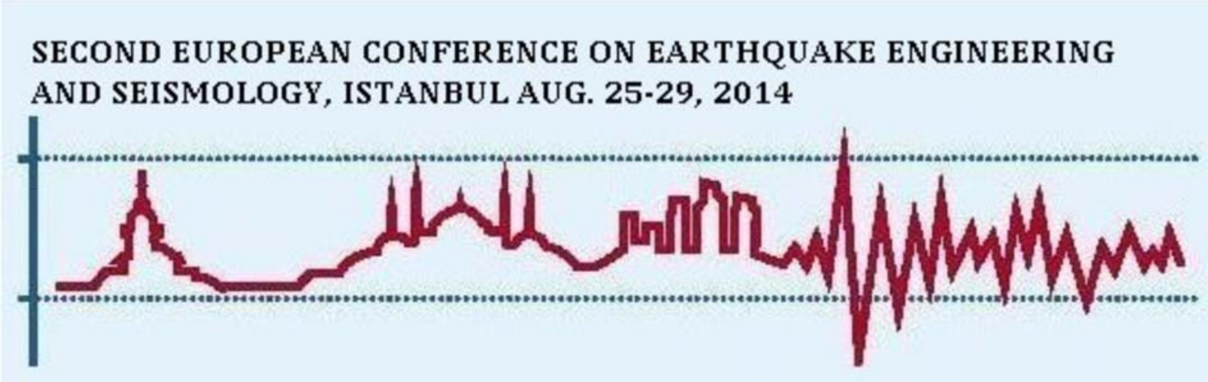

ENERGY-BASED DESIGN OF A SEISMIC PROTECTION SYSTEM OF TIMBER PLATFORM FRAME BUILDINGS USING ENERGY DISSIPATORS

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ABSTRACT

This paper deals with a new seismic protection system for timber platform frame buildings, either for retrofit or for new construction. The system consists in connecting the timber frame to a steel framed structure that includes hysteretic energy dissipators designed to absorb most of the seismic input energy thus protecting the timber frame and the other steel members; alternatively, the system might contain other dissipative devices. The steel structure comprises horizontal beam-like elements, vertical column-like elements and chevron-like bracing members; the beam-like elements are steel collectors (belts) embracing each slab of the building and the bracing members hold the energy dissipators. The steel structure is self-supporting, i.e. the timber frame is not affected by horizontal actions and can be designed without accounting for any seismic provision; in turn, the steel members do not participate in the main load-carrying system. The timber-steel interface has been designed to avoid any stress concentration in the transfer of horizontal forces and to guarantee that the yielding of the dissipators is prior to any timber failure. This research belongs to a wider project aiming to promote the structural and constructional use of timber by improving the seismic capacity of wooden buildings; this research includes experiments and advanced numerical simulation aiming to derive accurate design criteria.

INTRODUCTION

Timber construction offers relevant environmental benefits, if wood is collected from local and sustainably exploited forests, since this promotes the planting of trees and itself stores carbon during its lifetime, thus reducing the greenhouse effect. Moreover, wooden construction presents other relevant advantages: high reusability and recyclability, moderate cost, high resistance / weight ratio, simpler foundations because of the timber lightweight, construction rapidity, insulating qualities, and pleasing appearance. Conversely, timber has some major drawbacks for structural and constructional use: limited strength, heterogeneity and anisotropy, hygroscopicity, shrinkage, swelling, controversial fire resistance, degradability, maintenance requirements, difficulty of connections and contentious seismic resistance. This research aims to contribute to overcome the last two limitations by proposing additional steel elements including dissipative connections to protect the wooden members from damage generated by seismic demands. The proposed solutions are oriented to the most vulnerable types of timber buildings.

Recently, timber construction is gaining attention worldwide due to the increasing concern for the aforementioned environmental issues and to the most recent relevant advances in timber engineering, particularly in the development of new industrialized structural elements with minimized timber imperfections: CLT "Cross-laminated timber", LVL "Laminated-veneer lumber", glued-laminated

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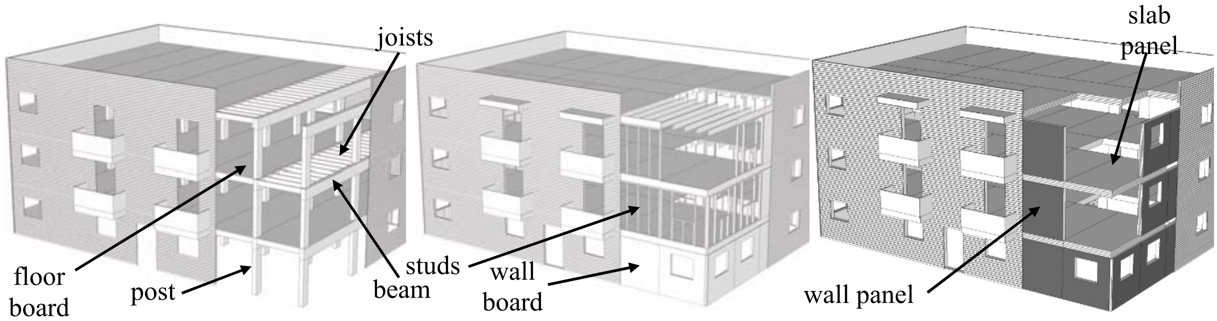
sections “glulam”, “plywood”, among others. As well, there have been significant developments in innovative connections, fire protection and durability. Timber building construction can be classified in heavy timber framing, timber platform frames, and cross laminated timber systems. Figure 1 shows sketches of these three structural types. Heavy timber framing (Figure 1.a) is composed of timber columns and of timber slabs made up of beams, joists and a top sheathing board. Heavy timber framing members are mainly made of sawn (solid) timber or of glulam. Timber platform frame construction (Figure 1.b) differ mainly from the heavy timber frames in the use of thin-panelled walls as the only carrying-load system. In timber platform frames, sheathing boards framed with timber studs form the vertical panels. Cross-laminated timber systems (Figure 1.c) are assemblies of thick panels; such panels are used both as vertical supporting members and as slabs. Heavy timber framing and timber platform frame are appropriate for low-height buildings while cross-laminated timber systems are convenient for mid-height and tall buildings (Green, Karsh 2012). Timber platform frame buildings consume little timber, are much spread and, as discussed later, are highly vulnerable to earthquakes; for these reasons, this work focuses on them.

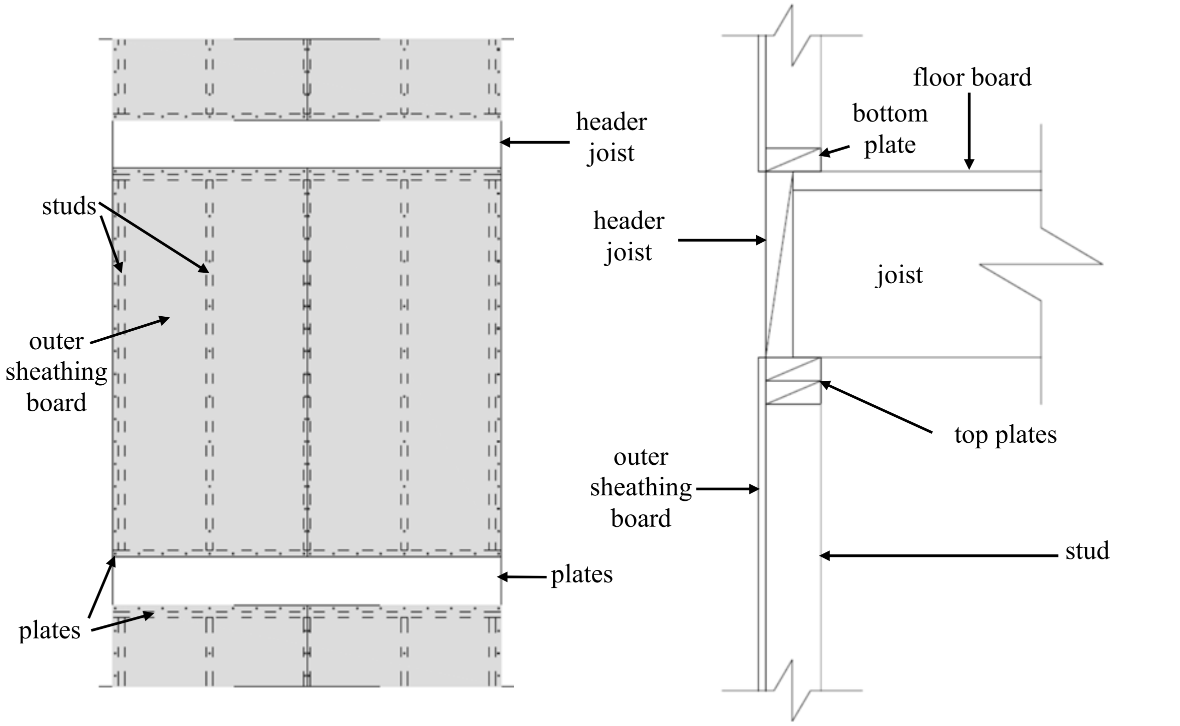


(a) Heavy timber (b) Platform frame (c) Cross laminated timber

Figure 1**.** Major types of timber building construction

As shown by Figure 1.b, the timber platform frame construction with thin-paneled walls are essentially an assembly of vertical and horizontal wooden framed panels (Handbook 1 2008), see Figure 2. The horizontal floor and roof panels are constituted by a top wood-based sheathing board (plywood, particleboard, oriented strand board, etc.) supported by side and inner timber joists. The vertical panels consist of wood-based sheathing boards (plywood, particleboard, oriented strand board, hardboard, etc.) framed with timber studs; the top and bottom sides of each panel are reinforced with plates (also known as rails or binders). The walls have load-bearing capacity and can be either external (cladding) or internal (partitioning). Belonging this construction technology to the platform timber frame family, the vertical panels are one-story high; can be either whole (unpierced) or with door or window openings. The sheathing boards are nailed to the framing elements, i.e. joists in the horizontal panels and studs and top and bottom plates in the vertical panels. The connection between the floor and wall panels is commonly established by nailing the top and bottom plates to the joists. Given that the vertical and horizontal strength of the wall members is limited, this construction type is ordinarily used only for short-to-mid-height buildings, usually not exceeding six floors.

Wooden construction is, potentially, highly resistant to earthquakes; it is due to their lightweight, to their high damping, to the increased resistance of timber to rapidly varying forces, and to the high structural redundancy (mostly in thin and thick paneled buildings, Figure 1.b and Figure 1.c, respectively). Nevertheless, these qualities do not suffice because, under extremely severe seismic excitations, the constructions need to be ductile; i.e. capable to absorb the input energy by means of inelastic deformations but without collapse. Contrary to a certain common belief, timber is a rather brittle material and cannot provide enough ductility to the construction; this lack impairs the seismic qualities of timber construction, together with the important unpredictability of the damping characteristics of timber. The traditional solution to this problem consists in using connections with enough ductility. In timber structures, three major types of connections exist: traditional (craftsman or carpenter), chemical (glued) and mechanical (using metal elements); only the last type is ductile. Mechanical connections are ductile but, if they are damaged after severe ground motions, such damage is mainly focused in timber, thus preventing any possibility of repair. Current research has focused in designing and testing connections with higher energy dissipation capacity (Parisia & Piazza 2002, Leijten et al. 2006, Awaludin et al. 2007, Andreolli et al. 2011, Piazza et al. 2011, Popovski & Karacabeyli 2012), but in such connections the damage initiates in timber instead of concentrating only in the metal elements. In other words, mechanical connections are capable of absorbing part of the energy introduced by the seismic input but such energy is absorbed through irreparable damage in the timber members; moreover, in most of the cases, the energy absorption capacity is only limited. The use of energy dissipators looks particularly convenient.



(a) Wall panel. Front view (b) Wall and slab panels assembly. Side view

Figure 2. Timber platform frame structures

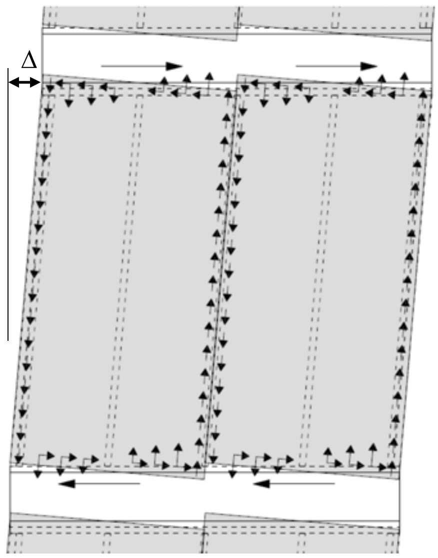


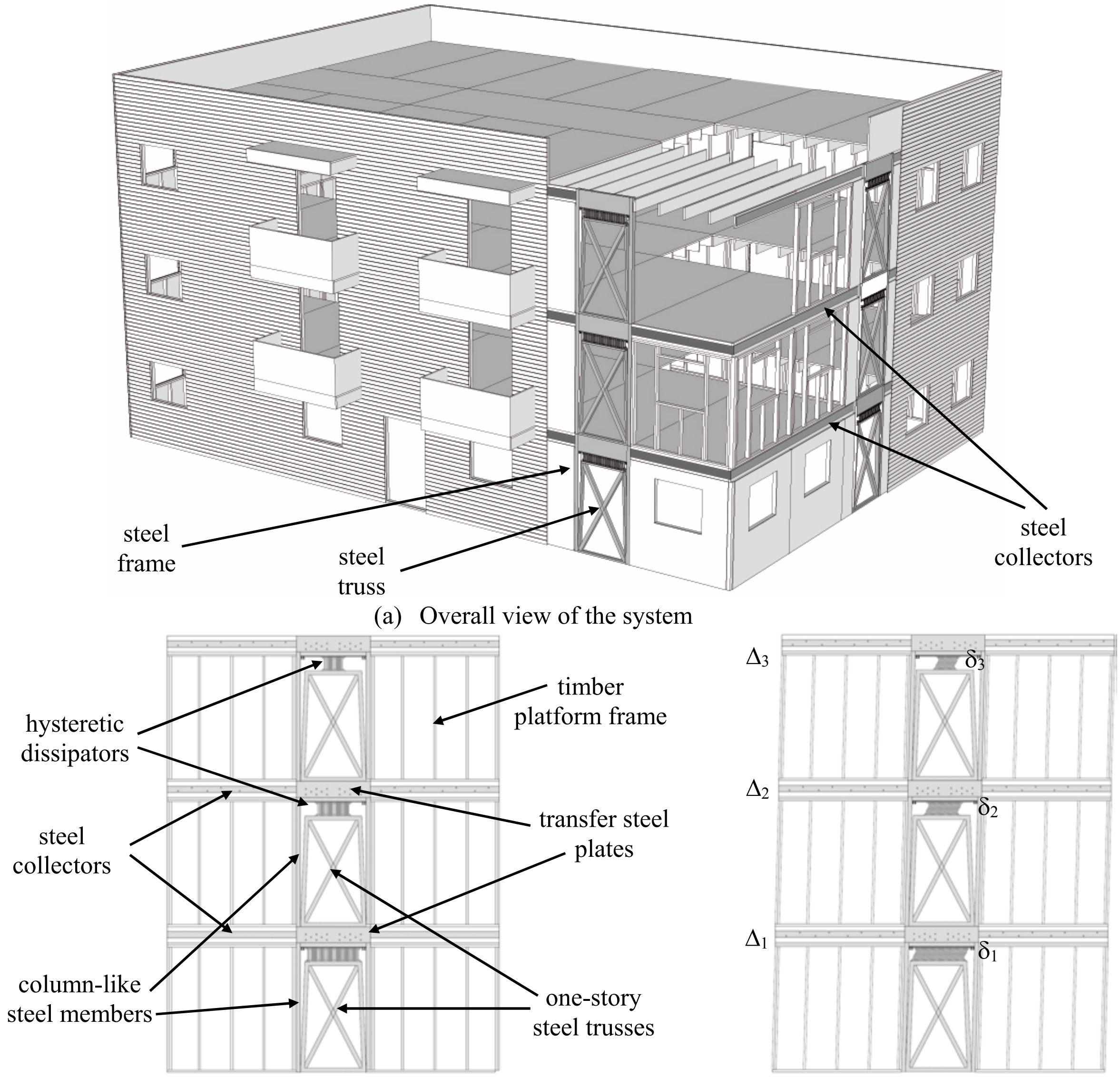
Figure 3. Lateral deformation of timber platform frame buildings during seismic excitation

The seismic performance of timber platform frame buildings (Figure 1.b and Figure 2) can be analyzed in the context of the seismic functioning of timber construction, which has been discussed previously. Specific considerations for this structural type are: high structural redundancy, extreme lightweight construction, and limited horizontal resistance. About this last issue, Figure 3 displays the deformations involved in the interstory drift racking motion of timber platform frame buildings; Δ is the interstory drift displacement. Figure 3 shows that the lateral strength is mainly provided by the shear capacity of the vertical sheathing boards and by the nailed connections between the studs and the boards (Hoekstra 2012). Moreover, in buildings with high plan aspect ratio, the limited strength and stiffness of the floor diaphragms does not guarantee an even transmission of horizontal forces to the lateral loads resisting elements. Given that the lateral strength is low, it can be globally said that timber platform frame buildings are vulnerable to severe earthquakes.

This work describes a new seismic protection system for timber platform frame buildings, either for retrofit or for new construction. The system consists of connecting the timber frame to a steel framed structure that includes hysteretic energy dissipators. The timber frame is not affected by horizontal actions and the steel members do not participate in the main load-carrying system. The timber-steel contact is even, smoothed and spread; then, the yielding of the dissipators is prior to any timber failure. This research belongs to a project including experiments and advanced numerical simulation.

PROPOSED SEISMIC PROTECTION SYSTEM

The proposed solution is primarily meant for timber platform frame buildings with symmetric rectangular plan configuration, i.e. without relevant re-entrant corners or edge recesses, and with uniformity along the height, i.e. without significant setbacks. The steel protective system is formed of four vertical truss-like planar structures connected to each of the building façades by steel collectors, as shown by Figure 4.a. This layout is intended to provide plan symmetry and torsion strength; special attention has been paid to this last issue, giving the torsion effects detected in the NEESWood research project (Pei et al. 2010). Figure 4.b displays a 3-story timber building incorporating a steel protective structure including energy dissipators at each story; such devices are flexible steel plates which are intended to yield (and thus, to dissipate energy) under interstory drift motions. The truss-like steel structure comprises: (i) column-like vertical members, which are continuous down to foundation, (ii) horizontal plate members embracing the whole building at each story level thus constituting a kind of confining steel collector and (iii) single-story trusses. The connections among these members will be rigid. The column-like vertical members have two purposes: to transmit to the foundation the vertical forces generated by the horizontal shear forces in the flexible steel plates, and to provide residual lateral stiffness to the building after the yielding of the flexible plates. The plated steel collector is continuously joined to the adjacent timber header joist by nailing or other equivalent mechanical connectors. The segments of the collector that are right above the dissipative devices are locally reinforced to reach higher bending stiffness to guarantee the transmission of forces between the steel members of adjoining stories; such stiffened segments are termed as "transfer steel plates", see Figure 4.b. In each floor, the dissipators are rigidly connected by their lower end to the aforementioned single-story rigid truss and by their upper end to the top transfer steel plate. Under seismic excitation, the massive floor slabs draw the steel collectors; the subsequent interstory drift motion generates strains in the devices, as shown by Figure 4.c. In Figure 4.c, Δ_i represents the i-th interstory drift, i.e. the relative displacement between stories i and $i - 1$, and δ_i is the transverse displacement of the flexible steel plates; due to the lateral flexibility of the truss, δ_i is slightly smaller than Δ_i. The steel collectors will be prestressed to avoid separation from the timber beams due to wood shrinkage or local wood compression; in this way, when the timber mass is pushing, the steel-timber stresses transfer will be generated mainly by compression of the front timber beams and by friction along the side timber beams. Furthermore, since the steel collectors produce a certain confinement of the slabs, an additional benefit of this solution is an improved diaphragm behaviour. The dissipators must constitute the "weakest link" in the lateral resisting system, namely, the yielding of the dissipative devices should be prior to any other failure. Since steel is a lot more resistant than timber, a highly distributed, even and smoothed contact between both materials is pursued. As discussed in the previous paragraph, this objective is accomplished mainly through the steel collectors.



(a) Overall view of the system

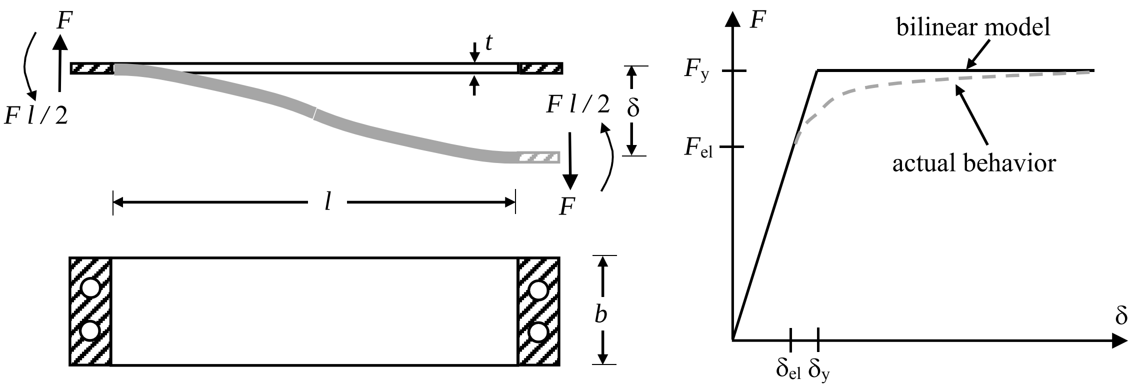
(b) Front view of a façade

(c) Behavior under interstory drift deformation

Figure 4. Proposed steel seismic protection system

DISSIPATIVE BEHAVIOR OF THE PROPOSED DEVICES

This section describes the dissipative behavior of the flexible steel plates; see Figure 5. Figure 5.a represents the deformation of a given flexible steel plate and Figure 5.b depicts the ensuing force-displacement law; the plots in Figure 5.b have been derived assuming an elastic-perfectly-plastic behavior for the steel. In Figure 5.a, l is the clear length and t and b are the thickness and the width of the plate, respectively; F and $F\, l\, /\, 2$ are the reaction forces and δ is the transverse displacement of the plate. Figure 5.b shows that the force-displacement law has an initial linear elastic branch followed by a curved plastic branch. F_{el} and δ_{el} are the force and the displacement that correspond to the onset of yielding in the extreme sections; after that, yielding is progressing and the curve is accordingly becoming less steep. Once the plastification of the extreme sections is completed, plastic hinges are formed and the plate loses utterly its stiffness; therefore, the plastic branch approaches asymptotically to the yielding force F_{y}. For modelling purposes, a bilinear diagram whose initial branch has the same slope than the elastic one and the second branch is horizontal represents the actual law. Figure 5.b shows that the corner displacement δ_{y} can be read as the yielding displacement of the bilinear model.



(a) Deformation mode (b) Force-displacement relationship

Figure 5. Dissipative behavior of a flexible steel plate

By performing simple linear elastic analyses neglecting the contribution of the shear force to the deflection, the following closed-form relations among the involved elastic quantities are obtained:

$$\delta_{el} = \frac{f_y\, l^2}{3\, E\, t} \qquad F_{el} = \frac{f_y\, b\, t^2}{3\, l} \qquad k = E\, b \left(\frac{t}{l}\right)^3 \tag{1}$$

In equation (1), E and f_y are the steel modulus of elasticity and yield stress, respectively, and k is the stiffness of the force-displacement elastic branch, given by $F = k\,\delta$. Given that the plastic moment of a rectangular section is equal to 1.5 times the elastic one, it follows immediately that

$$\delta_y = 1.5\, \delta_{el} = \frac{f_y\, l^2}{2\, E\, t} \qquad F_y = 1.5\, F_{el} = \frac{f_y\, b\, t^2}{2\, l} \tag{2}$$

In the derivation of equations (1) and (2), the plastic interaction between the shear force and the bending moment has been neglected. This assumption holds as long as l is significantly bigger than t.

Under reverse cycling motion, the bilinear behavior depicted in Figure 5.b turns into the hysteresis loop displayed in Figure 6; δ_{max} accounts for the maximum transverse displacement. The smoothed branches of the unloading branches are typical of hysteretic devices (Palazzo et al. 2009).

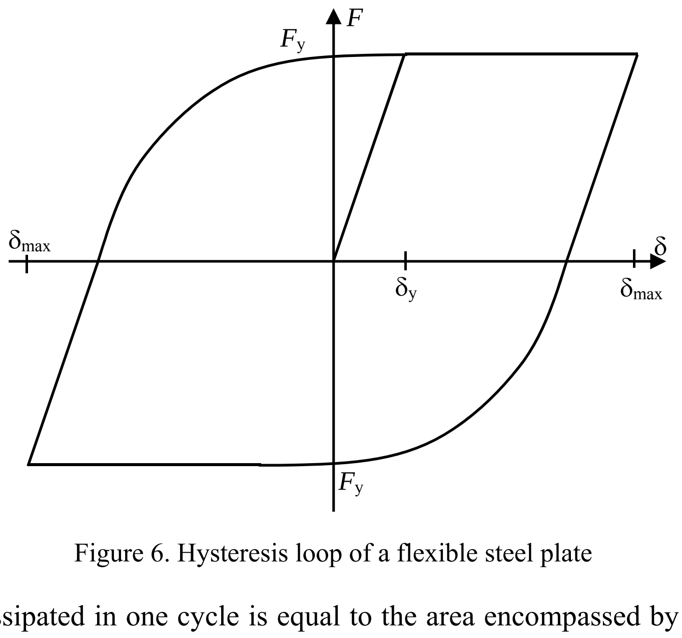


Figure 6. Hysteresis loop of a flexible steel plate

The energy dissipated in one cycle is equal to the area encompassed by the loop displayed in Figure 6; for a given seismic input the absorbed energy E_d depends on many parameters, such as yielding force, maximum displacement, duration, impulsivity, and frequency content, among others. However, in normal conditions it might be grossly accepted that such energy is only related to the yielding force and the maximum displacement; such conclusion relies on the assumption that the displacement ductility ($\mu = \delta_{max} / \delta_y$) and the cumulative ductility (in a single device, $\eta = E_d / F_y \delta_y$) are related by the approximate relation $\eta = 4 (\mu - 1)$ (Akiyama 1985):

$$E_d = \eta F_y \delta_y = 4 (\mu - 1) F_y \delta_y = 4 \left(\frac{\delta_{max}}{\delta_y} - 1 \right) F_y \delta_y = 4 F_y \left(\delta_{max} - \delta_y \right) \approx 4 F_y \delta_{max} \qquad (3)$$

Equation (3) holds if μ is sufficiently high, so that $\mu - 1$ can be approached by μ.

SEISMIC DESIGN BASED ON INPUT ENERGY SPECTRA

In conventional earthquake-resistant design of buildings and other constructions, the dynamic effect of the input ground motion is represented by static equivalent forces, which are obtained from acceleration response spectra defined as the ratio between the peak ground acceleration and the maximum absolute acceleration in the top of the construction. This approach entails several drawbacks: (i) these equivalent forces are strongly coupled to the elastic and hysteretic characteristics of the structure, thus making seismic design cumbersome, (ii) after the onset of yielding, the correlation between the design forces and the structural damage is poor, and (iii) the damage caused by the cumulative inelastic excursions (Fajfar, Vidic, 1994) is not accounted for. More recently, displacement-based design procedures have been proposed (Priestley, Calvi, Kowalsky, 2007); in these strategies, the dynamic effect of the input is represented by imposed displacements, in turn obtained from displacement response spectra relating the design ground acceleration to the maximum relative displacement in the top of the building. This formulation partially uncouples the input effect –in terms of displacement– from the characteristics of the structure and allows for a satisfactory correlation between the imposed displacement and the component of the structural damage that is related to the maximum displacement. Conversely, in this formulation, the component of damage that is related to the cumulative plastic strain energy cannot be appropriately considered. A more rational seismic design approach, which also overcomes this difficulty, consists in expressing the dynamic input effect through energy response spectra via the Housner-Akiyama energy formulation (Housner 1956, Akiyama 1985). Interpreting the effect of earthquakes in terms of energy is gaining extensive attention (Kuwamura et al., 1994; Bertero et al., 1996; Chou, Uang, 2003; Jiao et al., 2011). This approach features three major advantages: (i) the input effect in terms of energy and the structural resistance in terms of energy dissipation capacity are basically uncoupled, (ii) except in the short period range, the input energy, E_I, introduced by a given ground motion in a structure is a stable quantity, governed primarily by the natural period T and the mass m, and scarcely affected by other structural properties such as resistance, damping and hysteretic behavior, and (iii) the consideration of the cumulative damage can be directly addressed. In the energy-based methods, the design criterion resides in the comparison between the seismic resistance of the structure in terms of energy absorption capacity and the effect of the ground motion in terms of input energy. It is then necessary to establish the E_I input energy spectrum corresponding to the expected earthquake, i.e. design input energy spectrum. The structure is able to absorb, through damping, a part E_ζ of the input energy and the remaining part is dissipated by additional structural damage; this damaging part of the input energy is commonly termed as hysteretic energy E_H (Manfredi 2001). Once the kinetic and elastic energies have vanished, the energy balance equation can be written as

$$E_\zeta + E_H = E_I \qquad (4)$$

If the dynamic behavior of the building is described by lumped masses models, then

$$E_\zeta = \int \dot{\mathbf{x}}^T \mathbf{C} \dot{\mathbf{x}} \, dt \qquad E_I = -\int \dot{\mathbf{x}}^T \mathbf{M r} \ddot{x}_g \, dt \qquad (5)$$

C is the viscous damping matrix, **M** is the mass matrix, $\dot{\mathbf{x}}$ is the relative velocity vector, **r** is the influence vector and $\ddot{x}_g$ is the input ground acceleration; for 2D models of plan symmetry buildings $\mathbf{r} = (1,\ldots,1)^T$. The energy-based design is particularly well suited for constructions incorporating energy dissipators: they are designed to absorb the hysteretic energy. In other words, the energy dissipated jointly by all the devices E_D should be bigger or equal than E_H:

$$E_D \geq E_H \tag{6}$$

Commonly, E_I and E_H are normalized with respect to the mass m of the building and expressed in terms of equivalent velocities V_E and V_D:

$$V_E = \sqrt{2E_I / m} \qquad\qquad V_D = \sqrt{2E_H / m} \tag{7}$$

For practical energy-based earthquake-resistant design, V_E is obtained from available design energy spectra and V_D is estimated from V_E through empirical expressions of the ratio V_D / V_E: $V_D = V_E$ (V_D / V_E). Among other researchers, (Benavent-Climent et al. 2002) proposed design energy input spectra for moderate seismicity regions and (Benavent-Climent et al. 2010) and (Yazgan 2012, López-Almansa et al. 2013) proposed design energy input spectra for moderate-to-high seismicity regions based on Colombian and on Turkish records, respectively. These V_E input energy spectra depend on the soil characteristics (stiff / soft), the seismic design acceleration, the magnitude of the expected earthquakes ($M_s \leq 5.5$ and $M_s > 5.5$) and the type of seismic input (impulsive / vibratory records); conversely, they do not depend neither on the mass nor on the damping parameters. Moreover, except in the short period range, the V_E spectra are also independent of the hysteretic behavior of the structure. A number of researchers (Akiyama, 1985; Kuwamura et al., 1994; Fajfar, Vidic, 1994; Decanini, Mollaioli, 2001; Benavent-Climent et al. 2002, 2010; Yazgan 2012; López-Almansa et al. 2013) have derived empirical expressions of the ratio V_D / V_E; such expressions depend on the soil type, the structural damping ζ, the fundamental period of the structure T_F, and the displacement ductility μ.

DESIGN OF THE PROPOSED SYSTEM

This section presents simplified design criteria of the proposed protective system; despite many assumptions are made, the considered design approach is able to provide reliable and reasonably accurate results. In the framework of the Performance-Based Design (PBD), the Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) performance levels are accounted for (FEMA 356 2000). For buildings of normal importance, such levels correspond to 72, 475 and 970 years return period, respectively (SEAOC 1995). The design consists in selecting the geometrical and mechanical parameters of the flexible steel plates (width b, length l, thickness t and steel yielding point f_y) and the number of plates per level and per direction (n_{pi}) trying to fulfill the requirements for the 3 considered performance levels. Next three paragraphs describe the overall design strategies for IO, LS and CP.

Immediate Occupancy. FEMA 356 defines IO as the post-earthquake damage state in which only very limited structural damage has occurred, the construction remains safe to occupy, the structure essentially retains the pre-earthquake design strength and stiffness, the risk of life-threatening injury is very low, and there is no permanent drift (FEMA 356 2000). This last condition requires that the flexible steel plates do not yield; since, as discussed previously, their yielding should be prior to any other failure, all the other steel members must also remain elastic. In that case, the seismic design strategy based on input energy spectra (i.e., inequality (3)) is not useful since the timber members absorbed all the input energy. The design might be based on design acceleration spectra; since no relevant structural damage is accepted, the response reduction factor should be equal to one. As in the other performance levels, the seismic action should be withstood by only the additional steel structure. In the NEESWood research project it is estimated that the IO damage state corresponds to a peak interstory drift in the range 0.1–1% (Pei et al. 2010, Christovasillis et al. 2007). For this range of drift displacement, the wood framing and OSB/Plywood Sheathing would experience minor splitting and cracking of sill plates and slight sheathing nail withdraw.

Life Safety. FEMA 356 defines LS as the post-earthquake damage state that includes damage to structural components but retains a margin against partial or total collapse (FEMA 356 2000); as well, the risk of life-threatening injury is low, and it should be possible to repair the structure. In the NEESWood research project it is estimated that the LS damage state corresponds to a peak interstory drift in the range 1–2% (Pei et al. 2010, Christovasillis et al. 2007). For this range of drift displacement, the wood framing and OSB/Plywood Sheathing would experience permanent differential movement of adjacent panels, corner sheathing nail pullout and cracking/splitting of sill/top plates. The design criterion must be based on input energy spectra corresponding to 475 years return period; the maximum interstory drift displacement Δ_{max} for each floor should not exceed 2% of the story height.

Collapse Prevention. FEMA 356 defines CP as the post-earthquake damage state that includes damage to structural components such that the structure continues to support gravity loads but retains no margin against collapse (FEMA 356 2000). Structural damage potentially includes significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation, and —to a more limited extent— degradation in vertical-load-carrying capacity. The structure may not be technically practical to repair. In the NEESWood research project it is estimated that the CP damage state corresponds to a peak interstory drift in the range 2–4% (Pei et al. 2010, Christovasillis et al. 2007). For this range of drift displacement, the wood framing and OSB/Plywood Sheathing would experience splitting of sill plates equal to anchor bolt diameter, cracking of studs above anchor bolts and possible failure of anchor bolts. The design criterion must be based on input energy spectra corresponding to 970 years return period; the maximum interstory drift displacement Δ_{max} for each floor should not exceed 4% of the story height.

The design value of the stiffness k and the yielding force F_y of a given plate can be stated separately in terms of the geometrical parameters b, t and l. For the sake of simplicity, all the flexible steel plates can be designed alike since the possibility of choosing separately the number of plates for each story allows sufficiently for a tailored design. For any series array of flexible steel plates, the joint yielding force and stiffness are equal to those of each plate times the number of plates:

$$V_{yi} = F_y \, n_{pi} \qquad\qquad K_i = k \, n_{pi} \tag{8}$$

In equation (8), V_{yi}, K_i and n_{pi} are the yielding shear force, the shear stiffness and the number of flexible steel plates of the i-th story in a given direction (two opposite façades), respectively. The energy that can be dissipated in the whole building in a given direction cannot be obtained by merely adding the capacities of each story; it depends on the distribution, among the different stories, of the dissipated energy and on the accidental eccentricities between their centers of mass and rigidity. To cope with this issue, a number of formulations to select the variation, along the building height, of the design yielding forces of the steel members have been proposed; in this paper the complex approaches in (Akiyama 1985) and (Benavent-Climent 2011) or the simpler method in (Foti et al. 1998) are considered. The formulations of Akiyama and of Benavent-Climent are based on a number of nonlinear time-history analyses and aim to obtain a rather uniform distribution of the cumulative inelastic deformation ratio η in each level along the building height. In both studies, the yielding force is normalized with respect to the weight above that floor:

$$\alpha_i = \frac{V_{yi}}{\sum_{j=i}^{N} W_j} \tag{9}$$

N is the number of floors. According to Akiyama, the distribution of α_i obeys to two polynomial expressions:

$$\frac{i-1}{N} < 0.2 \qquad\qquad \frac{\alpha_i}{\alpha_1} = 1 + 0.5\,\frac{i-1}{N} \tag{10}$$

$$\frac{i-1}{N} \geq 0.2 \qquad \frac{\alpha_i}{\alpha_1} = 1 + 1.5927\frac{i-1}{N} - 11.8519\left(\frac{i-1}{N}\right)^2 + 42.5833\left(\frac{i-1}{N}\right)^3 - 59.4827\left(\frac{i-1}{N}\right)^4 + 30.1586\left(\frac{i-1}{N}\right)^5$$

In the study by Benavent-Climent, the variation of α_i obeys to an exponential equation:

$$\frac{\alpha_i}{\alpha_1} = \exp\left[\left(1 - 0.02\frac{k_1^t}{k_N^t} - 0.16\frac{T_F}{T_G}\right)\frac{i-1}{N} - \left(0.5 - 0.05\frac{k_1^t}{k_N^t} - 0.3\frac{T_F}{T_G}\right)\left(\frac{i-1}{N}\right)^2\right] \qquad (11)$$

In equation (11), k_i^t is the lateral stiffness of the i-th timber floor, T_F is the fundamental period of the building in the direction under consideration and T_G is the corner period of the V_E design spectrum; T_G period separates the initial growing and the horizontal branches. Since the lateral resistance of the timber frame is neglected compared to the steel system, in the considered timber platform frame buildings it can be assumed reasonably that the vertical distribution of the lateral timber stiffness is constant; therefore: $\frac{k_1^t}{k_N^t} = 1$.

The formulation described in (Foti et al. 1998) relies on representing the effect of the expected seismic action in terms of equivalent static forces; then, the yielding force at each story V_{yi} is selected as a given percentage of the corresponding internal shear forces in each of the arrays of flexible steel plates. Usually, the considered percentages range in between 50 and 100%.

If the vertical variation of α_i is selected according to the aforementioned researches (Akiyama 1985, Foti et al. 1998, Benavent-Climent 2011), the cumulative inelastic deformation ratio η in each floor is expected to be rather uniform along the building height. The aforementioned approximate closed-form relation between η and μ ($\eta = 4(\mu - 1)$, (Akiyama 1985)) indicates that the distribution of μ will be also rather uniform; since $\mu = \delta_{max} / \delta_y$, if δ_y is the same in all the floors, the vertical distribution of the maximum transverse displacement $\delta_{max,i}$ will be also approximately constant. Hence, according to equations (3) and (8), the energy dissipated in the whole building is:

$$E_D = \frac{1}{\gamma}\sum_{i=1}^{N} E_{di}\, n_{pi} = \frac{1}{\gamma}\sum_{i=1}^{N} 4\, F_y\, \delta_{max,i}\, n_{pi} = \frac{1}{\gamma}\sum_{i=1}^{N} 4\, V_{yi}\, \delta_{max,i} = \frac{4\,\delta_{max}}{\gamma}\sum_{i=1}^{N} V_{yi} \qquad (12)$$

In equation (12), γ is a safety factor accounting globally for the irregular distribution of $\delta_{max,i}$ among the different stories and for accidental eccentricities.

- The proposed design approach consists in selecting the geometrical and mechanical parameters of each flexible steel plate (b, l, t and f_y) and of the number of plates per level and per direction n_{pi} to fulfill the requirements for IO, LS and CP conditions. Given the strong interdependence among the involved quantities, the design process must be carried iteratively following a trial-and-error strategy. In future stages of research, more refined design criteria will be derived. They will be based on extensive nonlinear analyses of the building equipped with the protective steel system and undergoing seismic inputs representative of the actual seismic hazard conditions. The behavior of the timber members and the steel structure will be described with advanced numerical models. The steel members other than the flexible steel plates can be designed later; they should yield after the flexible plates and be significantly stiffer. This higher stiffness allows minimizing the difference between the interstory drift Δ and the transverse displacement δ, see Figure 4.c.

CONCLUSIONS

This work proposes a new seismic protection system for timber platform frame regular buildings, both for new construction and retrofit. The building is embraced with an outer steel structure, designed to take the lateral forces arising from the seismic excitation; the steel structure comprises four frames (located in each of the façades) and of collectors, which hug the building at each story. Each steel frame includes newly designed hysteretic energy dissipators; those devices are intended to absorb most of the damaging energy fuelled by the ground motion, thus protecting the rest of the steel structure and, mainly, the timber elements. Noticeably, the proposed system might hold any other type of dissipative devices. The members of the protective steel structure are designed in the framework of the Performance-Based Design following an energy-based design approach; the IO, LS and CP levels are considered. Application examples, testing and advanced numerical simulation of the proposed energy dissipators are currently in progress; the final aim of the research is to derive accurate design criteria. The proposed system can be also useful for strong winds.

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