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Trends in research and design of structures with seismic protection systems

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Abstract

This article describes some of the new trends observed in research and design of structures equipped with seismic protection systems (SPS). This field of earthquake engineering has almost 40 years of formal development and techniques have gained increasing acceptance through time in design practice, especially due to the successful performance of these structures during the severe ground shaking of recent earthquakes. The time window considered in this investigation of the field is 10 years, defined mainly by research and practical applications after the important cluster of severe earthquakes starting with Sumatra in 2004, Haiti and Chile in 2010, and Japan and New Zealand in 2011. A brief overview of the research done in the field around the world is presented first to provide a general context to the reader, and also identify possible trends in research and practice that could lead the future development of the field. The article then highlights some results and applications derived from current research in earthquake behavior and seismic protection of buildings in Chile, followed by an overview of the design methodologies available in the literature. Furthermore, some details and results are provided for a robust design procedure that has been used to design a number of buildings equipped with SPS technologies in Chile. And finally, a list of the Chilean structures known up to now to be equipped with seismic protection was generated as future reference for the local development of the field.

Keywords: seismic protection, state of the art, seismic isolation, energy dissipation, structural control

1 Introduction

Structural and nonstructural damage is intrinsic to the current earthquake design philosophy of conventional structures when subjected to moderate and severe earthquake motions. The total cost worldwide attributed to earthquakes in the last 10 years amounts to about 220 billion dollars [1]. In Chile, the Superintendence of Values and Securities (SVS) in a report ([2]) estimated damage in the last Chilean earthquake in 2010 in about US$ 30 billion, or an equivalent annual cost of 18% of the country´s GDP. Some other simpler estimates show that average annual earthquake damage in the last 50 years in Chile goes easily above a billion dollars per year, i.e., of the order of half of a point of the national GDP. The normalization with GDP is important since as the country develops, the value exposed to earthquake damage also increases.

Although structural and infrastructure damage have important societal and environmental consequences, as critical as this damage is the loss of function of structures. Unfortunately, preserving functionality during a severe earthquake is difficult in general if the system has been designed by conventional building codes that are intended to dissipate the vibration energy by plastic deformation of the structural members, i.e. through damage. Seismic protection systems are used to tackle this problem by providing structures with supplemental devices that substantially reduce or avoid damage depending on the chosen solution. The design goal is to guarantee continuous functionality in earthquake loading cases a conventional structure does not.

Several previous studies have performed a state-of-the-art and state-of-the-practice in the use of seismic protection systems in civil structures. An comprehensive review of seismic protection systems was presented earlier by Housner et al. [3]. This review linked structural control with other fields of control theory, and suggested future high-priority research topics on the field. Some of these topics were: (1) devices and algorithms for passive, active, semi-active, and hybrid control of nonlinear systems; (2) development of innovative, high-performance, and intelligent material systems; and (3) near-field strong earthquake ground motion issues impacting structural control applications. Another similar state-of-the-art on seismic protection systems presented a brief historical outline of the technologies and highlighted their advantages and limitations [4]. A more recent work summarized the current practice and recent development of passive energy dissipation systems [5], and discussed the current code-based approach used to analyze and design structures that incorporate these systems. Other more specific state-of-the-art reviews are available in the extensive literature on the field, and are dedicated to specific seismic protection systems such as semi-active control [6], seismic isolation [7], shape memory alloy devices [8], and piezoelectric materials [9].

The field of seismic protection technology has over 40 years of development, and it would be impossible to achieve an insightful overview within the space limitations of this article. Consequently, the strategy followed herein is to concentrate in the developments of the last 10 years, motivated in part by the cluster of important earthquakes that have occurred along the Pacific Rim since the Sumatra earthquake in December 2004. Because of the several manuscripts that have addressed in depth the state-of-the-art in this field, our goal is to complement such work with some of the newer ideas contained in more recent publications.

The reader is probably familiarized with the customary classification of SPS into three categories depending on their requirement of external power: (1) passive, (2) semi-active, and (3) active. In turn, passive seismic protection systems are classified into: (i) seismic isolation systems, which filter out the high frequencies of the input and damp out the motions by usually placing an interface with low lateral stiffness and high internal damping between a super and substructure; and (ii) energy dissipation, which transforms and dissipates the vibration energy of a structure into heat (e.g., friction dampers) or plastic deformation (e.g., metallic dampers). One special class of energy dissipators are tuned mass dampers (TMDs), which are inertial dampers connected to a single point of a structure and usually synchronized with one of the natural frequencies of the structure. TMDs can also be interpreted as a structural element (dissipator), which is an important dynamic amplifier of internal damping.

On the other hand, practical applications of semi-active systems are becoming more common today. These systems have the capacity to adapt to the changing conditions of the motion in the structure by continuously modifying their own mechanical behavior based on some feedback signal. They need very low external power supply to work, but require some feedback signal to feed into a control algorithm, which in turn generates the control signal that modifies the constitutive behavior of the passive device. One can conceptually understand them as a large collection of passive dampers from which the structure can choose the most convenient one at each instant of time. Furthermore, the combination of a passive and active system is known as a hybrid system [10]. Because forces (and power requirements) are extraordinarily large in active systems, the direct use of active control in structures has been very limited.

Elastomeric, lead-plug, high-damping, and sliding bearings are the most frequently used devices. Rubber isolators with steel plates were first implemented in a school in Macedonia in 1969 [11], and lead-plug rubber isolators in New Zealand in the late 1970s [12, 13]. Later, high damping rubber bearings (HDRB) appeared and used different compounds to increase the internal damping ratio to the 10-20% range at 100% shear strain [10]. These HD materials were first produced in 1982 by the Malaysian Rubber Producers' Research Association in the UK [14], and their first implementation occurred in 1985 for the Foothill Communities Law and Justice Center in California [15]. Sliding bearings, which basically use friction between surfaces to resist lateral loads, are also widely used nowadays. The best known of these devices is the friction pendulum system (FPS), developed and tested in the late 1980s, and has evolved into its current variations (double and triple FPS). Other common types of isolation systems include pot-type bearings, which combine elastomeric and friction type bearings, other kinematic bearings, and rubber isolation in combination with passive energy dissipation [11], e.g., spring-type systems, sleeve-pile isolation, and resilient-friction base-isolation. Isolators can be understood as a combination of a component that provides lateral flexibility acting in parallel with any kind of an energy dissipator.

TMDs are inertial devices that use a small mass (as compared to the weight of the structure) connected to a single point of the building through a linear or non-linear damper. The concept of TMD dates back to the early 1900´s ([16, 17]). Their use was extended to control multiple modes by using multiple TMDs as first proposed in 1988 [18]. Moreover, TMDs have been used in conjunction in some cases with seismically isolated buildings [19] since these structure have the advantage of presenting an strongly predominant isolated mode. Several strategies for optimally placing and tuning the TMDs to the structure have been developed and can be found elsewhere [10]. Another related application are Tuned Liquid Dampers (TLDs), which dissipate energy through turbulence and friction of a fluid interacting with the wall or other elements within a container. Its civil engineering applications began in the 1980s [20], and TLDs are further divided into: (1) sloshing dampers, which dissipate energy by the flow through meshes and rods, and are tuned by modifying the container size or depth of the liquid; and (2) column dampers, which generate damping flowing through an orifice, and are tuned by changing the column shape or air pressure.

A concise mechanical comparison between friction, metallic, viscoelastic, and viscous fluid dampers can be found in the literature [5]. Friction dampers were first proposed in the late 1970s [21, 22]. The concept came from car brakes, and has been applied to concrete shear walls [23, 24], and to braced frames [25]. After successful experiments, several types of these devices have been implemented, such as the Sumitomo friction damper, which uses a copper alloy lining pad typically connected to a chevron bracing [26]; slotted bolted connections, where the slots of the bolts are aligned to the direction of loading [27]; and energy-dissipating-restraint [28], which is a self-centering device where the sliding force is proportional to the displacement. On the other hand, metallic dampers use the hysteretic work of yielding metals to dissipate the vibration energy. This concept was first proposed in the early 1970´s [29, 30], where several configurations of mild steel dissipators were tested, such as torsion of rectangular bars, flexure of short beams, and rolling of U-shaped strips. Among many typologies, a well-known type of metallic damper is the ADAS [31]. Another more recent metallic damper that has been extensively investigated and used in practice is the unbonded buckling restraint brace (BRB) used mostly in braced structures to limit axial forces on the braces and protect connections by allowing axial yielding of a steel section confined by an external casing with a small slack between the yielding element and casing [32]. Its popularity comes mainly from its symmetrical behavior in tension and compression.

Viscoelastic dampers dissipate energy through shear deformations of materials such as rubber, polymers, and glassy substances. One of their first applications in civil engineering was the wind-induced vibration reduction system of the Twin Towers of the WTC in New York city in 1969 [33]. Closely related to these dampers, viscous fluid dampers produce work through the motion and shearing of a highly viscous fluid, which also produces heating of the fluid. The first implementation of these devices in civil engineering was in the 1970s [34]. The first industrial device was made by GERB Vibration Control in the 1980s, and was used together with base isolation [35]. Many other viscous devices exist, such as the viscous damping wall [36] developed by Sumitomo (Japan), which requires very high viscosity fluids.

Semi-active systems were proposed as early as the 1920s as automobile shock absorbers [37]. Civil engineering applications began in the 1980s, and include: semi-active TMDs to counteract wind induced vibrations in tall buildings by changing the damping force of the tuned mass [38]; sloshing TLDs consisting of baffles orientated in real time in order to change the tuning of the device [39]; semi-active column TLDs, where a variable orifice controls damping [40]; semi-active friction dampers, which use electromechanical actuators [41]; and piezoelectric friction dampers [42]. Other types are the electrorheological (ER) and magnetorheological (MR) dampers. ER dampers use fluids containing dielectric particles that align when subjected to an electric field offering flow resistance; this change can occur within milliseconds and is completely reversible. In the early 1990s, ER dampers were proposed to protect base-isolated structures from long-period motions [43], usually attributed to near-source earthquakes, and to reduce the acceleration of a shock isolation platform for naval applications [44]. Similarly, MR dampers use the power of a coil to change the shear characteristics of an MR fluid, and in the 1990s several experiments were carried out to show their usefulness [45–49]. MR dampers have been proposed to control the vibration of train suspension systems [50] and have been used in scaled models of bridges to evaluate their effectiveness [51–53]. MR fluids have also been used to control damping in column TLDs designed for wind-induced vibration mitigation of tall buildings [54]. Furthermore, semi-active viscous fluid dampers may also change their properties by controlling the opening of orifices inside the piston and were first discussed in the early 1990s for bridge applications [55, 56] based on prior successful laboratory tests [38].

Given this general overview of some developments of the field, it is convenient to start with a very brief description of the equations of motion that govern the vibration and seismic protection problem of structures. Indeed, seismic protection may be understood as an automatic control problem of a structure (plant), which seeks passive or semi-active forces to equilibrate the moving masses by modifying directly or indirectly the stiffness, damping, and mass of the system. After that, a brief overview of the developments in seismic protection over the past 10 years is presented with the goal of identifying future trends in the field. Some recent local research results associated with these new trends are also presented and discussed. The article concludes with a presentation of the advances in design of structures with Seismic Protection Systems (SPS) with especial emphasis on a methodology that has been recently proposed, and a summary of the Chilean implementation of SPS.

2 Formulation of the problem

The equation of motion of a MDOF linear structure with inelastic SPS can be written as a dynamic equilibrium problem:

$$\boldsymbol{M}\ddot{\boldsymbol{u}}_t(t) + \boldsymbol{C}\dot{\boldsymbol{u}}(t) + \boldsymbol{K}\boldsymbol{u}(t) + \boldsymbol{L}^T\boldsymbol{f}_d(t) = \boldsymbol{0} \tag{1}$$

where $\boldsymbol{M}, \boldsymbol{C}, \text{and } \boldsymbol{K}$ are the mass, viscous, and stiffness matrices, respectively; $\boldsymbol{L}$ is the kinematic transformation matrix between the DOFs $\boldsymbol{u}$ and the damper deformations $\boldsymbol{v} = \boldsymbol{L}\boldsymbol{u}$, and $\dot{\boldsymbol{v}} = \boldsymbol{L}\dot{\boldsymbol{u}}$; $\boldsymbol{f}_d$ is the vector of damper forces; $\boldsymbol{u}_t(t) = \boldsymbol{u}(t) + \boldsymbol{r}\boldsymbol{u}_g(t)$ is the vector of total displacement of the masses relative to an inertial frame; $\boldsymbol{u}$ is the vector of relative displacements of the building masses relative to the ground; $\boldsymbol{u}_g$ is the displacement of the ground relative to an inertial frame; and $\boldsymbol{r}$ is the kinematic transformation matrix between the ground displacements and the DOFs $\boldsymbol{u}$ of the structure. Multiplying Equation (1) on the left by $\dot{\boldsymbol{u}}^T$, and integrating with respect to time, the following absolute energy equation is obtained [57]

$$\begin{aligned}\underbrace{\frac{1}{2}\dot{\boldsymbol{u}}_t^T(\tau)\boldsymbol{M}\dot{\boldsymbol{u}}_t(\tau)\Big|_0^t}_{E_k(t)} + \underbrace{\int_0^t \dot{\boldsymbol{u}}^T(\tau)\boldsymbol{C}\dot{\boldsymbol{u}}(\tau)d\tau}_{E_v(t)} + \underbrace{\frac{1}{2}\boldsymbol{u}^T(\tau)\boldsymbol{K}\boldsymbol{u}(\tau)\Big|_0^t}_{E_s(t)} + \underbrace{\int_0^\tau \dot{\boldsymbol{u}}^T(\tau)\boldsymbol{L}^T\boldsymbol{f}_d(\tau)d\tau}_{E_d(t)} \\ = \underbrace{\int_0^t \dot{\boldsymbol{u}}_g^T(\tau)\boldsymbol{r}^T\boldsymbol{M}\ddot{\boldsymbol{u}}_t(\tau)d\tau}_{E_i(t)}\end{aligned} \tag{2}$$

where E_i is the seismic input energy; E_k is the total kinetic energy; E_v is the viscous damping energy; E_s is the elastic energy; and E_d is the SPS energy [57], i.e. $E_k(t) + E_v(t) + E_s(t) + E_d(t) = E_i(t)$. After enough time passed the end of the ground motion, say $t = t_f$, the kinetic and elastic strain energies vanish, and Equation (2) implies that $E_v(t_f) + E_d(t_f) = E_i(t_f)$. Therefore, for an elastic structure, the final input energy must be dissipated by the internal damping and the work done by the SPS. If the energy transferred to the structure exceeds this dissipation capacity, the structure will be damaged and ended with residual deformations. Seismic protection systems can significantly reduce these effects using two strategies: (i) by increasing E_v and E_d with energy dissipation devices, and (ii) by reducing the input (demand) through modifications of the dynamic characteristics of the system (e.g., seismic isolation).

3 World context in seismic protection

A detailed literature review was carried out by inspecting all seismic protection articles published in four leading journals of the field over the past 10 years. The journals studied were Earthquake Engineering & Structural Dynamics, Engineering Structures, Structural Control and Health Monitoring, and Earthquake Spectra. Shown in Fig. 1 is the number of articles on passive, semi-active, and active systems as the percentage of all published articles. It can be seen that passive seismic protection, which is the oldest, has been more studied than semi-active and active protection. Please note that these percentages can add to more than 100% since a single article may contain more than one device. Although it is older than the rest, seismic isolation has been studied more than twice the next technology in the ranking, which are TMDs. The fourth highest is MR dampers, which is the most studied non passive system. This comparison is just a reference since energy dissipation is subdivided into many classes of devices, while isolation groups different families of devices. The plot on the right (Fig. 1) classifies these articles further into different observed thematic trends; those articles that did not fit into this classification were classified as “other”.

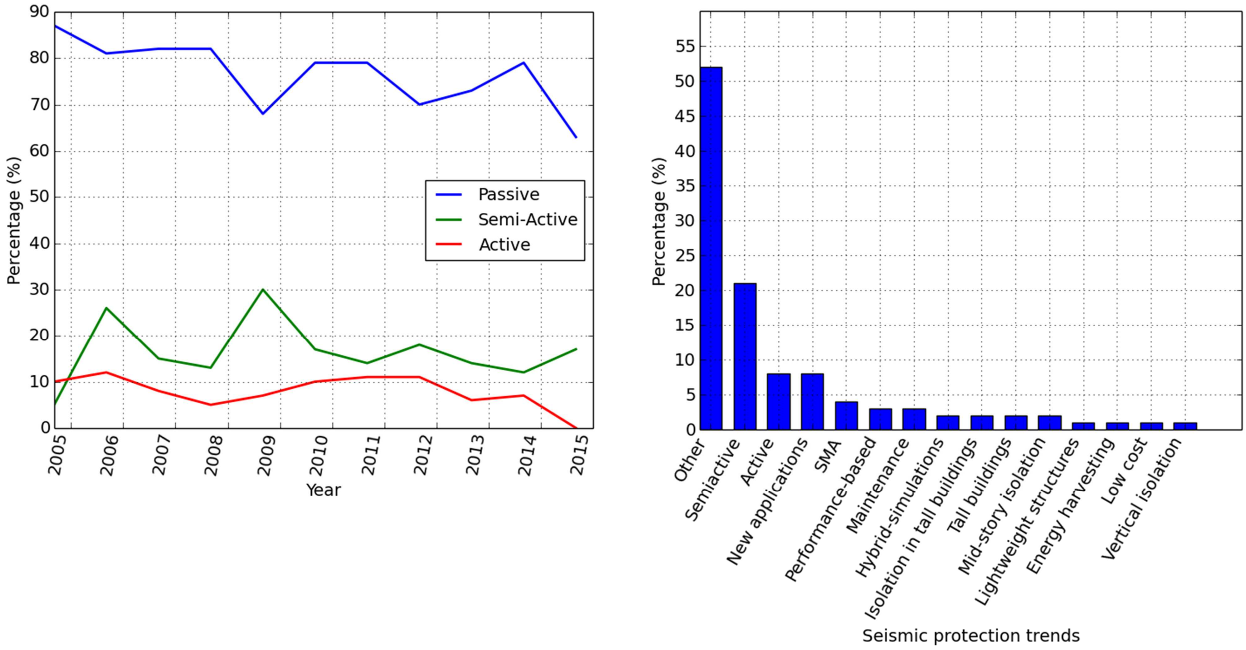


Fig. 1 – Publication trend in passive, semi-active, and active seismic protection over the last ten years

It is apparent that publication trends in Fig. 1 are quite steady over the last 10 years, and regardless of the successful performance of structures with SPS, it has not triggered any special concentration in research. Indeed, by grouping TMDs and TLDs, all the semi-active systems, and all the passive dampers, the distribution becomes more homogeneous (Fig. 2). In this case for example, energy dissipation as a topic, excluding TMDs and TLDs together that are the second highest, has been studied in average in 26% of the searched journal papers, while seismic isolation, which is the highest, in 36%.

The study performed on the bulk of literature not only included a statistical analysis in terms of the publications according to the classes shown, but also searched for plausible future trends in the field. Although it could be debatable how identify such trends, since they are not only a result of a statistical analysis but rather, more often than not, from seminal work that creates fertile lines of research that lead to advancing knowledge. At the risk of being somewhat speculative in some cases, the ideas that come up in several publications and are more frequently been discussed these days in the field are:

(a) **Seismic isolation of tall buildings**. The construction of tall buildings is naturally increasing in earthquake regions worldwide as a result of population growth. Although not as effective as in massive squat structures, and within physically reasonable limits, seismic isolation may be effective in reducing the earthquake response of tall buildings. In such buildings, the uplift forces generated at the isolation system need to be limited to prevent stability failures and cavitation of rubber [58]. Several solutions can be proposed to reduce tension besides increasing the size of the isolators [59] such as outrigger elements or a grid of RC walls capable of distributing forces at the foundation and isolation system [60]. In these studies it was proven that horizontal displacements of the isolation system remain within the displacement capacity of conventional bearings. Another case [61] showed that soil inelasticity provides supplemental energy dissipation, and that the effects of rocking isolation may play a relevant role in building survival providing extra dissipation.

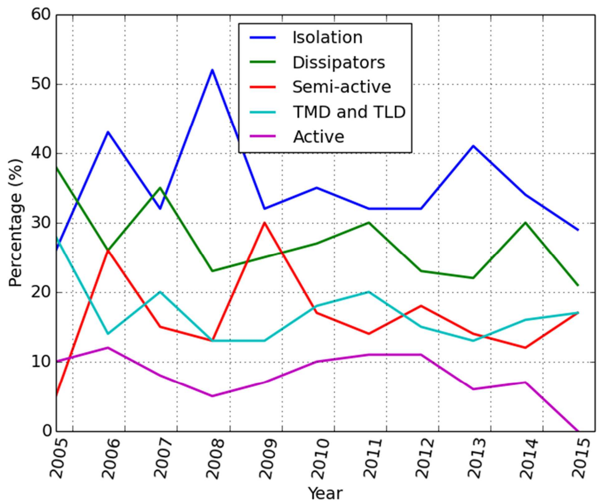


Fig. 2 – Trends of SPS publications over the last ten years and grouped as indicated in the legend

Another possibility for tall buildings is intermediate-story isolation. Some studies have dealt with multi-story structures suggesting the use of FPS [62], or semi-active[63] and active control [64] of nonlinear base-isolated structures. Intermediate-story isolation has to account for the specific characteristics of the isolation system and its location in height [65]. A building structure tested with base and mid-height isolation systems showed that the latter has smaller fundamental modal quantities than base-isolated buildings [66–68]. Some recent studies have tried to minimize the maximum floor acceleration of buildings, while constraining the maximum interstory drift, the maximum base displacement, and the total seismic isolation cost [69].

The protection toward vertical motions is also critical for some nonstructural systems [70], and cannot be achieved by conventional elastomeric and frictional bearings. The vertical response still remains as a topic of study for base-isolated buildings, perhaps more so than in conventional fixed-base buildings since isolation is used in some structures to attain high performance requirement levels [71]. It can been shown that vertical isolation can reduce the vulnerability of nonstructural components in base-isolated buildings [72], and some innovative isolators have been developed for that purpose [73]. Earthquakes, but also human behavior [74, 75] and machine vibrations [76] also induce vertical vibrations in a structure, and vertical TMDs has also been proposed.

(b) **Semi-active systems**. Structures are becoming increasingly complex systems in their operation, with the structure becoming less and less important in terms of cost. As a consequence, structures need to adapt better to different environmental conditions, among them, earthquake induced vibrations. Because semi-active systems use small amounts of energy to change the system properties in real time, they are an excellent alternative to control the building response as long as they are usually implemented in conjunction with other systems, such as TMDs [77, 78]. Since TMDs are large mechanical amplifiers of deformation and damping, the use of semi-active systems together with TMDs may be an important application in the future.

(c) **Industrialized and precast structures**. Industrialized and precast structures are attractive construction technologies that increase productivity and quality in building construction. Almost inevitably, the resulting structures are prone to non-ductile behavior due to the types of connections used among components, unless conventional wet connections are emulated at the site. This results in a strong design and construction disadvantage of prefabricated structures in seismic countries. The use of SPS provides a tremendous opportunity to improve the seismic behavior of such structures by providing two important conditions: (i) a reduced seismic demand, and (ii) the necessary energy dissipation capacity [79]. Many technical issues still need to be investigated in order to effectively generate a methodology that fully takes advantage of the merge of precast construction and seismic design technologies.

(d) **Self-centering protection systems.** Because one of the main goals in implementing SPS is to get rid of structural and non-structural damage and guarantee operational continuity for the structure, the residual condition of the building after the earthquake turns out to be very important. Although dampers are designed as true *fuses* within a structure, and after a strong earthquake should be replaced, in some cases the inelastic condition of the dampers may keep the structure from returning to its initial condition. Moreover, users are not familiar in general with the seismic design philosophy, and relate any kind of damage as a design failure. Consequently, the development of self-centering SPS has brought substantial attention from a number of researchers [80, 81].

One case of self-centering devices are the shape memory alloys (SMAs), which can be passive, semi-active, or active components [82]. SMAs have two advantages: (i) they return to their predetermined shape upon heating, and (ii) they present super-elasticity and can undergo large inelastic deformations without residual deformations upon unloading. This phenomenon provides ideal centering capabilities that can be used in passive control of structures subjected to earthquakes [83–86]. Besides, it can be shown that SMA wires supply recoverable hysteretic behavior [87, 88] and serve as an additional restoring force [89–92]. Several researchers have studied the behavior and performance of structures with SMAs [93, 94].

(e) **Seismic protection of low-cost and lightweight structures**. Perhaps one of the most relevant challenges in the development of future SPS technology, is to make it accessible to everyone, like any other construction material or construction technology available. Low-cost seismic isolation for instance has been a subject of research over the years [95, 96] and solutions range from complete foundation interventions [97], the use of friction and rubber-soil mixture interfaces [98], and also lower cost devices [99, 100]. In general, the energy dissipation solutions available cover a larger range of prices, and it is simpler to foresee low-cost SPS ideas implemented in the future. The application of seismic protection to lightweight structures is directly related to low-cost protection, and still has several technical and cost challenges, since these structures are less significant in size and cost, and conventional solutions are prohibitive for these structures. Technically, to be able to isolate lightweight structures, devices with stiffness independent of their height and kinematic isolators seem most appropriate [101]. Moreover, the large improvement observed in very low friction materials represents an interesting avenue to develop solutions for future lightweight structures.

4 Recent research results in SPS technology

This section contains a quick overview of some of the most relevant research results obtained by the principal author and a group of master and doctorate students over the past 5 years. The emphasis is in the work of this research group, but some appropriate references are made when needed to the results of other national researchers. This research is strongly related to the trends presented earlier and can be classified in the following three aspects: (i) building performance; (ii) new devices; and (iii) design methodologies.

4.1 Building performance

Understanding the detailed performance of different structural systems under severe seismic conditions remains one of the main goals of earthquake analysis. Thus, the research performed during the last years has focused on understanding the behavior of two types of systems, the so-called free-plan buildings, and the typical Chilean shear wall buildings. Although it was not evident from the dynamic properties and design characteristics, the former performed without damage during the 2010 Chile earthquake, while some of the latter developed a brittle failure induced by excessive compression in shear walls [102, 103].

Because free-plan buildings have structures with small lateral stiffness—their fundamental period is smaller than that of a frame structure of the same height—initial research was dedicated to understand their earthquake behavior that was uncertain since no history on the performance of this typology was available. In particular, the research aimed to represent the dynamic response of these systems with a sufficiently simple model that would enable the designer to try out different structural configurations and SPS solutions at initial stages of the design process. In the model synthesis process, the relative importance of the different structural components was evaluated discovering the surprising effect of the warping stiffness of the shear wall core, and the very relevant bending stiffness of the floor slab that couples, especially for lower modes, the shear wall core and the perimeter frames of free plan buildings structures [103]. Thus, a new simplified column model was developed for free plan buildings that is significantly less time consuming and whose errors are in general less than 15% [104]. The second idea was to use an adaptive neural fuzzy inference system (ANFIS) model [105] to quickly predict the seismic response of these types of buildings. For that, the central idea was to model the structure as a linear combination—with combination factors to be determined—of two extreme structural modeling cases [105]: (i) a rigid out-of-plane floor diaphragm of beams and slab in bending, and (ii) and infinitely flexible out-of-plane floor diaphragm of beams and slab that would result in cantilever-like deformations for all vertical elements. Given a proper calibration of the neural network, analyses can be carried out very fast due to the simplicity of the model, which is crucial when a large number of earthquake runs are needed as required for optimal SPS design.

In case of reinforced concrete shear wall buildings, several studies have been performed since February 27th, 2010. First, a detailed study of the seismic behavior of 8 buildings in the city of Concepcion was done after the earthquake and has been published [103, 106]. The purpose was to understand their failure, propose solutions for their stabilization, study patterns of damage and correlate those with the seismic code demand, and collect all possible perishable earthquake data. The data collection effort was then extended to a set of 43 buildings in Chile and a formal statistical analysis process of the data has been performed and published [107]. Results show that the axial load ratio (ALR) played a fundamental role in the brittle damage of these buildings. Furthermore, statistical analysis of the collected data from the suite of damage building shows that damage was localized generally at lower levels of buildings and that there was a strong correlation between damage and soil quality [103, 107]. A study of the performance of specific walls was also initiated using state-of-the-art inelastic modeling tools for shear walls (e.g., Diana). In order to reproduce this damage, two approaches were taken: (1) two-dimensional finite element inelastic models; and (2) three-dimensional nonlinear fiber models. Results show that to reproduce the actual failure in 2D inelastic models, both vertical and lateral displacements need to be accounted for in the imposed displacement pattern [108]. Two dimensional models were carried out in Diana using shell elements with well-known inelastic stress-strain constitutive laws for concrete, but using rather simple constitutive laws for steel.

Also, a three-dimensional inelastic fiber model for shear walls was recently developed [109]. The model is capable of generating objective results for a big range of loading conditions, and reproduces well very different experimental cyclic tests of different shear wall cross-sections reported in the literature. The model took into account the full nonlinear 1-D stress-strain constitutive relationship for concrete, and also fracture, buckling, and Bauschinger´s effect on steel bars. Moreover, the model accounts for inelastic shear deformations depending on the current axial load in the element. The first part of this investigation related to the inelastic cyclic modeling of reinforced concrete shear walls has being published elsewhere [109], and currently the inelastic dynamic response of 3D shear wall buildings is being considered.

4.2 New devices

Although the global concept of seismic protection has gained important traction in the profession over the last 10 years, it is evident that certain device characteristics are key in facilitating the incorporation of seismic protection technologies in building construction. Of course, cost and efficiency are two of them, and though very important, the final decision to incorporate seismic protection resides also in other aspects, like applicability of existing solutions, maintenance characteristics, device durability, physical appearance, magnitude of the structural and architectural intervention needed, commercial availability of alternatives, life and replacement of devices, device residual deformations, etc. Therefore, the work in developing new devices, the so-called hardware component of the SPS, is extraordinarily important and will still evolve significantly in the future.

This research team has tested several types of devices, covering metallic dampers in steel and copper, elastomeric and self-centering isolation and sliders, passive and semi-active friction devices, MR-elastomers, and MR-fluid dampers. A very extensive research work was developed in electrolytic tough pitch (ETP) copper. Initially, Added-Damping and Stiffness (ADAS) copper dampers were developed and tested resulting in very stable cyclic behavior and large energy dissipation capacity that when applied to 6-, 12-, and 25- story planar structures, resulted in drift reductions ranging from 20-40% [110]. Recently, a bi-directional ETP copper-based energy dissipation device has been proposed [111], which is capable of enduring more than one severe earthquake without reaching failure; a good number of other configurations such as shear panel and honeycomb dampers have been designed and tested. These studies successfully reproduced the experimental behavior of copper devices by using different stress-strain inelastic constitutive models capable of representing the cyclic behavior of copper including large deformations and isotropic and kinematic hardening. Applications of EDDs have been developed not only for structures but also for nonstructural components such as partition walls [112, 113].

Seismic isolation of lightweight structures has been addressed mostly with kinematic isolators. One idea for poor foundation soils was a self-centering precast pre-stressed pile (PPP) isolator, first introduced in 2006 as a solution for seismic protection of low-income people housing [81]. The PPP isolator, which crosses the bad layers of soil, consists of a reinforced concrete pedestal with end caps of prescribed rolling shape, and linked to a top and bottom concrete capitals by means of a pre-stressed steel cable which provides the self-centering capacity of the PPP, and bars or EDDs that provide the dissipation capacity. The shape of the rolling surface defines the force-deformation constitutive relationship of the device and can be controlled by the designer. The top capital is connected to the superstructure of the building, and the bottom capital to the foundation system. Different variations can be proposed also for self-centering lintels with the same idea. The cost of the PPP isolation systems was estimated to be around 25-50% of a typical rubber isolation system [81], and since low-income people housing is generally placed in bad soil conditions at the periphery of cities, the PPP isolators also works as a foundation pile. Following this idea, a tridimensional analytical formulation of the PPP isolator was proposed [97] and tested [101]. A simplified design procedure was also proposed, which combined PPP isolators with steel-PTFE sliders in parallel [101].

More recent research on passive devices has focused on modeling the multi-physics of viscous and magneto-rheological dampers [114]. Models developed account for the complete fluid dynamics and thermal behavior inside the damper, and present excellent accuracy with experimental cyclic tests. Experimental evidence also shows that for high frequencies, the compressibility of the fluid must be accounted for in order to reproduce the experimental results. In those cases, the commonly accepted design formula of viscous force being proportional to a power of velocity needs to be modified to include the compressibility of the fluid.

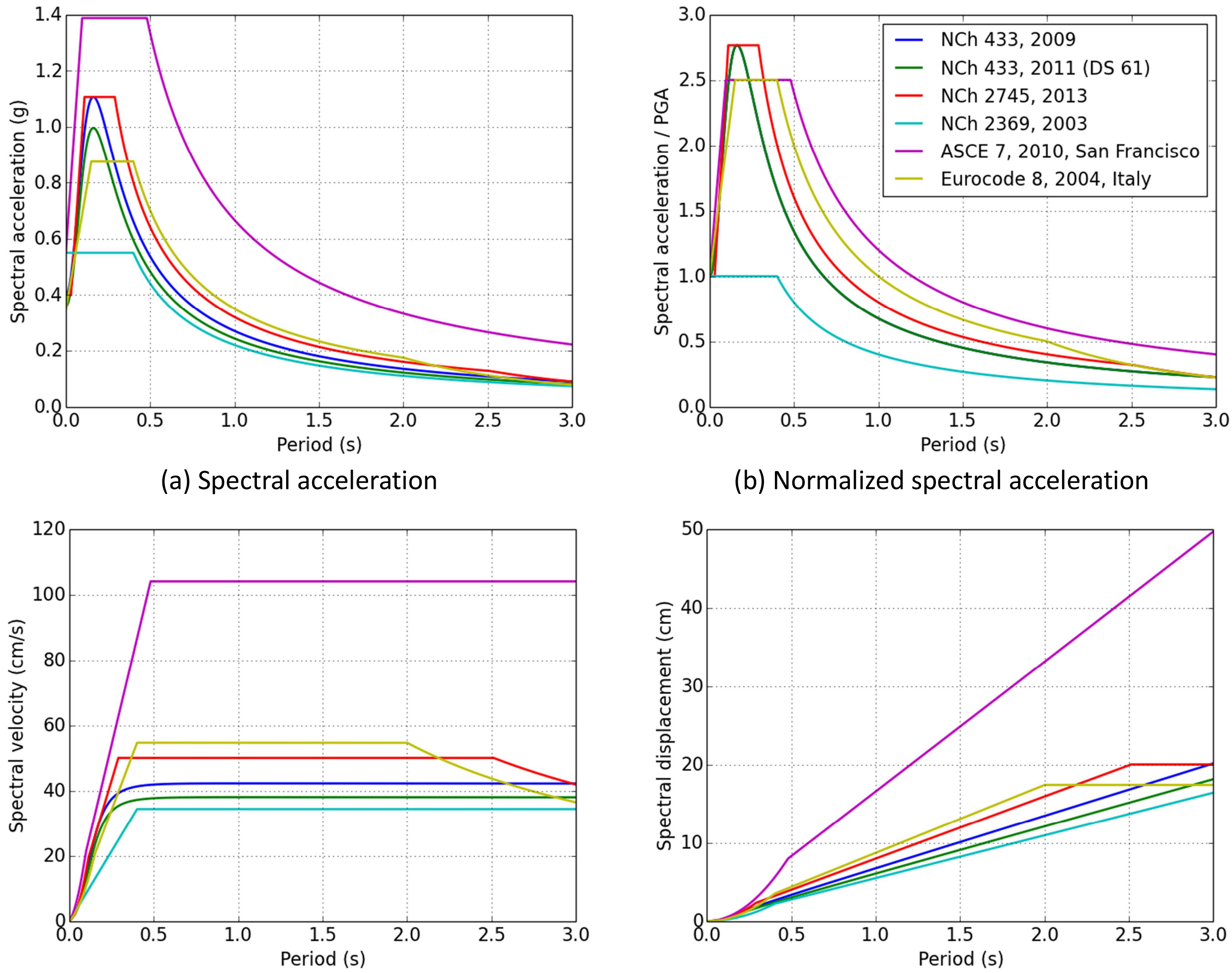
Even though the application of more intelligent devices is just beginning in Chilean practice, research has been generated for semi-active systems, such as: (1) MR dampers [115–118]; (2) semi-active piezoelectric friction dampers [119]; and (3) MR elastomers [120]. MR dampers consists of an MR fluid which properties can be changed from a viscous fluid into a semi-solid material by subjecting the fluid to a magnetic field. One of the advantages of MR fluids relative to viscous fluids is its stable behavior over a broader temperature range [118]. Again, a multi-physics finite element model that couples the Maxwell equations of magnetism with the Navier-Stokes equations for the fluid was develop to parametrically study the effect of different physical geometric design characteristics of the dampers on the force-velocity constitutive relationship, and results were validated experimentally [115]. The design, building, and testing of a prototype MR-damper, specifically designed for one of the two TMDs of a 21-story building located in Santiago, led to the first real-life implementation of such a semi-active SPS in Chile [116, 117]. The experimental validation of the final TM-MR damper assembly on the building was carried out by subjecting the system to pull-back tests of 10 cm of amplitude [116], and the experimental results showed that the MR dampers decreased the peak displacement of the building by 22% relative to the TMD system with no dissipation devices. Another study considered a semi-active elastomer, which consists of a composite material with an elastomeric matrix filled with micro-sized iron particles [120]. This is an initial investigation since the magnetic fields required are still large, but the material is capable of varying its force-deformation constitutive relationship when subjected to a magnetic field and in principle could vary stiffness and damping by means of a simple control system.

One of the drawbacks of regular friction dampers is that their efficiency depends on the selection of the activation force and the ground motion intensity. Semi-active piezoelectric friction dampers overcome this limitation by controlling the normal force of the contact surfaces using piezoelectric actuators. Recent research has proposed a new piezoelectric friction damper that can work in passive or semi-active mode using contact surfaces made of stainless steel and brake pad material [119]. The device is able to vary its initial normal force by a dynamic range factor of 1.9, and hence modify its dissipation force capacity by the same factor.

Other Chilean researchers have carried investigations in the field of shape memory alloys that includes experimental testing [121–125]. SMA bars have shown good dissipation properties with an equivalent damping of 12% [124] and they have also found applications in structural cables showing more energy dissipation and better re-centering capabilities [121]. A SMA damper was informed to reduce floor peak accelerations when tested as part of a braced system; the reduction obtained was around 60% [125]. Also, by means of genetic algorithms, an optimal control of the maximum acceleration in a building together with the roof lateral displacement was numerically studied for an isolated building with magnetorheological dampers [126].

4.3 Design methodologies

The true vehicle to transfer knowledge in structural engineering to the society is through a design algorithm, generally included in codes. Thus, any relevant analytical or numerical capacity to evaluate the structural performance and force-deformation constitutive behavior of the EDDs used in seismic protection need to converge into a formal and coherent design procedure, capable of producing physical solutions that work under the uncertain loading conditions of an earthquake.



(a) Spectral acceleration (b) Normalized spectral acceleration

(c) Spectral velocity (d) Spectral displacement

Fig. 3 – Design spectra for NCh 433, NCh 2745, and NCh 2369 Chilean codes for zone 3 and soil type A ($V_{S30} \geq 900$ m/s); ASCE 7 code for San Francisco and site class A (hard rock, $V_{S30} > 1524$ m/s); and from the Eurocode 8 for zone I in Italy and ground type A ($V_{S30} > 800$ m/s).

A key aspect in any structural design is the definition of the earthquake demand, which varies from site to site. In order to compute the structural response, earthquake excitations need to be taken into account using a design spectrum. Shown in Fig. 3 is a comparison of the design spectra of several building codes: Chilean codes NCh433 modified in 2009 [127], current NCh433 code modified in 2011 by Supreme Decree 61 [128], NCh 2745 code for base-isolated buildings [129], NCh 2369 for industrial structures [130]; ASCE 7 code [131] for the city of San Francisco, California; and Eurocode 8 [132] for Italy's zone I. All spectra are calculated for the best possible soil (i.e., type A) and with an importance factor of 1. Each design spectrum has an associated return period and peak ground acceleration as indicated in Table 1.

Table 1 – Design return period and peak ground acceleration of each design code.

Design code	Design Return Period (years)	PGA (g)
NCh 433, 2009	-	0.40
NCh 433, 2011 (DS 61)	-	0.36
NCh 2745, 2013	475	0.40
NCh 2369, 2003	-	0.55
ASCE 7, 2010	475	0.55
Eurocode 8, 2004	475	0.35

The philosophy of current design methods states that the demand on the earthquake resistant structural system of a building with SPS can be reduced due to their incorporation into the design. In practical terms, the more supplemental damping is introduced into a building—which can be done by energy dissipation devices—the more the design spectrum is reduced [131, 133]. Given this demand, the design of EDDs consists of finding their mechanical properties and spatial distribution. This problem has been dealt with in literature through two approaches: (i) an optimization problem where the designer has to find the EDD distribution and capacities for minimizing some response related function; and (ii) a problem to find an optimal capacity given a spatial distribution of EDDs.

Optimal damper distributions have been widely proposed in the literature, e.g. optimal placement of dampers in height by minimizing both the sum of amplitudes of linear transfer functions [134] and also their peak [135]. The effect of torsion for controlling the response of asymmetric-plan systems, where the geometric and rigidity centers do not coincide in plan—generating stiffness eccentricity—has also been studied in the framework of seismic protection. The idea is to find a spatial distribution on the plan of a structure that balances the response, and it has been suggested that more damping is required along the flexible edge of the plan, as it was shown for linear [136] and nonlinear [137] structures. Other studies have develop the concept of torsional balance as an optimal design criteria, and have found that the optimal eccentricity of dampers not only depends on the structural eccentricity, but also on the frequency content of the excitation and the amount of supplemental damping [138]. Shown in reference [139] is that for two-story frames with asymmetric plan, the EDD distribution is similar for linear and inelastic structural models, and the same applies to linear and inelastic damper models.

Speaking strictly about optimization methodologies, several methods have been proposed and used, such as those based on control theory, H_∞ and H_2, which lead to optimal spatial distributions and capacities of dampers [140]. The advantage is that this solution neither depends on the loading conditions nor on the structural response. This is not the case of genetic algorithms that require multiple runs of structural analyses that are generally inelastic [141] or that deal with uncertainty in the seismic loading [142]. Genetic algorithms have not only been used for passive dampers but also for intelligent ones [143]. In the latter case, genetic algorithms were combined with fuzzy logic controllers and stochastic linearization method to allow an efficient solution. Reference [144] used genetic algorithms for magnetorheological dampers to solve a multi objective optimization problem given by several performance based design targets. There are other methods such as optimal control using a linear quadratic regulator [145], Cutting Planes Method for solving the optimization problem with Lagrange multipliers [146], and gradient based algorithms [135] among others.

ASCE 7-10 provisions define four procedures for the analysis and design of structures equipped with SPS: equivalent lateral force analysis, response-spectrum analysis, nonlinear static-pushover analysis, and nonlinear dynamic response history analysis. The latter can always be used, but other constraints apply for the other three methods that require much less computational effort. These other three procedures use a damping reduction factor for the building response which results from the effective damping of the EDDs. In design practice, the ASCE 7-10 provisions, which can be used for all kinds of dampers, imposes that the lateral resisting system withstands the nominal forces calculated for the unprotected structure, though if some special requirements are met, the minimum base shear can be reduced. On the other hand, the protection system can be accounted for controlling the displacement requirements [131].

If the design of structures with EDDs is not thought as an optimization problem, it can be done by using displacement or energy based approaches. In the former, if the structure and EDDs are modeled with their effective stiffness and damping properties, a linear iteration involving only static analyses can be used for design [147], where given a target displacement and an effective damping ratio, the strength and stiffness are obtained. Another displacement approach is that of performance-based philosophy, as shown earlier for bridge piers with hysteretic dampers [148]. In this case, a displacement or strain index performance is chosen and the structure is transformed into an equivalent single degree of freedom after performing pushover analysis. Then, by means of capacity-demand spectra, and given a required ductility, one can find the required stiffness and yield strength. Among other displacement based design methods, a very well-known one is the capacity spectrum method [133] where a performance point is obtained by equating the inherent equivalent damping of the cyclic response obtained from a pushover curve with the associated damping used to reduce the demand spectrum.

In case of the energy based approaches, they directly account for the accumulated hysteretic energy, and hence, accumulated damage, as opposed to the displacement based approaches. Reference [149] presents a method where an energy demand spectrum is used after transforming the system into a SDOF by using the two modes with the highest mass participation factors. The design can also include the assessment of the structural seismic risk, which is normally calculated using the Pacific Earthquake Engineering Research Center (PEER) framework [150]. This framework has been used to estimate the probability of failure of building equipped with viscous dampers [151], and of three different base isolation systems in a nuclear power plant [152].

5 Design of structures with SPS

This section contains a quick overview of the procedure that has been calibrated with the design of several structures with SPS in Chile. It has been developed over 15 years [153] and continuously improved by different research and real design cases. It would be impossible to describe the procedure in detail herein and the interested reader may look for further details elsewhere [154]. In this study the design flowchart shown in Fig. 4 is presented. This conceptual design procedure is developed for elastic buildings equipped with linear and non-linear EDDs, but it can be readily extrapolated to the case of an inelastic structure. The procedure consists of the following: (i) to select the building key design performance indices (KDPI) (*e.g.*, peak floor displacements, peak inter-story drifts, *etc.*); (ii) to reduce the order of the dynamic structural model through generalized modal or physical coordinates; (iii) to determine the most significant mode according to a significance ratio (*i.e.* the modal significance factor (MSF)); (iv) to reduce the response of interest until a second mode becomes the controlling mode, and so on until a suitable reduction factor for the response is reached; (v) to build iso-performance curves (providing the amount of supplemental stiffness and damping required to reduce the building response to a specific target value); (vi) to compute a height-wise damper distribution and obtain linear equivalent parameters for each story and device; and (vii) to select the EDD parameters and validate by computing the inelastic response of the structure.

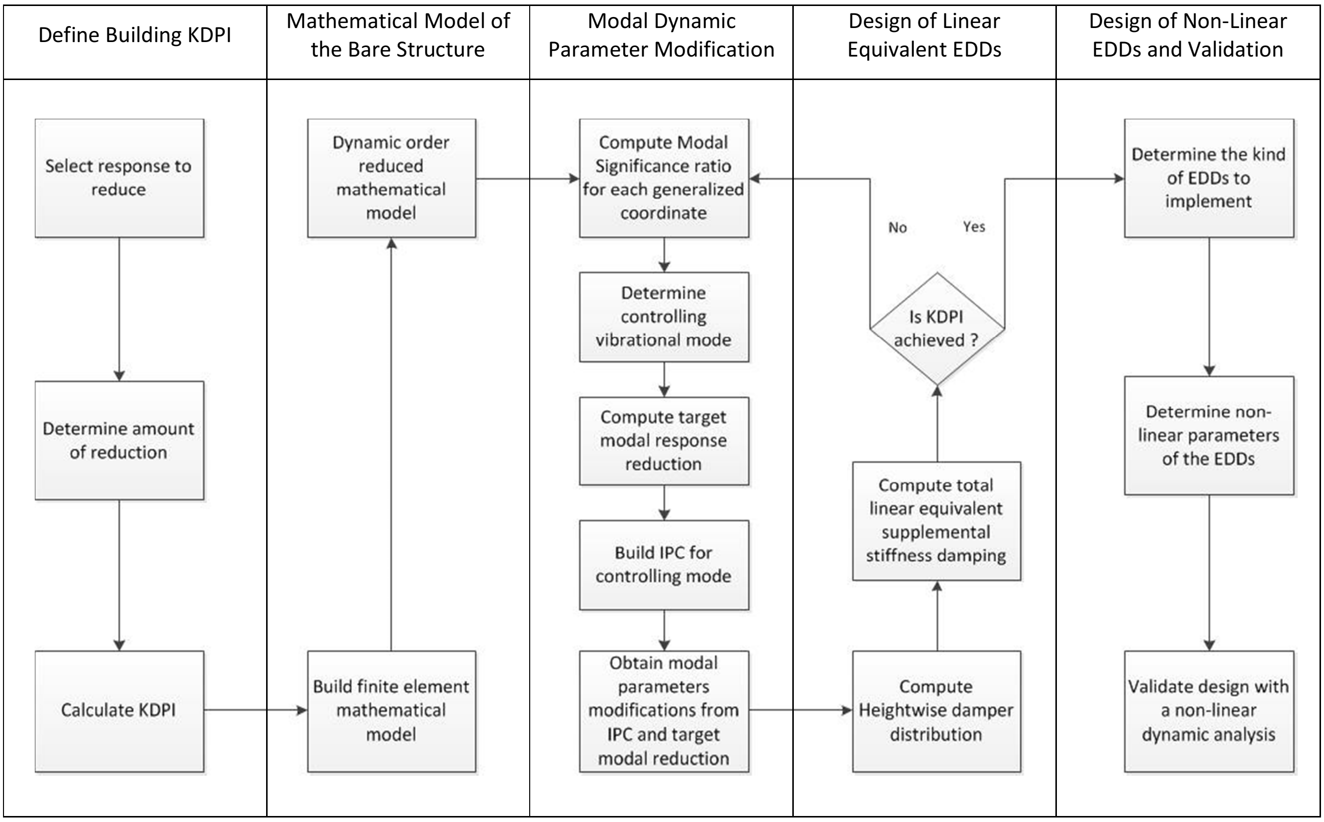


Fig. 4 – Schematic flow diagram of the proposed design procedure [154]

In what follows, only three key aspects of the procedure are highlighted in this article, which turn out to be critical to the procedure: the calculation of the modal significance factor (MSF), the global determination of stiffness and damping, and the definition of the optimal localization of damping.

5.1 Modal significance factor (MSF)

MSF are calculated by decomposing the selected KDPI on the different building modes and choosing the mode with the highest contribution to the KDPI. This parameter is proven to be a robust and extensively applicable method but should consider the non-classical nature of the problem. Thus, the peak response of the KDPI, $Z(t)$ at time t, is decomposed into m modal contributions $Z_m(t)$, where the $m-th$ modal contribution to the MSF, χ_m , is defined as:

$$\chi_m = \frac{Z_m(t)}{\sum_j Z_j(t)} \tag{3}$$

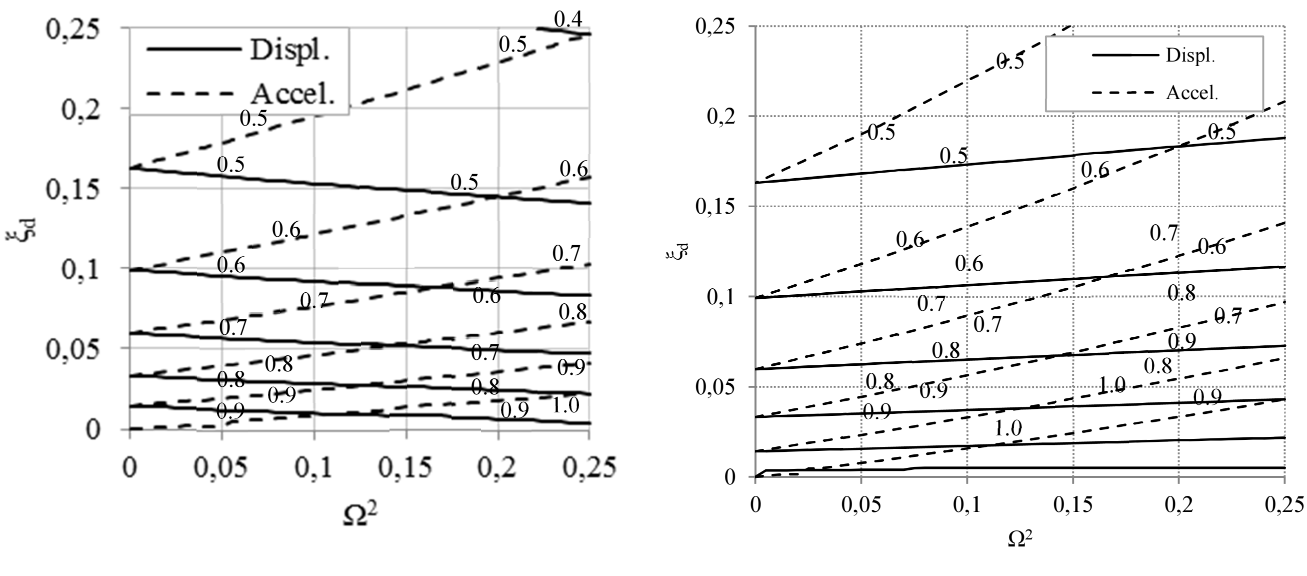
The most significant mode is the one that leads to the largest MSF. It has to be noticed that for non-classically damped modal analysis, this modal decomposition needs to be done using complex modes [154].

5.2 Iso-performance curves

Iso-performance curves (IPCs) allow us to determine the global stiffness and damping that needs to be added to the structure to achieve an optimal KDPI (Fig. 5). Iso-performance curves state geometrically that there is an infinite number of pairs of supplemental stiffness and damping that lead to the same reduction of the KDPI. IPCs are derived from the response of a model of the structure that could be an accurate 3D model, or a lower order SDOF model. For each combination of supplemental damping ratio ξ_d and frequency shift Ω^2, the modal response is determined using this model and its response reduction recorded. The locus of parameters (Ω^2, ξ_d) defining the iso-performance surface corresponding to a response reduction factor R_m defines the IPC. The relationship between these parameters and the frequency and damping shift can be stated as:

$$\omega_f = \omega_0\sqrt{1+\Omega^2} \quad \text{and} \quad \xi_f = \frac{\xi_0+\xi_d}{\sqrt{1+\Omega^2}} \tag{4}$$

where the sub index *0* corresponds to the structure without dampers. Then, by knowing (Ω^2, ξ_d) it is possible to obtain ξ_d and ω_f, which are the $m-th$ new modal parameters used next to compute the required equivalent stiffness and damping in the structure.



(a) T_0=1s (b) T_0=3s

Fig. 5 – IPCs for displacement and acceleration corresponding to the NCh2745 spectrum, soil type I (rock), and Seismic Zone 1.

5.3 The Perturbation Based Optimal Distribution Algorithm (PBODA)

The optimal localization of damping is assessed according to the Perturbation Based Optimal Distribution Algorithm (PBODA) introduced elsewhere [154], which may be summarized as follows: (i) define an initial distribution of dampers $\boldsymbol{\alpha}^0$ that is consistent with the target frequency and damping shift and select a desired tolerance for the iteration—please note that $\boldsymbol{\alpha}^0$ is a distribution and that its components add to one, and that this initial guess is critical for the success of the iteration [154]; (ii) using, say, this distribution after k iteration steps, $\boldsymbol{\alpha}^k$, obtain the overall supplemental linear equivalent parameters k_d^k and c_d^k by the solution of the eigenvalue problem presented in reference [154]; (iii) update the distribution $\boldsymbol{\alpha}^{k+1}$by solving the following optimization problem that maximizes the modal damping of the controlling mode (m):

$$\boldsymbol{\alpha}^{(k+1)} = \max_{\boldsymbol{\alpha}} \xi_m \left(\boldsymbol{\alpha}, c_d\left(\boldsymbol{\alpha}^{(k)}\right), k_d\left(\boldsymbol{\alpha}^{(k)}\right)\right), s.t. \qquad \sum_l \alpha_l = 1, \quad 0 \leq \alpha_l \leq 1 \tag{5}$$

(iv) if the tolerance is not reached, set $k = k + 1$ and iterate back to step (i); otherwise, set $\alpha = \alpha_k$ and exit. Upon exit, compute again the two-parameter eigenvalue problem to get the global stiffness and damping k_d^{k+1} and c_d^{k+1}. These parameters are then introduced into the new stiffness and damping system matrices, and the overall system response can be computed. With the response evaluate the response reduction factor and check if the algorithm ends when the target global response reduction factor R_z is achieved.

5.4 Case studies

The first example used to validate the proposed design procedure is a free-plan building with two interconnected symmetric towers (Fig. 6), and in which the conventional design method assuming classical damping does not work well. The second example is a group of eight shear-wall buildings damaged during the 2010 Chile earthquake. Further details of these structures and results can be found in [155].

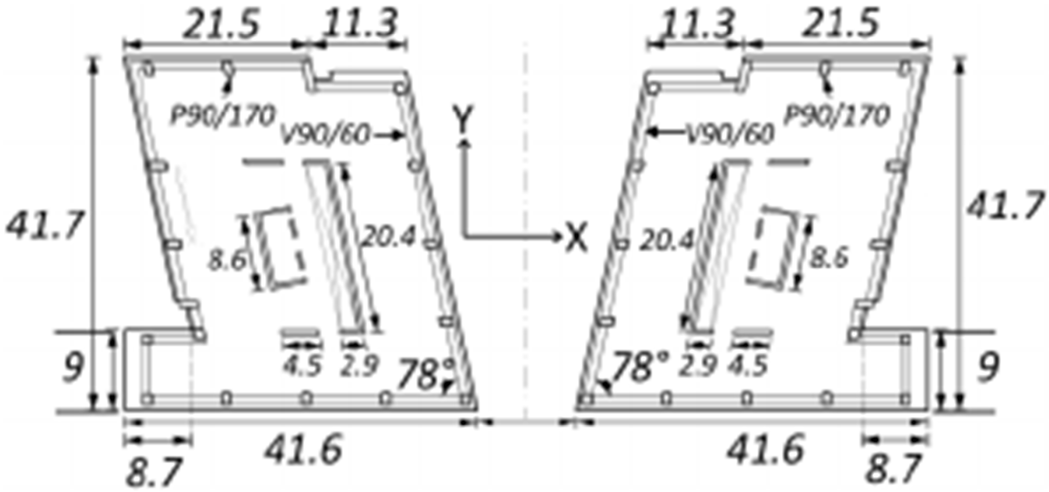


Fig. 6 – Structural plan of the free-plan building

The KDPI chosen for the free-plan building is the maximum lateral displacement of the roof. The target for the global response reduction factor R_z is 0.6. Modal analysis shows that building responses in the X- and Y-directions are spread in several modes. The final optimal linear equivalent supplemental damping and stiffness (c_d, k_d) resulting from the design procedure are obtained in the X-direction after 2 iterations and in the Y-direction after 5 iterations. The final optimal height-wise distribution of damping is presented in Fig. 7 and tends to agree with theory that says that the optimal damper location to control a given mode is to place damping in a single story. Also, a numerical comparison with other NCH2745 compatible ground motions recorded during the March 5, 1985, Chile Earthquake and for the same building design is presented in [154]. Results show the importance of selecting several compatible records for a robust design that ensures a minimum reduction level for the structure.

The second example deals with eight shear-wall buildings damaged during the 2010 earthquake [106]. Again, the target for the global response reduction factor R_z is 0.6. As the structural configurations of the buildings are quite simple [106], one or two iterations were needed for convergence of the design procedure and find the required total linear equivalent supplemental damping and stiffness (Table 2). Similarly to the first building case, the results on these eight buildings prove that the applicability of the design procedure is general and that the linear equivalent method used leads to consistent results when compared to the inelastic building responses.

6 Chilean implementation case

This section describes the Chilean case with seismic protection and includes buildings as well as industrial facilities—bridges and project in the design phase were excluded from this enumeration. Table 3 presents what is to the best of our knowledge the list of building projects with SPS that currently exist in Chile, or which are very likely going to be implemented in 2015. To generate this Table, a request was made to several structural engineers that have designed these structures in the country. The information was received from them in most of the cases and has been directly compiled in this Table. The original Table includes several additional fields but the data could not be obtained in many cases and was left out. It is apparent that seismic isolation is by far the most common system in Chile, used in about 75% of all structures. TMDs are the second most used system with 18%, and the remaining 7% includes all dissipation systems. Only one case is a semi-active proof-of-concept implementation, and no active system has been implemented so far.

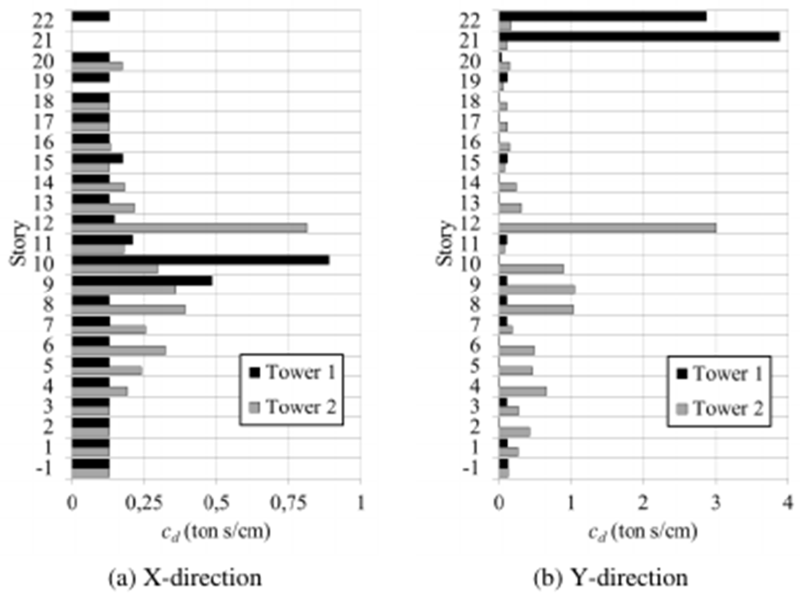


Fig. 7 – Optimal heightwise distribution of damping capacity for the free-plan building

Table 2 – Summary of the total linear equivalent supplemental damping and stiffness required to achieve a response reduction factor $R_Z \leq 0.6$

Building	Period T (s)	Equivalent Damping $\boldsymbol{c_d}$ (ton s/cm)			Equivalent stiffness $\boldsymbol{k_d}$ (ton/ cm)			Design Iterations X ; Y
		X	Y	$\boldsymbol{R_Z}$	X	Y	$\boldsymbol{R_Z}$	
AA-1	0.71	4.06	3.67	0.59	18.61	21.91	0.60	1 ; 1
AH-2	0.70	6.31	10.39	0.59	43.18	125.88	0.42	1 ; 2
CM-3	0.80	6.99	3.01	0.62	44.25	22.48	0.58	1 ; 1
TL-4	0.77	2.95	1.43	0.58	15.06	7.94	0.58	1 ; 1
PR-6	0.50	1.73	2.03	0.56	11.20	21.22	0.56	1 ; 1
PP-7a, RT-8a	0.36	3.85	1.49	0.59	40.38	22.17	0.59	1 ; 1
PP-7b, RT-8b	0.34	2.98	1.52	0.54	55.42	16.93	0.58	1 ; 1
TO-9	0.93	1.70	6.50	0.58	8.04	39.40	0.55	1 ; 1

Table 3 – Building and industrial projects with SPS in Chile organized alphabetically

N	Project	Use	N° of stories	Location	Protection system
1	Centro Anacleto Angelini	Educational	11	Macul	43 isolators y 13 sliders
2	Centro de Distribución Sodimac Lo Espejo	Industrial	1	Lo Espejo	309 elastomeric and 127 frictional dampers
3	Cerro Colorado, Torre A	Residential	16	Las Condes	2 isolated TMDs
4	Cerro Colorado, Torre B	Residential	14	Las Condes	2 isolated TMDs
5	Clínica Cruz Blanca	Hospital	7	Santiago	212 isolators, 86 sliders
6	Clínica UC San Carlos de Apoquindo	Hospital	5	Las Condes	52 isolators
7	Costa Laguna La Portada	Residential	22	Antofagasta	2 isolated TMDs
8	DUOC UC Santiago Centro	Educational	5	Santiago	60 isolators, 21 sliders
9	Edificio ACHS Viña del Mar	Office	7	Viña del Mar	25 isolators
10	Edificio Amura	Residential	21	Antofagasta	41 isolators
11	Edificio Comunidad Andalucía	Residential	4	Santiago	8 isolators
12	Edificio Angamos Oriente	Residential	21	Antofagasta	33 isolators, 8 sliders
13	Edificio Angamos Poniente	Residential	21	Antofagasta	35 isolators, 7 sliders
14	Edificio Cámara Chilena de la Construcción	Office	25	Las Condes	Pendular TMD
15	Edificio Chacay	Office	6	Temuco	10 isolators
16	Edificio Ciencia y Tecnología UC*	Office	22	Macul	22 isolators
17	Edificio CIO Chuquicamata de Codelco	Office	2	Calama	10 isolators, 13 sliders
18	Edificio Civic	Residential	23	Concepción	2 isolated TMDs
19	Edificio Corporativo Komatsu Cummis	Office	6	Quilicura	16 isolators
20	Edificio Corporativo Sodimac	Office	6	Santiago	76 isolators
21	Edificio Data Center Claro	Industrial	3	Colina	110 isolators
22	Edificio Data Center Sonda	Industrial	-	Santiago	90 FPS isolators
23	Edificio Deloitte	Office	16	Las Condes	56 viscous dampers
24	Edificio Geocentro Agustinas	Residential	35	Santiago	2 isolated TMDs
25	Edificio Idahue	Residential	10	Concepción	40 isolators
26	Edificio Jardines de Infante	Residential	17	Ñuñoa	2 isolated TMDs
27	Edificio José Joaquín Vallejos	Residential	16	Copiapó	2 isolated TMDs
28	Edificio Las Heras (4)	Residential	18	Concepción	256 metallic dampers
29	Edificio Los Castaños	Residential	20	Viña del mar	Seismic isolation
30	Edificio Neruda	Office	8	Huechuraba	20 isolators, 2 sliders
31	Edificio Magnus II	Office	5	Huechuraba	31 isolators, 12 sliders
32	Edificio Manchester	Residential	9	Temuco	37 isolators, 10 sliders
33	Edificio Marina Pai Hue	Residential	10	Pucón	13 isolators
34	Edificio Minvu Serviu de Antofagasta	Office	7	Antofagasta	37 isolators, 2 sliders
35	Edificio Las Condes Capital	Residential	16	Las Condes	2 isolated TMDs
36	Edificio Las Condes Capital	Office	20	Las Condes	46 viscous dampers
37	Edificio Nueva La Dehesa	Office	8	Lo Barrenechea	42 FPS isolators
38	Edificio Nuevo Poniente	Residential	22	Viña del Mar	2 isolated TMDs
39	Edificio ONEMI	Office	3	Santiago	16 isolators
40	Edificio Parque Araucano	Office	22	Las Condes	2 pendulum TMDs

41	Edificio Parque Manuel Rodríguez	Residential	-	Calama	-
42	Edificio Patio Mayor	Office	6	Huechuraba	36 viscous dampers
43	Edificio Portofino*	Residential	24	Antofagasta	36 isolators
44	Edificio San Agustín	Educational	4	Macul	53 isolators, 16 sliders
45	Edificio Titanium	Office	52	Vitacura	45 metallic dampers
46	Edificio Torre Capital	Office	24	Temuco	20 frictional dampers
47	Edificio Parque San Damián (2)	Residential	32	Vitacura	2 isolated TMDs each
48	Edificio Tucapel	Residential	33	Santiago	2 isolated TMDs
49	Edificio VULCO	Office	2	San Bernardo	12 isolators, 6 sliders
50	Edificios Condominio Parque	Residential	11	Vitacura	16 metallic dampers
51	Estanque GNL Mejillones	Industrial	-	Mejillones	501 isolators
52	Estanque GNL Quinteros (2)	Industrial	-	Quinteros	260 isolators each
53	Facultad de Química, U. de Concepción	Educational	3	Concepción	18 isolators, 18 sliders
54	Fiscalía Talcahuano	Office		Talcahuano	31 isolators
55	Generador GNL Mejillones	Industrial	1	Mejillones	1 isolator, 4 sliders
56	Generador Termoeléctrica Andina	Industrial	1	Mejillones	1 isolator, 4 sliders
57	Horno de fundición planta Llay Llay	Industrial	-	Llay-Llay	80 isolators, 18 sliders
58	Hospital Clínico U. de Los Andes	Hospital	7	Las Condes	118 isolators
59	Hospital de Antofagasta	Hospital	9	Antofagasta	280 isolators, 139 sliders
60	Hospital de Talca	Hospital	9	Talca	177 isolators
61	Hospital del Trabajador ACHS	Hospital	7	Providencia	32 isolators
62	Hospital Dr. Gustavo Fricke	Hospital	9	Viña del Mar	193 isolators
63	Hospital Exequiel González Cortés	Hospital	6	San Miguel	162 isolators y 38 sliders
64	Hospital HUAP	Hospital	3	Santiago	33 isolators y 19 sliders
65	Hospital La Florida	Hospital	5	La Florida	224 isolators
66	Hospital Las Higueras	Hospital	7	Talcahuano	169 isolators
67	Hospital El Carmen (Maipú)	Hospital	5	Maipú	347 isolators
68	Hospital Militar	Hospital	5	La Reina	164 isolators
69	Intendencia de Talca	Office	11	Talca	18 isolators, 15 sliders
70	Mirador del Santuario	Residential	8	Valdivia	Seismic isolation
71	Muelle Coronel	Industrial	1	Coronel	96 isolators
72	Ñuñoa Capital	Residential	28	Ñuñoa	Seismic isolation
73	Planta DTP de SQM	Industrial	9	María Elena	21 isolators
74	Proyecto Marconi 10 pisos (2)	Residential	10	Los Angeles	12 isolators each
75	Proyecto Marconi 8 pisos (4)	Residential	8	Los Angeles	12 isolators each
76	Security Vida	Office	17	Huechuraba	28 viscous dampers
77	Silos Termoeléctrica Andina	Industrial	-	Mejillones	24 isolators
78	Torre Uno	Office	11	Temuco	20 isolators, 10 sliders
79	Templo Bahai*	Temple	1	Peñalolén	10 isolators
80	Viviendas Sociales de Paniahue	Residential	4	Santa Cruz	7 isolators, 21 sliders

(*) Currently under construction;

7 Conclusions

This article described some of the trends observed in global research and design of structures equipped with seismic protection systems (SPS), but concentrated exclusively on the last 10 years of development in the field. It also summarizes some results of the SPS research done in Chile by the principal author and the graduate students, presents an overview of a new widely applicable design procedure that has been calibrated for the past 15 years, and shows the current state of building and industrial implementations with SPS in the country. From a very detailed literature review, it is concluded that there is no clear evidence of neither particular changes in emphasis in research work, nor a definite boom of SPS technology research relative to other fields in earthquake engineering despite the great success observed in the seismic performance of structures with SPS during large recent earthquakes. Some promising trends that could become increasingly relevant in the near future are seismic isolation of tall buildings, the more frequent use of semi-active (and smart) SPS, a better alignment of industrialized (and precast) construction techniques and SPS, the use of self-centering SPS and the minimization of damage, and the expansion of SPS toward the protection of low-cost and lightweight structures. It is also concluded that the proposed design procedure for structures equipped with SPS worked very well in all 9 building cases considered as test beds, and due to its simplicity relative to other static and dynamic inelastic models, it could be useful for the engineering profession and future energy dissipation seismic codes. Finally, the request done to several Chilean structural engineers enabled to compile a list of almost 80 seismically protected buildings and industrial facilities, which to the best of our understanding is an accurate reflection of the situation as of 2015.

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