SEISMIC RETROFIT OF A PRECAST STORAGE AND DISTRIBUTION FACILITY BUILDING USING ENERGY DISSIPATION DEVICES

M. Rendel [(1)], H. Sady [(2)], R. Zemp [(3)], J.C. de la Llera [(4)], C. Lüders [(5)]

[(1)] *Technical Manager, SIRVE S.A., mrendel@sirve.cl*
[(2)] *Head of Seismic Protection Engineering, SIRVE S.A., hsady@sirve.cl*
[(3)] *Technical Manager, Nüyün_Tek S.P.A., rzemp@nuyuntek.cl*
[(4)] *Principal Investigator, National Research Center for Integrated Natural Disaster Management (CIGIDEN CONICYT/FONDAP/15110017, School of Engineering, Pontificia Universidad Católica de Chile, jcllera@ing.puc.cl*
[(5)] *Co-founder and Senior Engineer, SIRVE S.A., cluders@sirve.cl*

Abstract

This paper reports on several findings that came up during the retrofit of an important 60,000 square meter industrial distribution facility using energy dissipation devices. The industrial facility consists of precast cantilever columns, simply supported precast beams, and metallic horizontal roof braces. The structure underwent minor damage during the 2010 Chile earthquake mainly at the connections between prefabricated elements, beam-column connections, and connections between horizontal roof braces and columns. Several precast storage and distribution centers exhibited catastrophic failures in the 2010 earthquake. Studies performed after the earthquake found that one of the main reasons for the damage and collapse of these structures was the loss of continuity of the roof diaphragm, resulting in large relative displacements between structural elements of the roof, causing detachment of the main precast beams and the subsequent collapse of columns. In order to avoid a potential operational shutdown and the collateral productivity losses of this distribution facility, a comprehensive retrofit solution based on energy dissipation devices was designed to significantly improve the seismic performance of the building, while producing little effect on the facility operations during implementation. The seismic devices were strategically located to achieve two main objectives: (i) take advantage of relative seismic displacements between structural elements to produce energy dissipation and add internal damping to the primary structural system; and (ii) develop ductile connections between structural members at the roof to ensure structural continuity of the diaphragm during the earthquake. Viscoelastic devices were located under the corbels of the beam-column connections to use the angular distortion between beams and columns as a source of energy dissipation. These devices were also designed to resist the beam-column connection forces in case the original simple supported (shear-pinned) connection fails as noted in other failures after the earthquake. Moreover, uniaxial frictional devices acting as mechanical fuses were located at one end of each of the horizontal roof bracing elements. The devices were calibrated to avoid failure of the brace-column connections in tension, and prevent buckling of the braces in compression. Another important advantage of this retrofit strategy, is its low impact on operational continuity since it was designed to avoid changes or temporary removal of structural members while some of the supplemental devices were installed. The distribution center remained fully functional during the retrofit and considered only some partial predefined closures, which were planned largely in advance and in sequence. Conventional retrofit was discarded since it required to disable the fire suppression system and other critical installations. These actions would require a complete removal of goods from the facility and the stop of operations of the distribution center, which would have a prohibitive cost. Therefore, the proposed retrofit strategy resulted in an economical alternative and with an important improvement of the seismic performance of the building, which could also be of interest to other projects. The implementation of this retrofit is currently under progress and is planned to be completed by the end of year 2016.

Keywords: Structural retrofit; Precast frame building; Energy dissipation; Viscoelastic damper; Frictional damper

1. Introduction

Some precast buildings, mostly related to industrial warehouses, suffered important damage as a result of the 2010 Chile Earthquake. A specific structural configuration showed damage in a significant number of buildings. Such configuration is defined by cantilever columns, simple supported precast beams, and horizontal metallic roof braces, where the beam-column connections are “dry” connections. One of the most important distribution centers in Santiago showed incipient damage after the 2010 earthquake, and since other similar buildings nearer to the epicenter suffered catastrophic damage and collapsed, the owner decided to perform a seismic retrofit to enhance the structure’s seismic performance to the minimum cost and smallest impact to the operations. This facility consists of two buildings with a total floor area of approximately 60,000 m^2. The applied retrofit solution is based on energy dissipation devices strategically located to achieve two main objectives: (i) take advantage of relative seismic displacements between structural elements to achieve energy dissipation and add internal damping to the structural system; and (ii) develop ductile connections between structural members at the roof to ensure the structural continuity of the diaphragm during an earthquake. Another important design constraint for the retrofit project was that the distribution center needed to remain functional and only specific sectors could be closed following a predefined and planned sequence. With all the merchandise inside the building, the fire suppression system and other installations could not be disabled during retrofit installation, resulting in more constraints for the design and implementation. As an example, the roof braces could not be replaced.

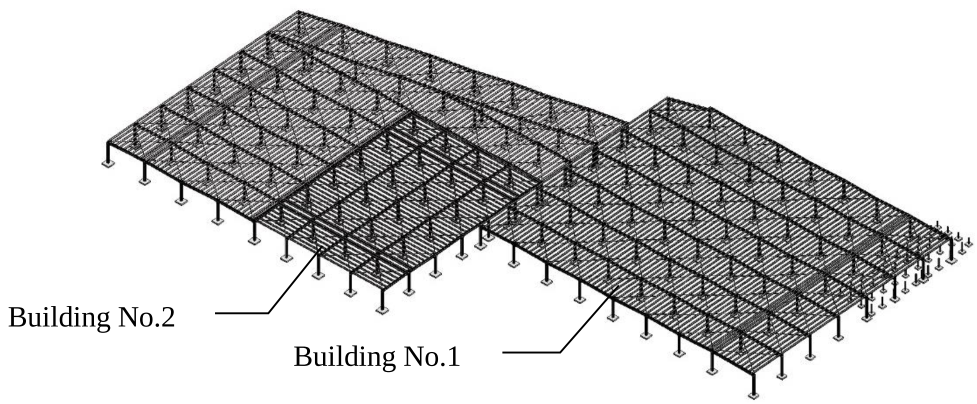


Fig. 1 – General view of the facility (BIM model)

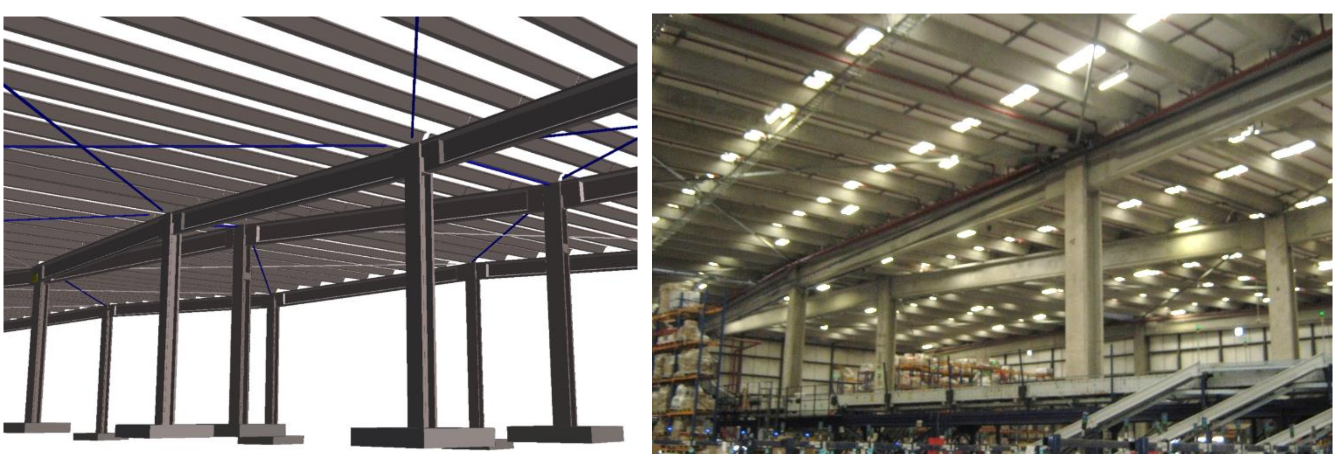


Fig. 2 – General interior view of building No. 2 in BIM model (left) and reality (right)

2. Structural Damage in Precast Warehouses

As stated previously, the damage in similar precast structures are concentrated in the connections between the following structural elements: the “dry” column-beam joints and the connections to steel elements. The source of these failures is mainly explained due of the lack of ductility inherent in the design of these connections and the consideration of a seismic response reduction factor in the analysis. Some of the most frequent types of these failures are shown in Fig. 3, from which only (c) and (d) happened to the buildings considered in this study.

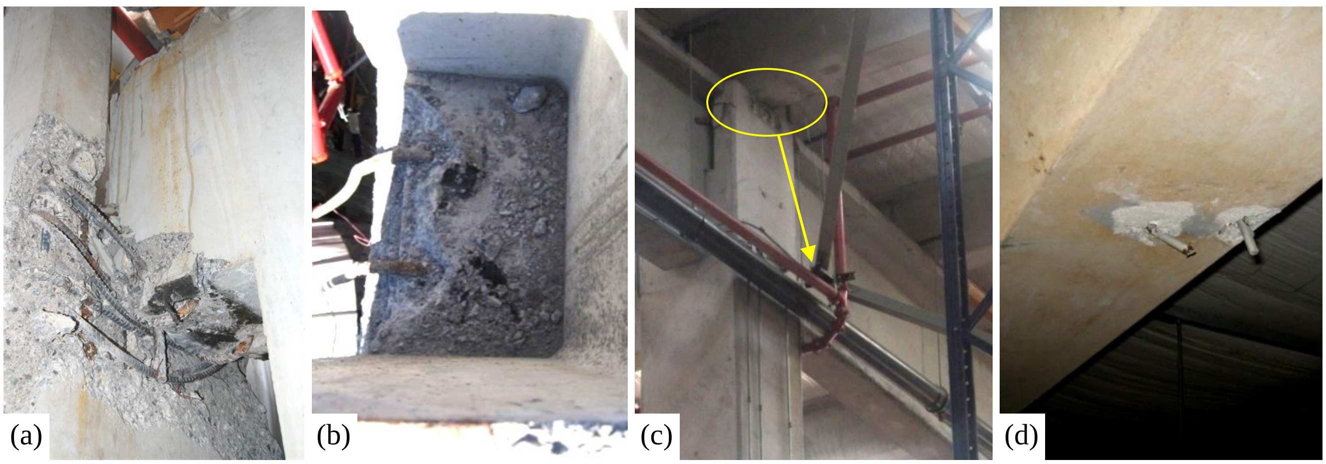


Fig. 3 – Typical failures in precast “dry” joint warehouses: (a) Partial damage in beam-column connections, (b) beam loss of bearing at column seat, (c) detachment of steel horizontal roof brace end-connection from columns, and (d) detachment of steel roof braces from their intermediate vertical supports

Studies performed after the earthquake found that one of the main causes of damage and collapse of these structures is the loss of continuity of the roof diaphragm, as result of the progressive detachment of horizontal roof braces. This failure leads to large relative displacements between structural elements of the roof, causing the detachment of the main precast beams and the progressive collapse of the roof structure.

3. Retrofit Project

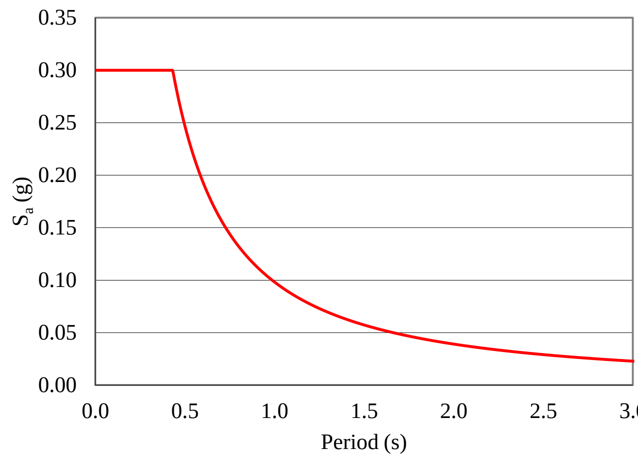
The retrofit project for the buildings was divided into three phases: (i) Structural review of the buildings according to available as-built information and applicable code regulations; (ii) design of the damping devices and their connections to the structure; and (iii) design of conventional reinforcing strategies for structural members that remain overstressed.

3.1 Structural review of the buildings

The review of the as-built structural design of the buildings was performed in 2013. Reviews were conducted using the applicable building codes that were in effect at the time, including: NCh2369.Of2003 [1], NCh430.Of2008 [2], D.S.61-2011 [3] and ACI318-2008 [4]. The structural model was developed using the SAP2000 software [5], considering all the existing structural elements as frames and the weight and mass of non-structural elements as loads. The total dead load of the buildings 1 and 2 is 4872 ton and 6943 ton, respectively. The parameters that define the seismic load to be considered for the structural verification of the structure, and the shape of the corresponding seismic spectrum, are shown in Table 1.

Table 1 – Seismic design load parameters and design spectrum according to NCh2369.Of2003 [1]

Parameter	Value	Commentary
Seismic zone	2	Santiago
Effective maximum ground acceleration, A_o	0.3	Table 5.2
Soil classification	II	
Soil parameter, T'	0.35	Table 5.4
Soil parameter, n	1.33	Table 5.4
Importance factor, I	1	Section 4.3.2
Structural damping ratio, ξ	0.02	Table 5.5
Response reduction factor, R	3	Table 5.6
Maximum seismic coefficient, C_{max}	0.30	Table 5.7
Minimum base shear, Q_{min}	0.12	Section 9.3



The result of the structural review revealed that the design of Building 1 was, in general, acceptable for resisting the code demand. Only one column was approximately 10% overstressed. Lateral deformation was also found to be within acceptable code limits; however, the capacity of the end connection of the horizontal roof braces was found to be designed close to full utilization for the design axial loads, considering a seismic response reduction factor of 3.0. The use of the response reduction factor assumes an amount of ductility for the elements and their connections; however, in this case it was not properly considered. The irregular plan layout of Building 2 (see Fig. 1) produces a more complex seismic response of the structure, and therefore, higher forces in the roof diaphragm horizontal braces were obtained. Due to this, approximately 25% of the axial forces in the braces exceeded the resistance of their end connections considering reduced design forces (see section 3.3.2). Also, approximately 30% of columns exceeded their axial-flexural interaction surface and the lateral deformation of the building exceeded code limits. Thus, the loss of the roof diaphragm during an earthquake in Building 2 was probable, which could lead to the progressive damage of the beam-column connections and the catastrophic collapse of the roof structures, as it happened in the 2010 earthquake in other warehouses located nearer to the epicenter.

From the analysis of the as-built condition, it was determined that a retrofit project that improves the seismic performance of the structures is required, especially for Building 2. It was decided to apply the same solution to Building 1 to maintain equal seismic standards keeping in mind that end connections of the roof braces are not suitable for developing the required ductility.

3.2 Description of the retrofit solution

As stated previously, an important design constraint for the retrofit project, is to provide a low impact on operational continuity of the facility during its construction. Thus, with the proposed solution there is no need to change, or temporary remove, structural members of the buildings while the supplemental devices are installed. The distribution center remains functional during construction and only specific sectors need to be closed following a predefined and planned sequence. The retrofit system is based on energy dissipation devices and was designed to improve the seismic performance of the building. The seismic devices were located strategically to achieve two main objectives:

i. Take advantage of relative seismic displacements between structural elements inherent in this type of construction in an effort to produce energy dissipation and add internal damping to the structural system.

ii. Develop ductile connections between structural members at the roof of the buildings to ensure the structural continuity of the diaphragm during and after an earthquake.

The retrofit system considers three types of interventions: (i) Add viscoelastic damping devices (VED) at the beam-column joints, (ii) add frictional damping devices (FD) at one end of each brace of the horizontal roof diaphragm, and (iii) build axial-flexural reinforcing at the base of some columns.

3.2.1 Viscoelastic devices at beam-column joints

The viscoelastic damping devices considered in this project consists of two high damping rubber layers sandwiched between three steel plates. When relative deformation is induced to the device between the center plate and the two external plates, the rubber layers are subjected to shear deformation and a viscoelastic force-deformation constitutive relationship is obtained. This constitutive relationship is defined by the material properties, the thickness, and total area of the rubber layers.

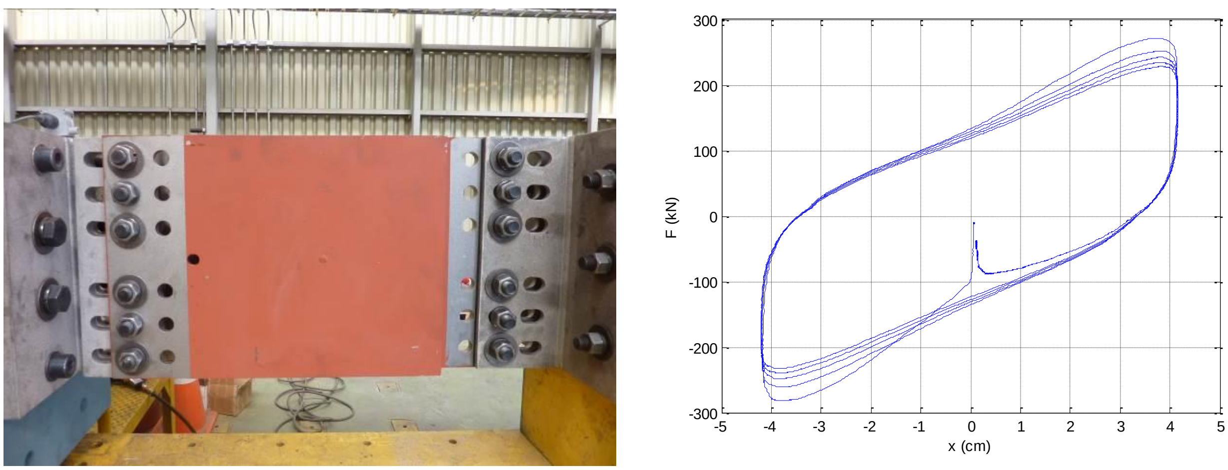


Fig. 4 – Typical Viscoelastic Device (left) and its force-deformation constitutive relationship (right) obtained from laboratory test (DICTUC S.A.)

The cantilever-column structural system has simply supported pretensioned beams between adjacent columns. Beams span typically in one direction with other precast pretensioned elements spanning the opposite direction that support the roof and completing the roof diaphragm (see Fig. 1 and Fig. 2). The viscoelastic dampers in this case are located and designed to comply with two main aspects: (i) Capture the displacements produced by the relative rotation of beam-column joints and add energy dissipation (damping) to the structure, and (ii) have enough resistance to avoid the unmounting of beams from the column corbels if the vertical pin of the precast joint fails or is already damaged (these buildings were affected by the 2010 Chile Earthquake and there is no guarantee on the status of the vertical shear pins at the beam-column seats).

Viscoelastic devices were selected for this task because of their ability of bidirectional in-plane deformation and elastic and dissipative properties, which are both required when the vertical shear pins are damaged (Fig. 5). The lateral displacement of beams is fully restrained at the joint with the column, as they are located inside the flanges of the H shaped columns (see Fig. 7). Therefore, viscoelastic devices will only be demanded for in-plane deformations.

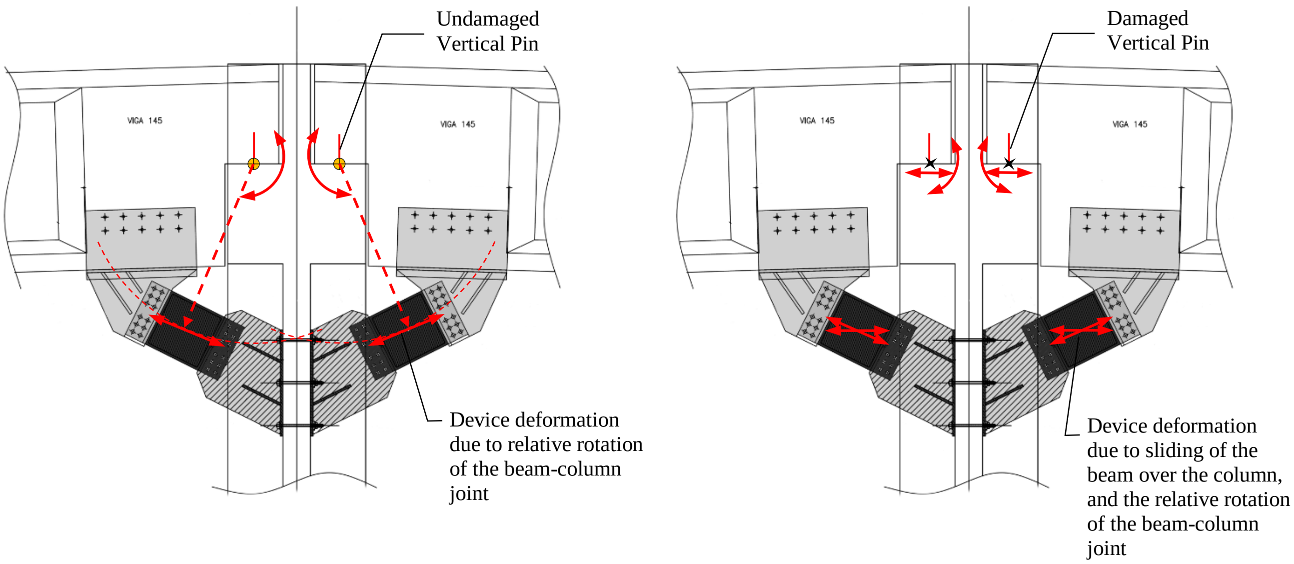


Fig. 5 – Deformation of viscoelastic devices under the two possible beam-column joint states: Undamaged vertical pin (left) and damaged vertical pin (right)

3.2.2 Frictional devices at roof braces end-connections

The frictional devices considered for this project work with the same general principle as the viscoelastic dampers, but replacing the viscoelastic layers with a specific friction material subjected to controlled compression forces. The force-deformation constitutive relationship in this case is determined by the friction materials properties and the compression force at the friction interface (Fig. 6).

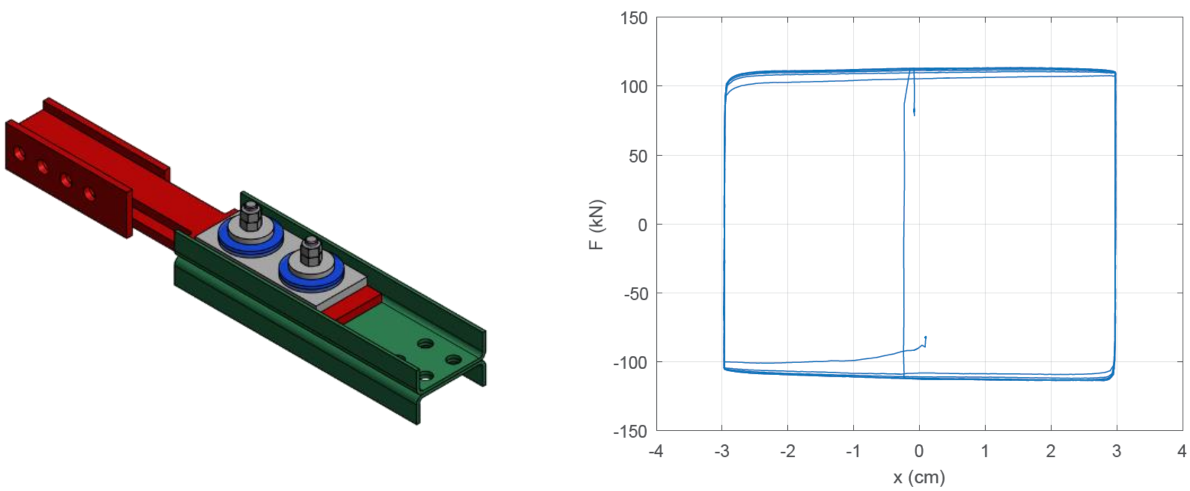


Fig. 6 – Typical Frictional Device (left) and its force-deformation constitutive relationship (right) obtained from laboratory test (DICTUC S.A.)

The most typical failure of roof diaphragms observed after the 2010 Chile Earthquake, was due to the failure of the end-connections or buckling of the roof braces. Therefore, the frictional dampers in this case, located at one end of each horizontal roof brace, were calibrated to become activated before the buckling load or the design load of the end-connections was achieved. Furthermore, the anchors to the concrete columns were reinforced to guarantee their resistance to be higher than the device's activating load.

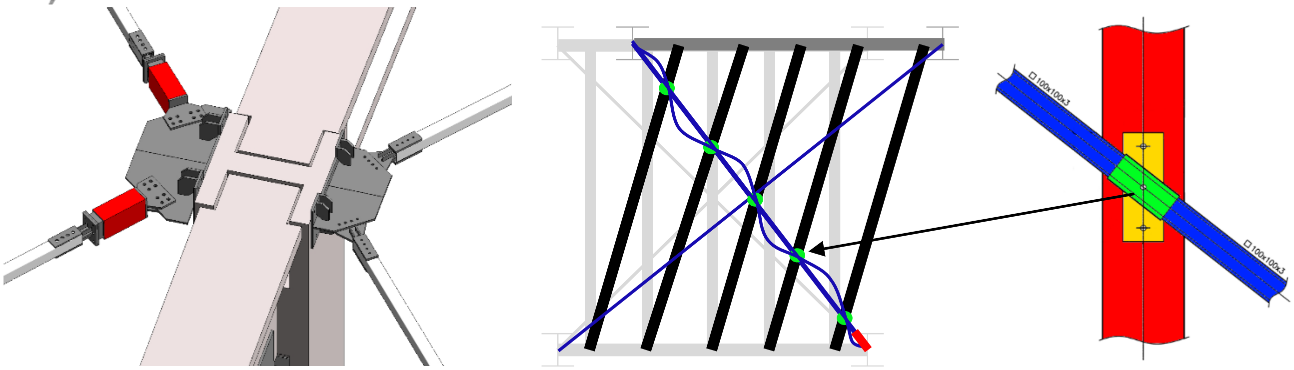


Fig. 7 – Brace-column joint with frictional devices on one side and fixed braces on the other (left), schematic in-plane deformation of a four column quadrant of the building (center), and intermediate axial-sliding support of braces to shorten buckling length (right)

Frictional devices were selected for this task because they are intended to work as a load limiter (mechanical fuse), so the braces will maintain their original stiffness to perform as a rigid diaphragm and only some sliding of the dampers will occur when axial loads get close to the braces' resistance. In this case, some small distortion of the diaphragm is preferred rather than the possibility of losing it progressively.

3.3 Analysis and design of the retrofit system

3.3.1 Structural model

The structural models of the buildings were developed in SAP2000, considering all structural elements as frame elements and creating the nonlinear dampers as links. Since the frame elements are uniaxial (not volumetric), rigid frame elements were used to connect elements to each other at their real 3D geometric locations. It is important to represent the geometry as realistic as possible, since the movement of the energy dissipation devices is directly related to this.

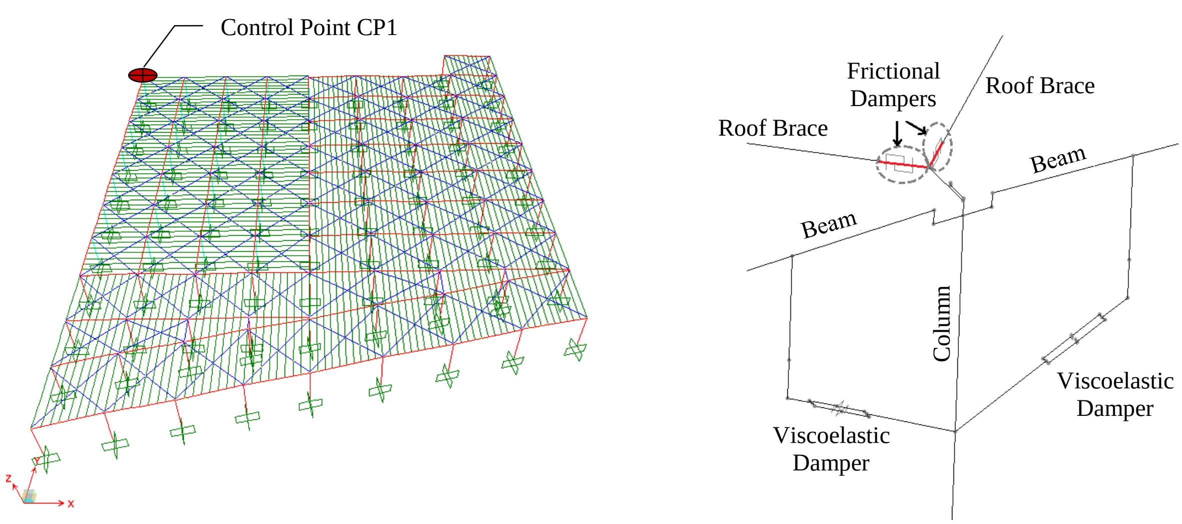


Fig. 8 – Structural model of Building 2 (left), and detail of a typical beam-column-brace joint (right)

The damping devices are located in the structure as indicated in the following layout. As indicated before, and illustrated in Fig. 9, viscoelastic dampers (in blue) are located in all the beam-column joints, and frictional dampers (in red) are located at one end of each horizontal roof brace.

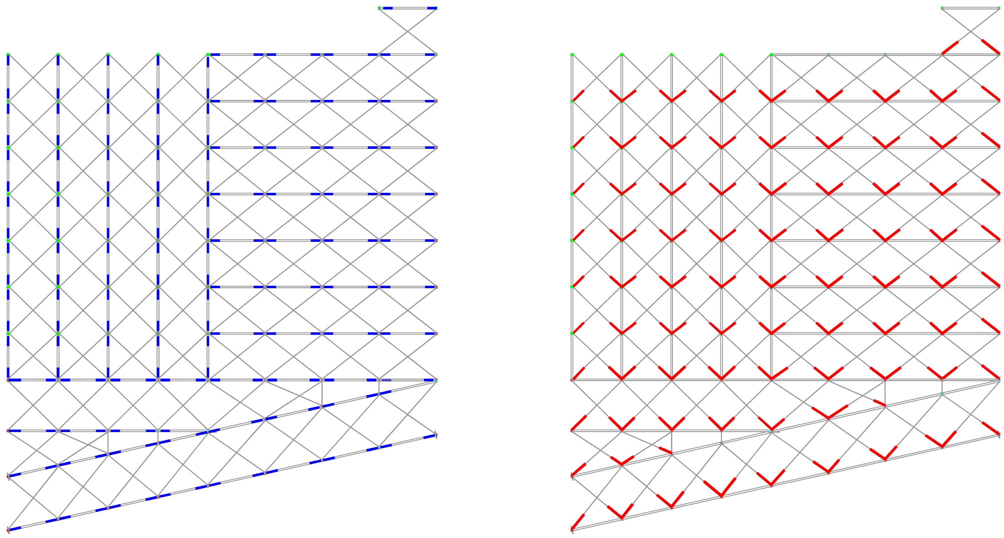


Fig. 9 – Plan view of the location of damping devices: Viscoelastic devices (left) and frictional devices (right)

Fig. 10 – Retrofit damping devices installed in Building 2 (left), and local modification of MEP for allowing the retrofit solution (right)

3.3.2 Analysis and results

The analyses performed of the retrofitted structure were nonlinear response history analyses, and spectrum compatible seismic records were used in order to represent the code design spectra. Since the design spectrum for isolated buildings from the NCh2745 code document has proved to be more accurate to estimate elastic seismic demands, whilst the design spectrum of NCh2369 code document is the governing code for the design of this building, two different seismic demands were considered for the design of the retrofit system:

i. The NCh2369 design spectrum for the analysis and verification of the structural members of the building (foundations, columns, beams, braces, lateral displacement of the roof, etc.).

ii. The NCh2745 [6] design spectrum multiplied by a factor of 1.2 for the analysis and design of the damping devices and their connections to the structure (10% probability of being exceeded in 100 years, MCE level).

Both spectra are shown in Fig. 11 and Fig. 12, where the NCh2745 spectra in the figure has not yet been amplified to MCE level.

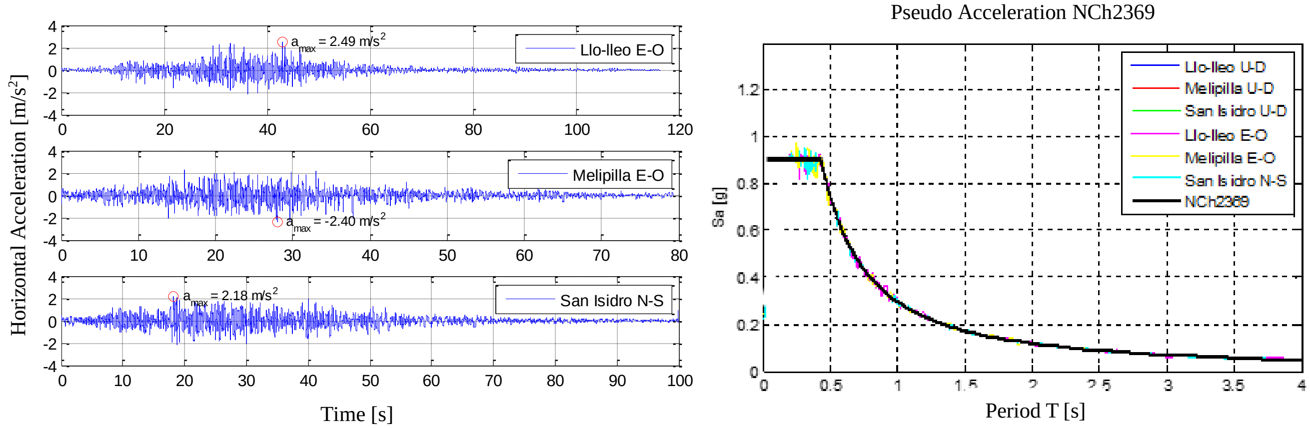


Fig. 11 – Seismic records (left) calibrated to NCh2369 design spectrum (right)

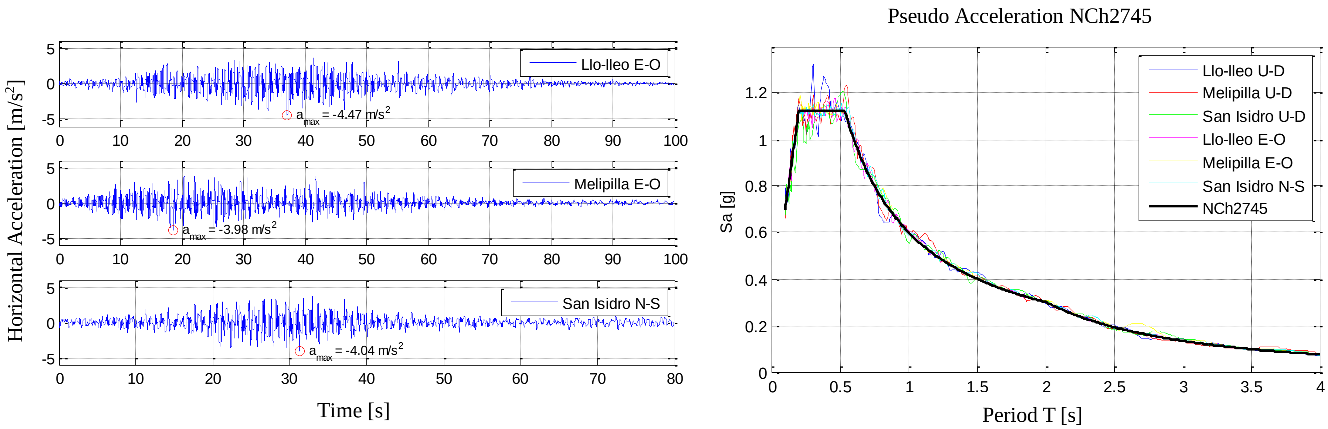


Fig. 12 – Seismic records (left) calibrated to NCh2745 design spectrum (right)

The global response parameters of Building 2 are shown in Table 2 below. The table outlines parameters for the envelope conditions where all the vertical pins at the beam-column "dry" connections are operative (undamaged), and where all of them are damaged (further explained in Fig. 5).

Table 2 – Global structural responses of Building 2

	Design Earthquake (NCh2369 R=1)	
	All Vertical Pins Undamaged	**All Vertical Pins Damaged**
Max X Roof displacement (cm)	12.8	16.3
Max Y Roof displacement (cm)	9.4	11.7
Base Shear X direction (Ton)	647.6 (9.3% W_s)	731.9 (10.5% W_s)
Base Shear Y direction (Ton)	641.9 (9.2% W_s)	593.9 (8.6% W_s)
Braces max. compression (Ton)	12.0	12.0
Braces max. tension (Ton)	12.0	12.0

W_s: Seismic weight of the structure

According to NCh2369, the maximum roof displacement for elastic response cannot exceed 0.015*h*, (21 cm for Building 2), and the design base shear must be greater than 12% of the seismic weight of the structure (W_s). When special damping devices are used, and a nonlinear time-history analysis is performed, this minimum base shear limit does not need to be considered (NCh2369 section 5.9.1.7). Thus, the global response of the building complies with the applicable code limitations.

The results obtained for each type of damping device are shown in Table 3, once again these are divided for the two possible conditions of the beam-column joint (operative and damaged), and also for the two seismic demands considered for the design. According to these results, the friction dampers were designed for a displacement of 100mm, and the viscoelastic devices where specified for a minimum of 40mm.

Table 3 – Demands on energy dissipation devices at Building 2

		Design Earthquake (NCh2369 R=1)		**Maximum Earthquake (NCh2745 x 1.2)**	
		Undamaged Vertical Pin	**Damaged Vertical Pin**	**Undamaged Vertical Pin**	**Damaged Vertical Pin**
Viscoelastic devices	Max u1 displacement (mm)	8	11	27	28
	Max F1 force (Ton)	22.4	22.7	36.3	36.5
	Max u2 displacement (mm)	-	6.	-	15
	Max F2 force (Ton)	-	14.3	-	25.5
Frictional devices	Max u1 displacement (mm)	39	8	86	78
	Max F1 force (Ton)	12.0	12.0	12.0	12.0

3.3.3 Column reinforcing

Some columns of Building 2 exceed their axial-flexural interaction capacity along their weak axis, according to the initial structural review done of the building. Even when the damping devices produce some response reduction and redistribution of column flexural moments, some of the columns of Building 2 require reinforcing at the base (5 of the total 56 columns). The redistribution of flexural moments mentioned, is produced because of the action of the viscoelastic dampers at the beam-column joints, where a counteracting flexural moment at the column top is produced. Thus, at least for the direction in which beams and viscoelastic dampers are present (weak axis of columns), the columns have a slightly smaller cantilever arm and behavior closer to that of a rigid frame. Thus, in Fig. 13 and Fig. 14, it can be seen how the displacements and base flexural moments in the columns are reduced in the X-direction of the building (direction of columns weak axis and most of the beams), while in the Y-direction both responses remain unaffected because of the damping devices.

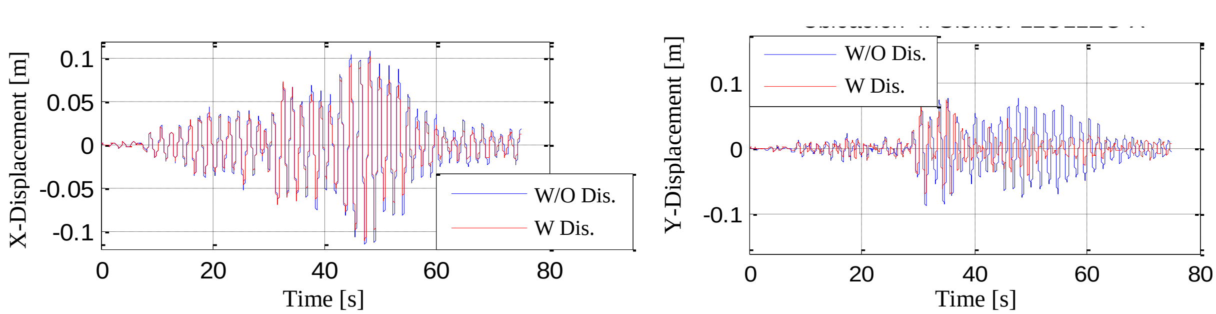


Fig. 13 – Example of column displacements with and without dissipation (At position CP1, see Fig. 8)

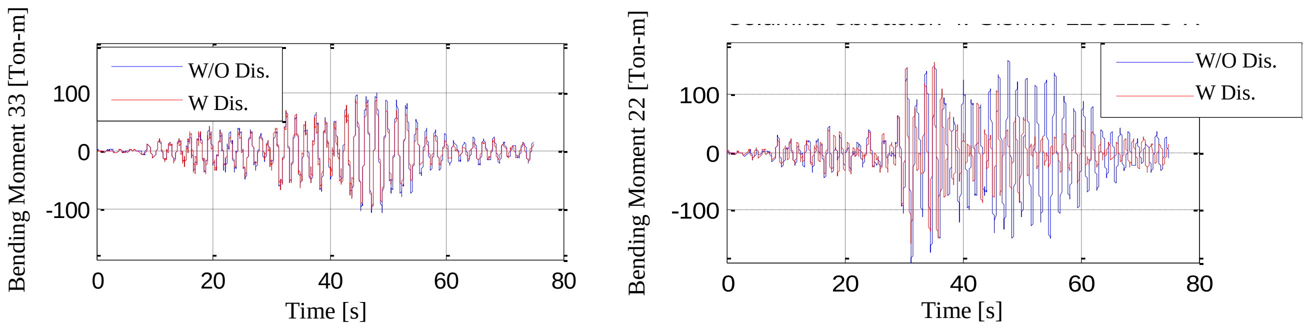


Fig. 14 – Example of column base flexural moment with and without dissipation (At position CP1, see Fig. 8)

The reinforcing designed for the columns consists of filling the H shaped cross section with reinforced concrete, transforming it into a rectangular section (Fig. 15). This is very effective for increasing the resistance along the weak axis of the columns (Fig. 16).

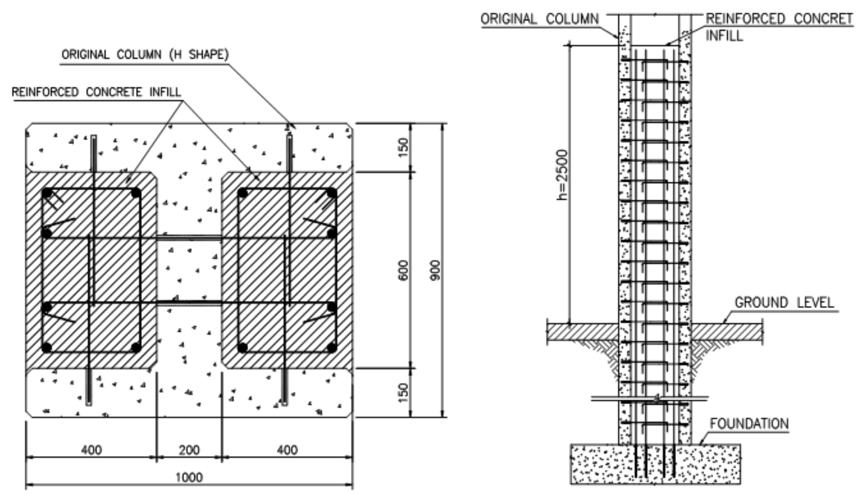


Fig. 15 – Schematic axial-flexural reinforcing of columns: Cross section (left) and elevation (right)

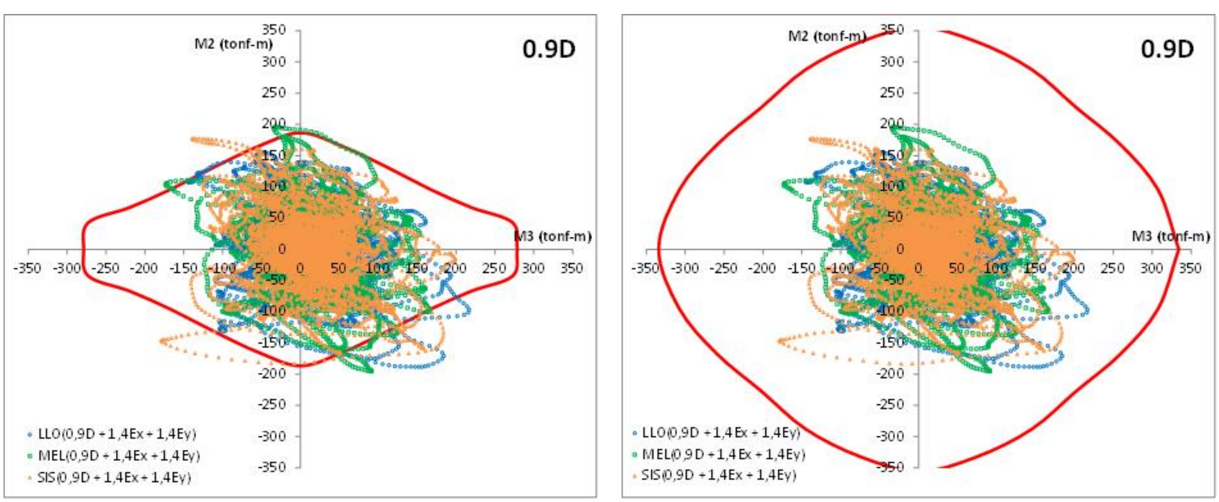


Fig. 16 – Biaxial flexural interaction curve for the original (left) and reinforced (right) column cross section (Axial compression for 0.9D=112tonf)

4. Conclusions

The proposed retrofit system accomplishes all design requirements, it enhances the seismic performance of the building, and is currently being implemented without interruption of the operations of the facility. The original damage susceptible connections of the precast structure were replaced by ductile elements (damping devices) that can guarantee the continuity of the roof diaphragm during a severe earthquake (capacity design criteria for roof braces).

Additionally, the dissipation devices were able to generate a reduction of flexural moment about the weak axis of the columns, reducing the number of columns to be reinforced from more than 50 to just 5. This reduction is achieved through damping and also by partially transforming the cantilever column structural system into a frame system, redistributing part of the flexural moment from the base of the columns to the top where the viscoelastic dampers act as connectors of the beam-column joints.

The retrofit solution presented herein works extremely well in theoretical and practical terms and may be used in other precast structures, thus providing an economical means of retrofitting existing low ductility precast industrial warehouses with dry connections. Failure of some of these precast structures during the 2010 Chile earthquake was rather catastrophic, and the procedure presented herein is an alternative to provide energy dissipation capacity to these brittle structures.

5. References

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