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Resilient structures in the seismic retrofitting of RC frames: A case study

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Abstract. It is very important to allocate valuable resources efficiently when reconstructing buildings after earthquake damage. This paper proposes the use of a simple seismic retrofitting system to make buildings more resilient than the stiffer systems such as the shear walls implemented in Chile after the earthquake in 2010. The proposal is based on the use of steel chevron-type braces in RC buildings as a dual system to improve the seismic performance of multistory buildings. A case study was carried out to compare the proposal with the shear wall solution for the typical seismic Chilean RC building from the structural and economic perspectives. The results show that it is more resilient than other stiffer seismic solutions, such as shear walls, reduces the demand, minimizes seismic damage, gives reliable earthquake protection and facilitates future upgrades and repairs while achieving the level of immediate occupancy without the costs of the shear walls system.

Keywords: Chevron-type steel bracing; Shear walls; Earthquake reconstruction; Resilient seismic structure; Seismic performance; Performance based design

1. Introduction

On February 27th 2010 one of the biggest earthquakes in history with a magnitude of 8.8 Mw struck the regions of Maule and Biobio in Chile. Its effects were felt for more than 600 km along the coastline and the quake was followed by a tsunami. The consequences of these two events caused more than 500 fatalities, destroyed over 350,000 houses hospitals and schools and devastated infrastructures, causing $30 billion worth of damage, as shown in Gobierno (2010) and Government (2010).

Many buildings in Chile are now fitted with seismic systems based on shear walls, which have often been fitted to reconstructed apartment blocks after seismic movements. This type of structure is quite stiff and attracts high seismic forces, but is able to withstand an earthquake, as reported in Jünemann et al. (2015), where the authors made a statistical analysis of observed damage in RC buildings with shear walls after the 2010 earthquake, concluding that medium-rise buildings experienced most damage and were prone to brittle failure in first stories of buildings because of high axial load ratio.

Most of the houses damaged in the earthquake were made of adobe, comparable to other cases (Sayin *et al.* (2014)), although many other types of construction were also affected, such as confined and reinforced masonry and timber and concrete buildings. Initial estimations after the earthquake suggested that from 50 to 100 concrete buildings suffered severe damage, although very few of them actually collapsed or needed to be demolished (Comerio (2013), Pontificia (2012)). However, human errors take an important role in the reliability assessment of these buildings after earthquakes, as stated by Tuken *et al*. (2017).

After the catastrophe, the Chilean government drew up a Reconstruction Plan, which was intended to help the victims on low incomes to return to normal as quickly as possible and help communities to recover their physical identities, keeping their ties to the land and promoting responsive innovation. A total of 220,000 home-owners received government aid. More than 70% of these homes were re-built in the same area as the original buildings. Families could choose their homes from predesigned houses certified by the Minister, build their own house under supervision or buy an existing one. Most of the families whose homes were destroyed (48,000 units) chose pre-certified houses, and more than 6,000 families were allocated to new social condominiums under a densification plan, Government (2011). Some of these blocks had four or five floors, similar to the model described in the case study, to save space and keep neighborhoods together.

Since one of the main aims of this Reconstruction Plan was to promote responsive innovation, it included seismic technologies. However, innovations were only implemented in one case, as commented in Government (2011), in the ‘Villa Nueva Paniahue’ project in Santa Cruz in the O’Higgins Region. In this project, 192 families were given a housing solution in the form of eight four-floor buildings, with a mixed system of base isolation and frictional sliding (e.g. Braga and Laterza (2004)) that was designed to counteract the poor soil conditions. The new buildings were made of reinforced concrete and the seismic isolation was installed by the SIRVE S.A. Company, which had

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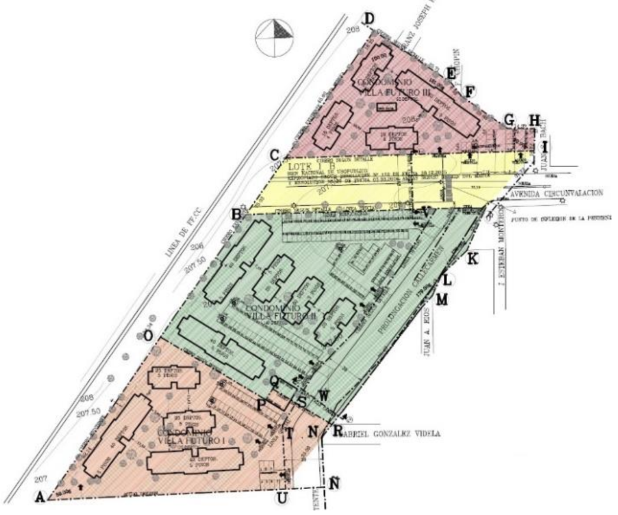


Fig. 1 Plan view of 'Villa Futuro I, II, III' project

implemented this technology in hospitals and public buildings, and installed the energy dissipators in the Titanium La Portada skyscraper in Santiago de Chile in case of earthquakes (Nunez *et al*. (2013)).

Nevertheless, the construction of these new social condominiums was (and is) oriented towards RC buildings with up to five floors and shear walls, in which elevators are not mandatory (see Ministerio (2016)). One example of this typology is the set of 13 buildings constructed in three phases, known as 'Villa Futuro', in the North sector of Curicó city, in the Maule Region, which was the subject of our case study.

In this context, this paper compares RC buildings fitted with shear walls, often used to reallocate people in need after earthquakes, with a proposal using steel bracing as seismic solution. This proposal has certain similarities with the reconstruction work carried out by the Marquis of Pombal in the city of Lisbon, devastated after the 1755 earthquake, which led to the structural solution known as "Gaiola Pombalina". This solution, as explained in Mascarenhas (2005) and numerically assessed by Cardoso et al. (2005), is based on a timber cage ("gaiola") with infilled masonry walls with a dissipative and stiffening purpose in case of earthquakes. The reader can also find examples of the use of steel bracing in seismic design in the following references, among others: Pincheira and Jirsa (1995), where the authors discuss the influence of the use of steel bracing on the seismic capacity of buildings; Turker and Bayraktar (2011) who compare the effect of four different steel bracing configurations in terms of dynamic response; and Eskandari and Vafaei (2015) where authors work on eccentrically braced buildings comparing the effects of near-fault and far-fault earthquakes.

It will be shown that the proposed system reduces the demand and minimizes seismic damage to avoid major repairs thus saving both money and time, achieving the level of immediate occupancy without the costs of the shear walls system. Other techniques, such as friction and hysteretic energy dissipators (Papadopoulos et al. (2013), Dominguez and Lopez-Almansa (2017)), or the use of replaceable steel coupling beams (Xiaodong et al. (2018)) were also considered, but the number of storeys and the desire to propose a light, resilient and economic structure, led the authors to retrofit a basic reinforced concrete frame building with concentric steel X-braces. The structural and financial aspects of both the traditional and proposed solutions for seismic loads are compared and a set of recommendations is given to improve post-earthquake reconstruction and make buildings more resilient. The steel braces were seen to clearly improve stiffness and capacity, and promise to be an interesting option for future post-earthquake scenarios instead of shear walls.

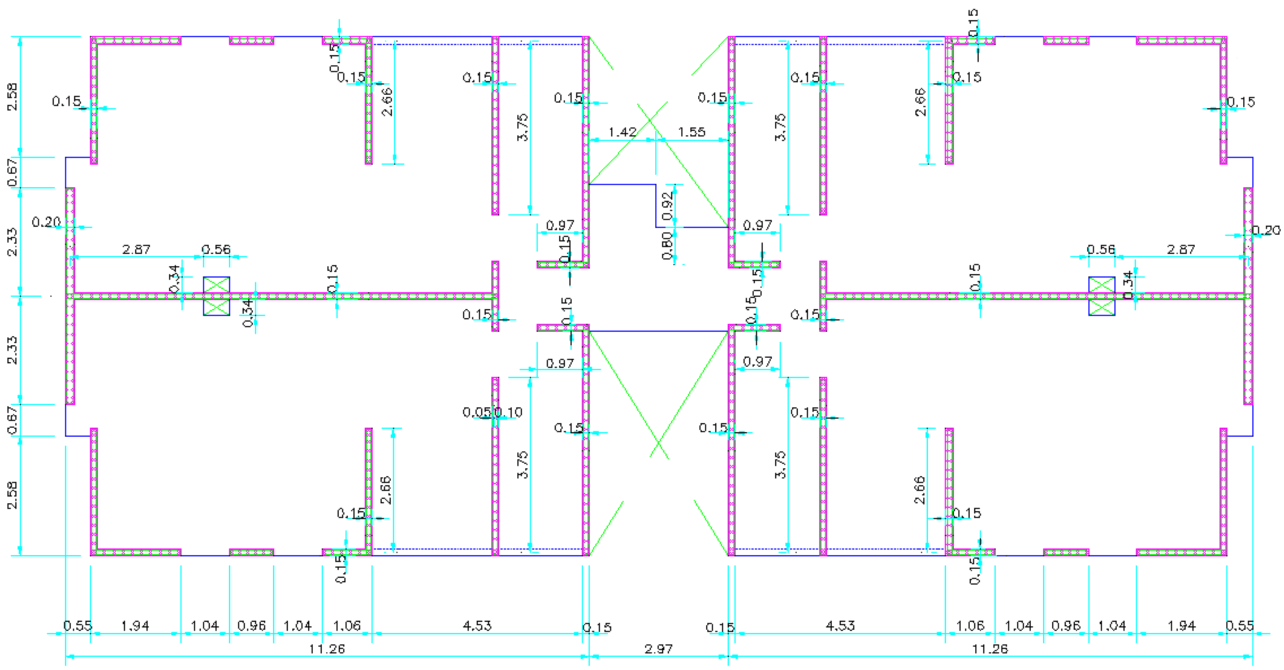


Fig. 2 Building structure of third floor: plan view (units in m)

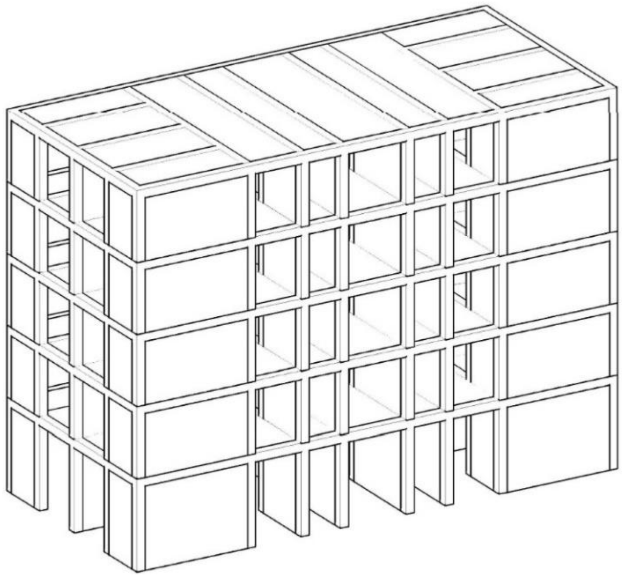


Fig. 3 Model of the 3D structure based on concrete shear walls

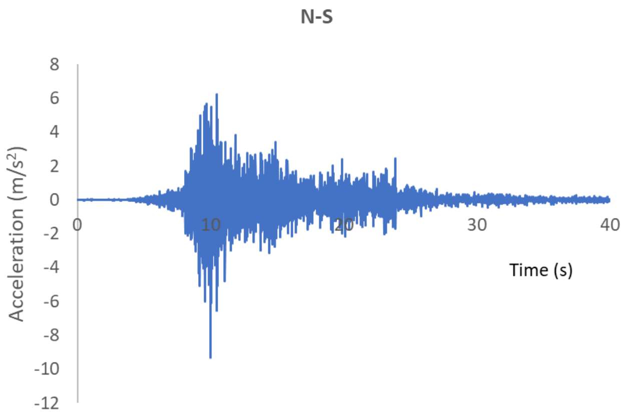


Fig. 4 Accelerations in N-S direction registered in Maule earthquake 2010

2. Case study

The Villa Futuro project described here is materialized and composed of 352 4- and 5-floor apartments of around 60 square meters each in 13 blocks. The project has parking lots, green areas, social spaces and different amenities. Construction was in three phases: Villa Futuro I (120 apartments), II (140 apartments) and II (92 apartments), as can be seen in Fig. 1.

All the buildings have the same design: foundation composed of shallow wall footings, concrete shear walls to account for horizontal forces and concrete floor slabs. Fig. 2 shows a plan view of the building structure of the third floor. The reinforced concrete walls are 0.15 and 0.20 m thick, with a compressive strength of 25 MPa. The Chilean Standards for the Seismic Design of RC Structures, NCh430 (2008) and NCh433 (2009), were used for design and verification. The slabs are 0.14 m thick and designed for a live load of 20 MPa and 25 MPa in common spaces such as corridors, staircases etc. Fig. 3 depicts the 3D model used to perform the structural calculations, with base fixed at ground level. All these data and structural scheme are in accordance with the typical seismic Chilean RC building reported by Lagos et al. (2012).

The earthquake load was introduced in the form of the actual accelerogram that was registered in the N-S direction (see Fig. 4), and time history analysis were conducted on all the structures under study.

3. Proposed model

Very stiff buildings can attract high seismic demands and incur serious damage. Currently, it is considered important in seismic design to use low-damage structural systems that can be easily replaced after earthquake damage. This provides resilient buildings and sustainable constructions that can be repaired at a low cost. The authors thus opted to use chevron-type braces in new buildings under the aforementioned conditions forming dual systems which are economic, able to concentrate plastic deformation and dissipate energy during seismic movements, and are easy to inspect and replace. As reported by Eskandari et al. (2017), there is little scientific literature considering structures composed of RC frames and steel bracings as a dual system. This latter reference, together with Kim et al. (2009), are examples of articles where authors conduct non-linear static and transient dynamic analyses investigating the behavior of steel braced RC dual systems under earthquakes, modelling in line with what has been done in the present work.

Two proposals are considered to be compared with the current building typology, based on the premise that seismic initiatives should be economically feasible: a) a basic RC frame system based on beams and columns with floor slabs; and b) a basic RC frame system plus steel braces in which the frame is based on beams and columns stiffened with steel chevron-type braces in selected bays and floor slabs. At first glance, the basic system is not a good choice for seismic-prone areas, since the structure is excessively deformable; however, it is used as a standard scientific practice to demonstrate the performance of the proposals compared to a reference, together with the aim to obtain information about how far in terms of strength and stiffness the two seismic systems are from the basic system.

Fig. 5 shows a plan view in which the beams and columns are used for the frame system, while Fig.6 shows the 3D models used to calculate these systems.

The basic system (Fig. 6a) is formed by 0.40x0.40 m columns on the ground floor and 0.30x0.30 m columns on the upper floors, 0.40Hx0.35V m cross section beams and 0.14 m thick slabs. System b (Fig. 6b) is based on the same structural frame with additional steel braces formed by 200.200.16 mm A250ESP steel beams (NCh203 (2006)). Two chevron-braced systems were installed on each of the outer walls. The concrete characteristics and design specifications were the same as those used for the case study. Correspondingly, models were base fixed at ground level.

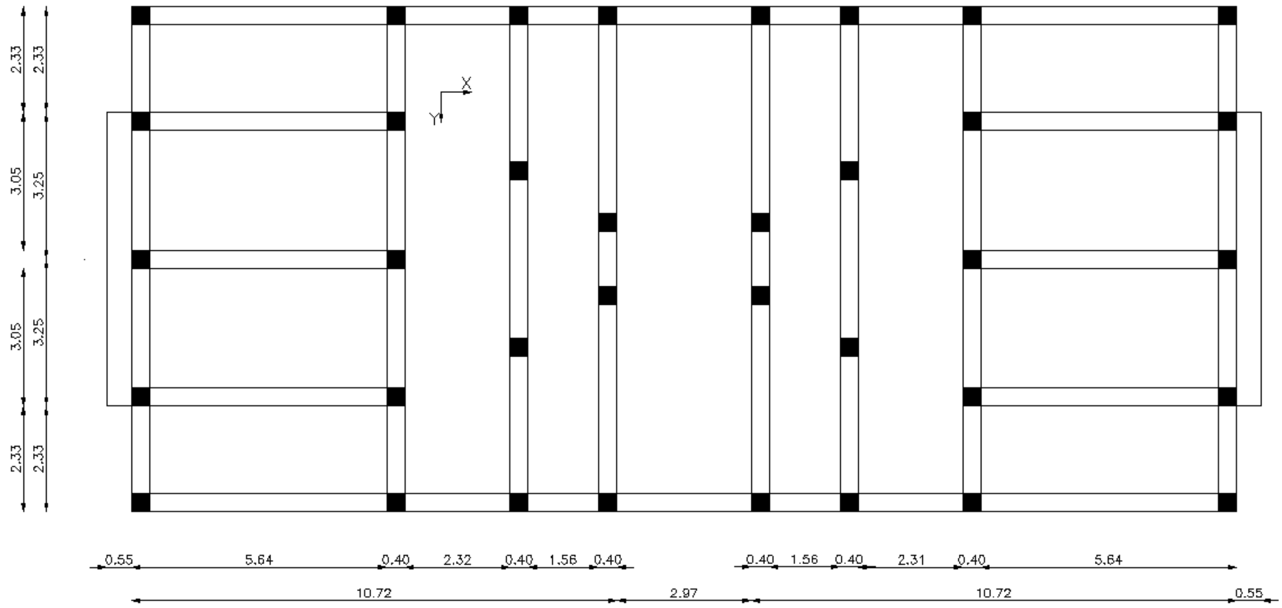
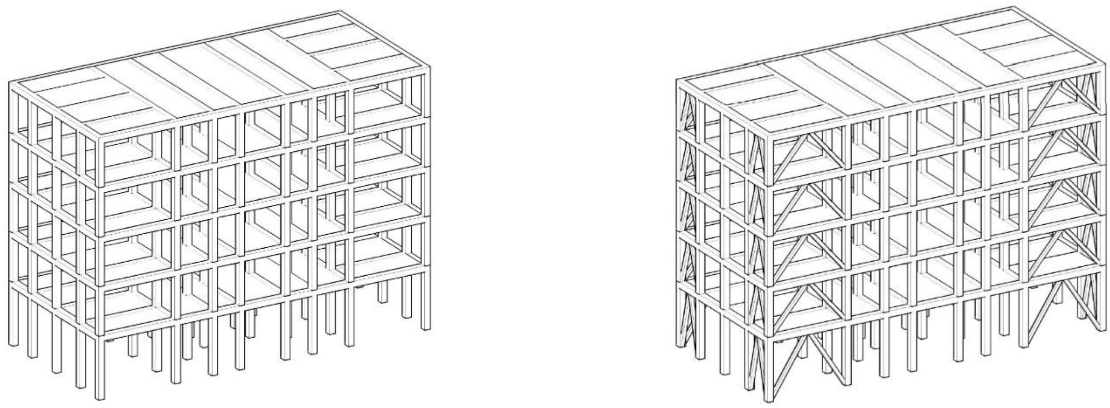


Fig. 5 Plan of beams and columns in the building ground floor of systems a and b (units in m)



(a) Basic frame system (b) Basic frame system plus steel braces

Fig. 6 Graphical representation of the numerical models

4. Results

SeismoSoft (2013) software was used for the calculations given in Figs. 3 and 6 in the two main X-Y directions. This software can consider material and geometrical non-linearities in both static and dynamic analyses. For additional information, the reader is referred to the work by D'Aniello et al. (2013), where modelling issues for braced structures are discussed using SeismoSoft. In the present paper, steel material is modelled according to the classic Menegotto-Pinto steel model (Menegotto and Pinto (1973)) with slight modifications to account for higher numerical stability and accuracy for transient seismic loads. Yield strength was established in 500 MPa for rebars and 270 MPa for structural steel, while modulus of elasticity was 2E5 MPa. Concrete material follows the constitutive relationship proposed by Mander et al (1988), considering confinement effects due to lateral transverse reinforcement and cycle rules stated by Martinez-Rueda and Elnashai (1997). This five-parameter model was defined providing mean compressive strength (25000 kPa), mean tensile strength (2800 kPa), modulus of elasticity (23500 MPa), strain at peak stress (0.0022 m/m) and specific weight (24 kN/m^3).

Using this numerical model, the following results were obtained:

4.1 Preliminary results

The total weight of the structure gave a value of 11,130·kN for the case studied, 6,770 kN for the system-a and 7,015·kN for system-b. The chevron-type braces increased the basic structure weight by less than 4%. Substituting chevron-type braces for the concrete shear walls reduced the weight by 37%. Table 1 gives modal periods and cumulative mass of each system for the first 10 modes of the lateral displacements X,Y and torsional rotation Rz.

As can be observed, the periods of the frame structure are between 4 and 6 times greater than the case under study, and between 2 and 3 times greater than the braced system. In terms of stiffness, the proposal would lie half-way

Table 1 Modal periods and cumulative mass

	The Case				Frame system (a-system)				Frame system with steel braces (b-system)			
Mode	Period (s)	[Ux]	[Uy]	[Rz]	Period (s)	[Ux]	[Uy]	[Rz]	Period (s)	[Ux]	[Uy]	[Rz]
1	0.183	0.0%	8.2%	69.0%	0.715	83.5%	0.0%	0.7%	0.432	0.0%	81.8%	0.0%
2	0.167	0.0%	73.7%	77.9%	0.605	83.5%	82.3%	4.2%	0.287	79.6%	81.8%	3.7%
3	0.133	72.6%	73.7%	77.9%	0.585	84.2%	85.8%	86.1%	0.259	83.2%	81.8%	84.3%
4	0.048	72.6%	73.8%	94.1%	0.273	95.8%	85.8%	86.2%	0.154	83.2%	97.4%	84.3%
5	0.038	72.6%	92.5%	94.1%	0.235	95.8%	96.8%	86.5%	0.103	96.7%	97.4%	84.9%
6	0.036	74.8%	92.5%	94.1%	0.230	95.9%	97.1%	97.2%	0.094	97.3%	97.4%	97.9%
7	0.035	75.2%	92.5%	94.1%	0.143	98.4%	97.1%	97.2%	0.079	97.3%	99.5%	97.9%
8	0.033	75.8%	92.6%	94.1%	0.125	98.4%	98.7%	97.3%	0.062	97.4%	99.5%	97.9%
9	0.033	76.7%	92.6%	94.1%	0.123	98.4%	98.8%	98.8%	0.059	97.4%	99.5%	97.9%
10	0.032	88.5%	92.6%	94.1%	0.102	99.6%	98.8%	98.9%	0.057	97.4%	99.9%	97.9%

between the current system and the basic one. With a mass around 4% larger than the basic system, stiffness is considerably higher, as expected.

4.2 Results from a structural point of view

4.2.1 Response accelerations

With the aim to better understand the behavior of the structural systems under study and get a general idea, response accelerograms at the top floor were obtained in the N-S direction for the three cases. Fig. 7 shows these responses, where it is interesting to observe the estimated maximum accelerations in the upper floor and time in which the accelerations exceed a certain threshold. For easier comparison, the positive envelopes of these accelerations are plotted in Fig. 8. The case under study is a very stiff structural system which replicates and amplifies the input accelerogram. A great peak acceleration, amplified from the input one, is withstood by the system at the early stages. Following, the response decays as the earthquake does, although amplifications are found in the whole time range. This amplified maximum acceleration is usually very destructive in this type of structures, inducing strong damage in structural and non-elements, costly to repair, so should be avoided if possible. Designing for this maximum value could overestimate the structure for the rest of the seismic motion. The basic frame system is quite more flexible and the response accelerations at the top are filtered by the structure, obtaining much lower values than the previous case. This could lead to think about a better performance; however, it will be shown that this model leads to excessive displacements that do not accomplish Code provisions. The third system, the braced frame, does not amplify the response at the peak, so reducing the expected damage when compared to the case study. The maximum level of accelerations is approximately maintained for longer period without clear decay, and amplifications are observed when compared to input acceleration. However, if displacements are admissible, designing for the maximum acceleration would not overestimate in excess for the rest of the earthquake.

These comments are specific for the cases under study, but can shed some light on the way each structural scheme responses under Maule earthquake, especially when observing real damages in shear concrete buildings where strong damage is observed in shear walls so, interesting conclusions can be outlined.

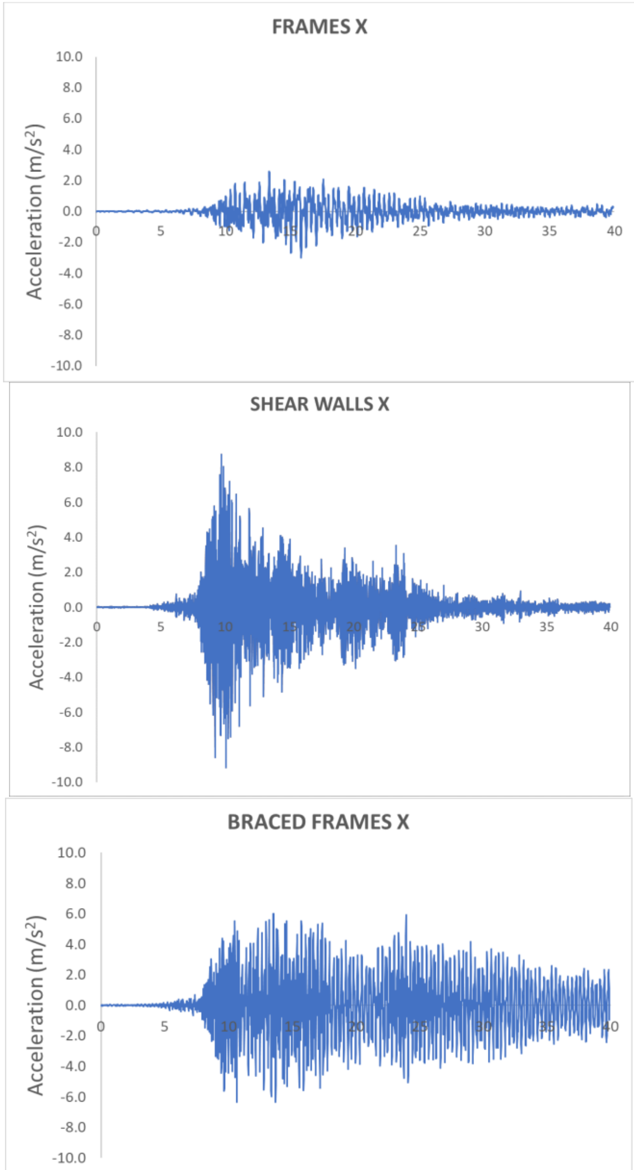


Fig. 7 Accelerations at top floor in N-S (X) direction for Maule earthquake 2010

4.2.2 Capacity

Since the aim of the study was to obtain an economic anti-seismic alternative for the reconstruction of earthquake-damaged buildings that conformed to the

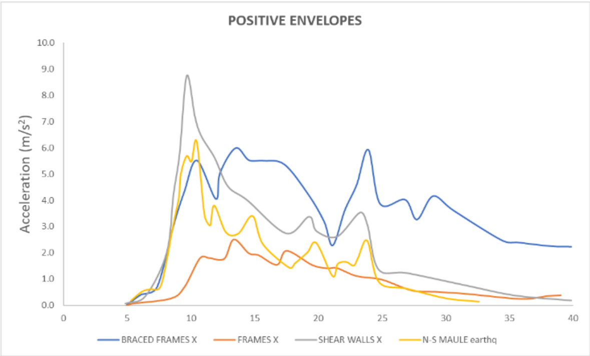


Fig. 8 Positive acceleration envelopes vs time for input earthquake and studied structures

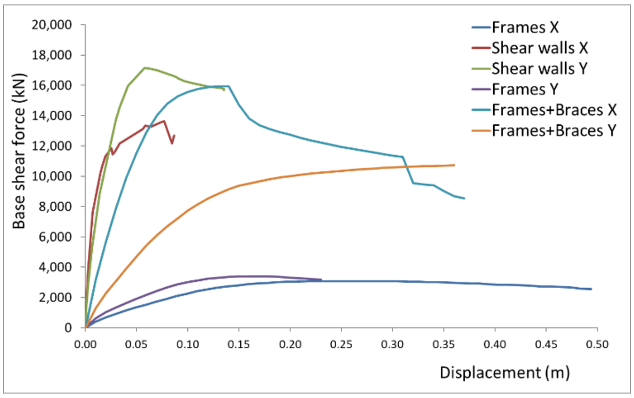


Fig. 9 Base shear force versus horizontal displacement in pushover analyses

Table 2 Performance points

PERFORMANCE POINTS (m)						
	Frames X	Frames Y	Shear Walls X	Shear Walls Y	Frames+Bracses X	Frames+Braces Y
IO	0.264	0.101	0.012	0.013	0.028	0.064
LS	0.339	0.130	0.016	0.016	0.036	0.082
CP	0.587	0.225	0.031	0.028	0.063	0.143

Chilean Standard, a simplified numerical-based fragility analysis was used to assess the structure's performance levels in seismic movements, using the capacity curve technique to account for the seismic vulnerability of the building (ATC (1996)), evaluated by means of a pushover nonlinear analysis. The damage thresholds were evaluated from the idealized bilinear capacity spectrum according to Lagomarsino and Penna (2003), using the yielding displacement (dy) and the ultimate displacement (du). These four damage thresholds are:

Sd,1 = 0.7dy,

Sd,2 = dy,

Sd,3 = dy + 0.25(du-dy),

Sd,4 = du,

representing 'Slight', 'Moderate', 'Extensive', and 'Complete' damage states.

The seismic action was obtained in terms of a response spectrum according to NCh433 (2009), adapted following the ASCE (2007) Code, by which the building's performance levels are defined using three states: the Immediate Occupancy limit state, Life Safety limit state and Collapse Prevention limit state. Table 2 gives all these values for the different structural systems, while Fig. 9 shows total base shear force versus horizontal displacement in the X-Y directions at the top of the building for each of the cases under study. As can be observed, only the Braced Frames curve in direction Y is lower than its comparable curve in this direction for shear walls, since the chevron-type braces have a different angle of inclination. It would be structurally easy to raise this curve by changing the brace ends or the number of braces.

It seems clear from Fig. 9 that chevron-braced system is able to provide similar levels of structural stiffness and strength to the current design.

Table 3 Displacements (in meters) and ductility analysis according to FEMA356 (2000)

Frames			
	Yield	Target	Ductility
Xaxis	0.12	0.50	4.35
Shear Walls			
	Yield	Target	Ductility
Xaxis	0.02	0.03	1.70
Braced frames			
	Yield	Target	Ductility
Xaxis	0.05	0.06	1.20

The different structural performance levels can be seen in Fig. 10, together with the damage thresholds. This figure compares the models and shows that the new proposal has better responses than the traditional system.

As expected, the basic framed structure does not provide good results in either direction. The best response (direction Y) causes moderate damage, even for the Immediate Occupancy state, so that this structural model does not meet the seismic demands of the Chilean Standard. More interesting conclusions are obtained when comparing the current shear wall building and chevron-braced frames. In direction Y both systems show almost identical behavior, with performance levels in the same damage states for both cases, even in the 'more flexible' direction Y for the braced frame (see Fig. 9). However, the braced frames show even better behavior in direction X, since the post-earthquake 'Collapse Prevention' state would be in the zone between 'Extensive' and 'Complete' damage for shear walls, while it would be between 'Slight' and 'Moderate' damage for the chevron-braced frame proposal.

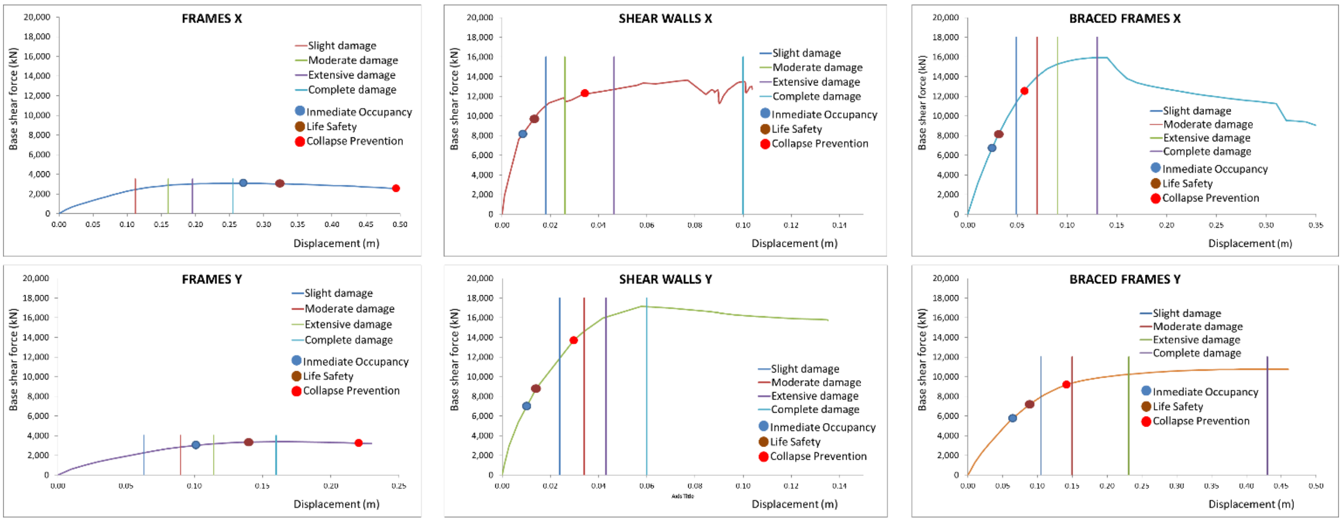


Fig. 10 Damage thresholds and performance levels of all models in each direction

Changes in stiffness observed in Fig. 10 for shear walls are due to progression of damage in shear walls, quite pronounced because the large stiffness of every shear walls greatly affects global behavior.

4.2.3 Ductility

A ductility analysis for the three cases has been developed for better comparison. FEMA356 (2000) considerations have been used to obtain ductility values. In this sense, target displacements are considered as the Collapse Prevention levels and yield displacements are obtained based on an idealized force-displacement curve built balancing areas above and below the curve. Table 3 shows target and yield displacements in meters, while ductility factors are calculated based on their ratio. It is shown that both structures (case under study and braced frames) have similar levels of ductility. However, target and yield displacements are greater in the braced structures, allowing for more energy dissipation under seismic motion. Displacement values obtained for the basic frame structure do not accomplish seismic provisions claimed by Codes as previously commented, so this structural system is not considered as an option for reconstruction.

4.3 Financial considerations

In the ‘Villa Futuro’ project, the Chilean government provided shelter and new homes for 352 families. After the earthquake, a huge budget was required to house around 6,000 families in new social condominiums, so that small savings in one typological project (maintaining the performance) or improvements in design can lead to big savings in future events. The aim is to offer not only structural alternatives at the lowest possible price for both the construction phases and future reconstruction scenarios, but to expand the range of choices when rebuilding homes after a disaster.

Bearing this in mind, an estimation of the increased costs, or savings, if possible, was made for the new proposal as compared to the current situation. This was done in terms of total amount of materials consumed and an approximation to the economic cost considering current Chilean prices for the project units (concrete and structural steel), including usage rate of materials, labor, auxiliary equipment and placing the materials. These prices are 135,000 CLP (160 USD) per cubic meter for concrete and 800 CLP (0,9 USD) per kg for structural steel.

Table 4 gives the main differences between the ‘building structure’ for the current system and the new proposal in terms of materials. Both cases are identical except in the resistant structure, so only the concrete and structural steel elements are considered. It can be seen that the new proposal would save around 180 m3 (450E3 kg) of concrete, which means a saving of around 40%. However, almost 28E3 kg of structural steel would be needed. At the prices of these two materials set in this study, both structures are financially comparable, since savings of 180 m3 of concrete are 2,4E6 CLP (29,000 USD) while increases of 28E3 kg of structural steel are 2,2E6 CLP (26,000 USD). Furthermore, the proposed structure shows better performance, so further savings are expected if optimized.

These savings, together with the facts: a) that the proposed structure is lighter and attract a smaller seismic demand, requiring fewer resources, and b) that chevron-braced frames are easier to assess than shear wall buildings and simpler to retrofit after an earthquake, are points that should be considered when deciding on the type of structure to be used in massive reconstruction programs after seismic events.

5. Conclusions

Post-seismic scenarios are always a great challenge for both the authorities and citizens, especially in the aftermath of a severe earthquake, as happened in Chile after the Maule-Biobio earthquake in 2010. However, after these catastrophic events lessons can be learnt and improvements can be made.

Table 4 Amount of materials required for each resistant system

FRAMED STRUCTURE (BEAMS AND COLUMNS)

CONCRETE	Number	X (m)	Y (m)	H (m)	Total
Columns	60	0.4	0.4	3.0	28.8 m3
	40	0.3	0.3	3.0	10.8 m3
					39.6 m3
			Density		
			2500 kg/m3		99000 kg

SHEAR WALLS

CONCRETE	Number	X (m)	L (m)	e (m)	Total
Walls	5	0.2	97.2	3.0	218.7 m3
					218.7 m3
		Density			
		2500 kg/m3			546863 kg

CHEVRON-TYPE BRACES FRAMED STRUCTURE

CONCRETE	Number	X (m)	Y (m)	H (m)	Total
Columns	60	0.4	0.4	3.0	28.8 m3
	40	0.3	0.3	3.0	10.8 m3
					39.6 m3
			Density		
			2500 kg/m3		99000 kg

STEEL BRACES (200,200,16)

b (m)	e (m)	Floors	Number	L (m)	Area (m2)	Total
0.200	0.016	5	8.0	4.2	0.01178	2.0 m3
0.168		5	8.0	3.3	0.01178	1.5 m3
						3.5 m3
				Density		
				7850 kg/m3		27843 kg

This paper describes an alternative type of apartment block for reconstruction in these cases for the consideration of those responsible for reconstruction programs when massive re-housing projects are undertaken requiring large amounts of public money.

After Maule earthquake, Chilean reconstruction programs focus on shear concrete walls as the main typological apartment block structure; however, the proposal is based on the innovative philosophy of providing more efficient earthquake-resilient buildings not only by increasing their resistance but also by concentrating the damage in easily replaceable elements, leading to lighter and more economical structures while maintaining performance. The proposal was compared with the traditional shear wall solution and a basic beam-column solution from the structural and economic aspects. While the basic solution was seen to be far removed from the required standards, the proposal was able to maintain performances and reduce damage levels at a lower cost by using a simple and versatile method to significantly modify stiffness.

The proposal also makes it easier to inspect and repair damaged buildings after earthquakes and can put them back into service at a reasonable cost, thus pointing in the direction of more sustainable buildings in future seismic events.

Acknowledgments

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