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ELASTIC OVERSTRENGTH OF REINFORCED CONCRETE SHEAR WALL BUILDINGS IN CHILE

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Abstract

The overstrength is a key property of engineered building structures, and its beneficial effects have been recognized in several studies. When ground accelerations are higher than those specified in seismic design codes, the overstrength helps prevent the collapse of buildings but not necessarily the occurrence of some structural damage. However, during the 2010 Chile earthquake (M_W 8.8), 90% of buildings in the affected regions showed no damage at all, despite recorded ground accelerations higher than those required by the Chilean seismic code. Many studies about damaged structures have been performed, and some of them have pointed out the need for research on undamaged buildings as well. Besides, no study has focused on the overstrength of Chilean buildings considering their particular design conditions and seismic hazard. In order to fill these gaps, this study defines the elastic overstrength and evaluates such quantity in actual reinforced concrete shear wall buildings that suffered no damage in the 2010 earthquake even though they were subjected to seismic demands greater than those specified by the seismic design code. Three buildings of 5, 17 and 26 are evaluated based on nominal, linearly elastic analysis. Possible influence of issues such as material strength and modeling assumptions were evaluated. Results show that the 5-story building is able to stand undamaged ground accelerations that are up to 3 times larger than those required by the Chilean seismic code, which explains the lack of damage in this building. On the other hand, however, results also indicate that the other, higher buildings should have suffered important damage, i.e., the elastic overstrength was not found to be large enough to explain the lack of damage in these buildings.

Keywords: reinforced concrete, shear wall, overstrength, 2010 Chile earthquake, seismic behavior

1. Introduction

Seismic design of structures is a critical issue in preventing casualties and economic losses due to the occurrence of earthquakes. Seismic events are of intrinsically random nature, and their occurrence and characteristics such as magnitude cannot be predicted with certainty. Further, due to the variability of material properties and to the inherent simplifications in practical analysis and design procedures the assessment of the actual seismic capacity of structures is affected by a non-negligible amount of uncertainty. This uncertainty is typically compensated for by means of conservative code requirements. As a result, the actual (unknown) seismic capacity of structures is usually larger than that explicitly intended by code requirements, for a wide margin in some cases.

In Chile, for instance, as in many other countries, structures are design to withstand a reduced level of seismic forces, i.e., the elastic seismic demand is divided by a response modification factor R, which is a function of the inelastic deformation capacity of the structure. Therefore, if the demand of an actual earthquake is equal or larger than the (reduced) design capacity, the occurrence of some damage is then expected. However, even though the elastic response spectrum of many ground motions recorded during the 2010 Chile Earthquake is larger than the (reduced) design spectrum stipulated in the Chilean seismic code (Fig. 1), only 2% of the multistory building inventory suffered significant damage [1]. The earthquake caused strong ground motions in the largest metropolitan areas in Chile (Santiago, Concepcion and Valparaiso-Viña) and affected a large pool of engineered buildings designed for a reduced level of seismic forces, but nevertheless the amount of damage was quite modest.

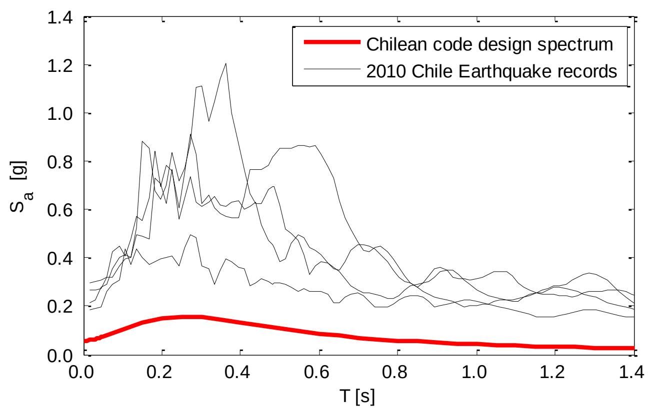


Fig. 1 – Pseudo-acceleration response spectra of four different records of the 2010 Chile Earthquake, all of them showing spectral accelerations greater than those of the Chilean design spectrum (reduced by R) for a RC wall building of T = 0.47 s

It is clear then that the actual seismic capacity of typical Chilean buildings is larger than that intended by the code. This observation is valid not only for Chilean buildings but also for buildings in other countries where modern seismic codes are correctly implemented. Actually, this reserve of capacity beyond that intended by code requirements is a well-known concept in structural engineering, called overstrength (Ω). It is a recognized property of structures designed in accordance with modern codes and is sometimes considered, either explicitly or implicitly, into the design process. The importance of the overstrength has been highlighted by several researches, up to the point that it has been stated that during strong ground motions the survival of code-designed structures is possible only because of the implicitly assumed overstrength [2].

Some typical sources of overstrength identified in the literature [3]–[6] include: higher materials strength, element sizes larger than required, conservative load combinations, contribution of nonstructural elements, ductility requirements, strength reduction factors, and moment redistribution, among others. The precise contribution of these factors to the overstrength, however, is somewhat unclear. As a consequence, the estimation of the global overstrength is affected by a high level of uncertainty. For instance, in buildings designed in accordance with Canadian normative, Mitchell and Paultre [3] found that overstrength values vary from 2.1 to 4.6 for concrete frames and from 2.8 to 5.3 for concrete frame-walls, and Humar and Rahgozar [6] found values ranging from 2.0 to 2.4 for steel frames. In buildings designed in accordance with European normative, Elnashai and Mwafy [7] found overstrength values varying from 2.1 to 2.5 for concrete frames and from 2.2 to 2.8 for concrete frame-walls. Jain and Navin [4] found large values, up to 15.0, for concrete frames designed in accordance with Indian seismic code. As it can be seen, the variability of Ω is large.

The typical overstrength formulation is usually related to the capacity curve obtained by a pushover analysis (Fig. 2a) where the overstrength Ω is the ratio of the maximum strength V_{max} to the design capacity V_d, Eq. (1). It can be observed in Fig. 2a that the maximum strength V_{max} is reached after a significant incursion into the inelastic range.

$$\Omega = \frac{V_{max}}{V_d} \tag{1}$$

Code-oriented structures are intended to guarantee life safety in the occurrence of a severe earthquake by preventing collapse, although at the expense of an accepted level of structural damage. This is presented graphically in Fig. 2a, where the demand imposed by a severe earthquake (that exceeds the design level) intersects the capacity curve in the inelastic range. Therefore, in the Chilean context, the traditional definition of overstrength could explain why buildings did not collapse but does not explain the lack of structural damage.

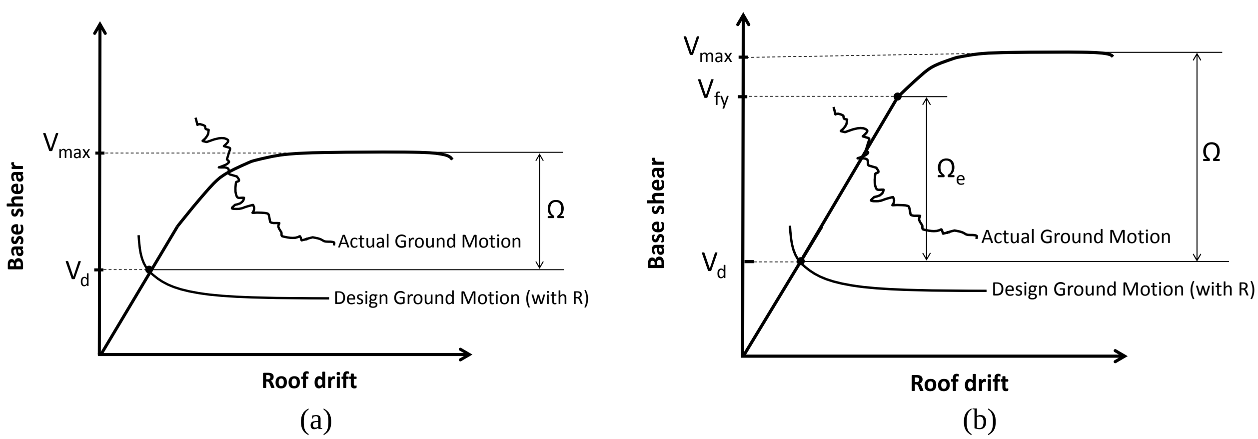


Fig. 2 – (a) Traditional overstrength formulation (b) Proposed elastic overstrength formulation

An alternative formulation of overstrength is presented in Fig. 2b. Here the structure remains elastic during a severe earthquake, and for this reason the seismic demand intersects the capacity curve in the linear range. This scenario is more consistent with what was observed in the 2010 Chile earthquake, where most buildings did not suffer damage. The elastic portion of the overstrength can then be defined as elastic overstrength Ω_e (Fig. 2b). It is given by the ratio of the first yielding strength V_{fy} to the capacity required by design V_d, Eq. (2). The first yielding strength represents the level of lateral force at which the first structural element yields or fails, therefore indicates the end of the elastic range of the structure.

$$\Omega_e = \frac{V_{fy}}{V_d} \tag{2}$$

Chilean residential buildings rely strongly on Reinforce Concrete (RC) walls. These buildings have a typical plan configuration clearly identified: the so-called “fish-bone” pattern. It consists of transverse short walls at the edges of each apartment unit, longer longitudinal walls along the corridors, and wall cores around the elevators and stairs. Most of the structural damage due to the 2010 Earthquake occurred in structures with these characteristics. Consequently, extensive studies have been conducted to analyze the damage in these structures [8]–[10], and high axial loads, high wall slenderness, and deficient ductility detailing have been identified as the main sources of damage. However, considering that most of the buildings suffered no damage, researchers have been exhorted to perform analysis of undamaged structures. Jünemann et al. [11] conducted an statistical analysis of the global properties of buildings subjected to the 2010 Chile Earthquake, both damaged (36 buildings) and undamaged (7 buildings). The study found that while selected undamaged buildings are somewhat more flexible that damaged buildings, "most of the analyzed properties of undamaged buildings are very similar to those of damaged buildings". Jünemann et al. [11] is the first study that includes some analysis of undamaged buildings, and apparently the only one to date. This is the main motivation of this paper, which aims at better understanding the good performance of Chilean RC shear wall buildings under severe earthquakes. The specific objectives are to evaluate the elastic overstrength, to verify if such overstrength is consistent with the observed structural behavior, and to examine possible inclusion of the elastic overstrength in the design process.

The current paper evaluates the previously introduced elastic overstrength of RC shear wall buildings undamaged by the 2010 Chile Earthquake. The sample set of buildings is made up of three actual buildings of 5, 17 and 26 stories located in Santiago, Chile. Several sources of overstrength related with either demand or capacity are analyzed. Chilean code response spectra as well as actual ground motion records are used to evaluate the seismic demand, and capacity assessments are refined beyond the standard procedures of structural engineering offices.

The evaluations presented in this study are based on nominal, linearly elastic analysis. Given the fact that the buildings considered in this study were not damaged by the 2010 Chile earthquake, it can in principle be assumed that their satisfactory behavior (i.e., the quantification of their elastic overstrength) can be reasonably supported by linear analysis techniques.

2. Building Inventory

Three buildings were selected in order to represent typical low, medium and high rise Chilean buildings. The first building has 5 stories, the second one has 17 stories, and the third building has 26 stories. All of the buildings are apartment complex located in Santiago city as shown with yellow marks in Fig. 3.

The 5-story building (Fig. 4a) was built in 2009. All the vertical elements in the building are RC walls of 150 mm thickness with f'_c = 20 MPa for concrete and f_y = 420 MPa for steel (A630-420H steel according to Chilean standards). It was completely detailed with minimum reinforcement for shear and flexure. The 17-story building (Fig. 4b) was built in 2006. The building has 2 columns at the underground levels but the rest of the vertical elements are walls of 150 mm and 170 mm thickness. Some retention walls of 200 mm thickness are located at the perimeter of the underground levels. The 26-story building (Fig. 4c) was built in 2005. The building is mostly made up of 17 mm thickness walls, and 20, 22 and 25 mm thickness walls are located at the underground levels either for shear or for soil retention. These and other building properties are summarized in Table 1.

The beams were not detailed as coupling beams in any of the buildings. Further, the models used in the actual design process include moment releases at the ends of the beams. However, none of the beams in any of the buildings was damaged during the 2010 earthquake. Table 2 shows the average wall density along both orthogonal directions of the buildings, i.e., the cross sectional area of all the walls divided by the floor plan area. According to Calderon [12] the average ratio in Chilean wall buildings is 0.028. Therefore, regarding this parameter, the analyzed buildings can be considered representative of the typical Chilean practice. None of the walls in the three buildings has confinement detailing in the boundaries, which has been pointed as source of damage in other buildings [9]-[10].

The buildings were designed in accordance with the Chilean seismic code NCh433-Of.96 [13]. As the three buildings are located in Santiago all of them were designed considering the same seismic zone, i.e. seismic zone 2. Although zone 2 is the intermediate seismic zone in the Chilean code, the corresponding level of seismic demand is nevertheless typical of regions of high levels of seismic activity.

Table 1 shows that the buildings were designed considering what is denoted soil type III (5-story building) and II (the other buildngs) in NCh433-Of.96 [13], which defines four soil types. Thus, the buildings are located in soils of intermediate quality.

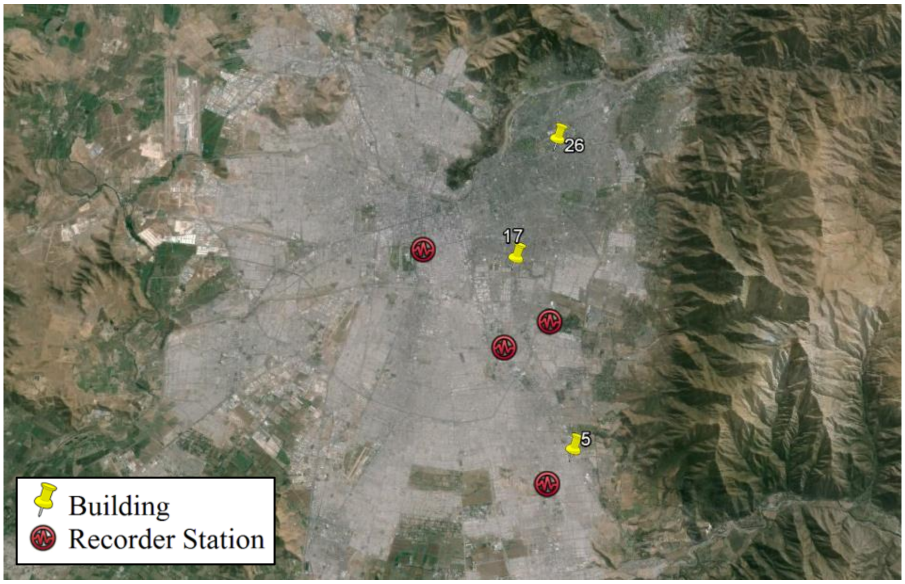


Fig. 3 Location of the 3 buildings and the 4 recorder stations in the city of Santiago

(a) (b) (c)

Fig. 4 – Picture and typical plan view for the three analyzed buildings

Table 1. Geometric properties, seismic classification and material properties for the three buildings

		Geometric Properties			Seismic Classification			Material Properties	
Building	**Year**	**Underground Levels**	**Height [m]**	**Typical floor dimension [m]**	**Seismic Zone**	**Soil type**	**Importance Factor**	**Concrete compressive strength [MPa]**	**Steel yielding strength [MPa]**
5-story	2009	0	12.11	41.2 x 8.7	2	III	1	20	420
17-story	2006	2	42.16	49.4 x 13.2	2	II	1	20	420
26-story	2005	4	72.24	23.3 x 18.4	2	II	1	25	420

Table 2. Direction dependent properties for the three buildings

Building	Direction	Period [s]	Modification Factor R*	Base Shear Factor	Effective Modification Factor R	Wall Density
5-story	Long	0.11	2.31	0.74	3.12	0.029
	Short	0.25	3.52	0.80	4.40	0.026
17-story	Long	0.43	7.48	1.00	7.48	0.026
	Short	1.11	9.49	2.44	3.88	0.024
26-story	Long	1.49	10.01	2.72	3.68	0.025
	Short	1.73	10.24	3.23	3.17	0.024

The design response spectrum in NCh433-Of.96 [13] is given by an elastic response spectrum (intended to represent the seismic demand) divided by a period-dependent response modification factor R*. The resulting spectrum is the so-called "reduced" spectrum. The corresponding reduced base shear demand is then calculated. The value of the reduced base shear demand must lay within specified limits, otherwise R* must be changed until the design base shear limitations are met. The final value of R* due to such adjustments is then the effective response modification factor R. As shown in Table 2, values of R can be very different from those of R*. The "Base Shear Factor" in Table 2 is the ratio of R* to R.

3. Nominal Evaluation of Elastic Overstrength

In order to estimate Ω_e, which is a single parameter for the whole building, it is necessary to evaluate the demand/capacity ratios (D/C) at a local level and considering all the requirements specified in the code. Once all the D/C ratios are obtained, the inverse of the maximum value among all the requirements is taken as the Ω_e. The requirements under evaluation are:

1. Interstory drift ratio (at the center of mass and at the critical corner)
2. Wall flexure-compression capacity (each wall at each level)
3. Wall shear capacity (each wall at each level)

3.1 Interstory drift ratio

The Chilean seismic design code NCh433 [13] section 5.9.2 establishes a requirement for maximum interstory drift ratio at the center of mass (Δ_{CM}) of 0.002. Then, the D/C ratio was estimated as:

$$\frac{D}{C}=\frac{|\Delta_{CM}|}{0.002} \tag{3}$$

Besides, to prevent excessive plan rotations due to torsion, the Chilean code in section 5.9.3 establishes that the maximum interstory drift ratio of the story, i.e., at the critical corner (Δ_{max}), cannot exceed the drift at the center of mass Δ_{CM} by more than 0.001. For this criterion the D/C ratio was estimated as:

$$\frac{D}{C}=\frac{|\Delta_{max}|-|\Delta_{CM}|}{0.001} \tag{4}$$

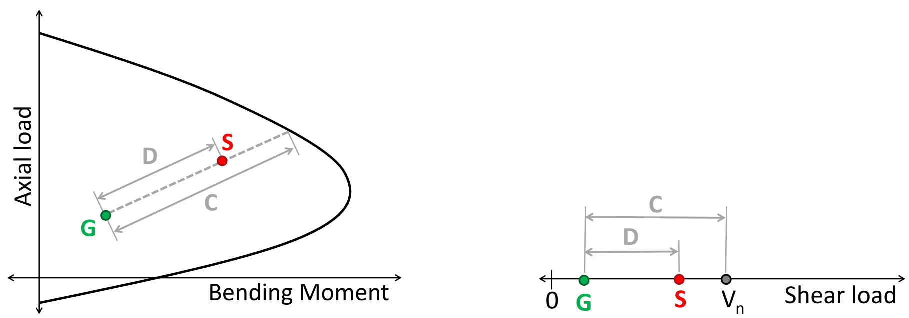
D/C ratios at each story of each building were calculated with Eqs. (3) and (4). The largest value was taken as the D/C ratio for drift, and are shown in Table 4.

3.2 Wall flexure-compression capacity

As for any concrete member subjected to both flexure and compression, the capacity of the walls was evaluated through interaction curves. In the flexure-compression 2D space, the demand due to a given load combination is a "point", and the capacity is represented by a continuous curve. Thus, the determination of scalar values for D and C is not straightforward. For this reason, the authors adopted the approach presented in Fig. 5(a), where point S represents the demand due to a seismic load combination (e.g., 0.9 D + 1.4 E) and G represents just the gravity portion of the same load combination (e.g., 0.9 D). The demand D is then the norm of the line between points S and G. This line is then extended until it intersects the interaction (capacity) curve, and the new norm (i.e., between G and the intersection point) represents the capacity C. It must be noted that the interaction curve is not reduced (i.e. $\phi = 1.0$) in order to better evaluate the actual elastic capacity. Nominal procedures, i.e., those specified in ACI 318-08 [14] were adopted to evaluate the capacity. For a given wall at a given story, D/C ratios were calculated for each load combination, and the largest value was then taken as representative value. Such values were calculated for each wall at each story of each building, and the critical (i.e., the largest) values are shown in Table 3.

3.3 Wall Shear Capacity

The estimation of the D/C ratio for shear is more direct because both the capacity and the demand are scalar values. The approach adopted is consistent with that adopted for flexure-compression. Demand D is the difference between the demand due to a load combination and the demand due to the corresponding gravity load. Capacity C is the difference between the nominal capacity and the demand due to gravity load only. This approach is depicted in Fig. 5b. Nominal shear capacity V_n was estimated according to ACI 318-08 [14] section 21.9.4. As before, D/C ratios were calculated for each wall at each story of each building. Critical (i.e., the largest) values are summarized in Table 3.



(a) (b)

Fig. 5. Graphical approach for D/C in (a) flexure-compression (b) shear

3.4 Values for D/C ratio and Ω_e

Once the D/C ratios for the three requirements were obtained, the largest value was then used to calculate the building elastic overstrength using Eq. (5). In other words, the elastic overstrength was calculated without performing pushover analysis, which would have been needed should Eq. (2) have been used instead of Eq. (5). It must be noted, however, that Eq. (2) and Eq. (5) are conceptually equivalent. Values of elastic overstrength are shown in Table 3.

$$\Omega_e = \frac{1}{\max(D/C)} \tag{5}$$

Except in the case of the 5-story building along the short direction, the largest D/C ratio is in all cases the one corresponding to flexure-compression in the walls. This suggests that the expected failure mechanism of the buildings is flexure-compression. It is also interesting to notice that drift D/C ratios have relatively low values, which is surprising because the drift limitations of the Chilean seismic code have long been considered too strict. This observation indicates that Chilean RC wall buildings are very stiff structures.

4. Variation in Elastic Overstrength due to analysis and design considerations

As it can be seen in Table 3, with the exception of the 5-story building, the values of elastic overstrength are just slightly larger than 1.0. With these low values of Ω_e it is difficult to justify building responses consistent with that shown in Fig.2b, which assumes a large amount of elastic overstrength. It is clear then that an evaluation of the elastic overstrength based on nominal properties turned out to be somewhat unrealistic. In what follows several issues are evaluated in more detail in order to get more insight into their possible influence on Ω_e. These issues are: (1) accidental torsion provisions; (2) expected concrete compressive strength; (3) expected steel yielding strength; (4) cracked section of walls; (5) modulus of elasticity estimation; (6) expected load combination; and (7) rigid diaphragm constraint.

Table 3. Demand Capacity Ratios and Elastic Overstrength

Building	Direction	Demand / Capacity Ratios			Elastic Overstrength
		Drift ratio	Wall flexure-compression	Wall shear	Ω_e
5-story	Long	0.04	0.48	0.31	2.07
	Short	0.19	0.24	0.33	3.07
17-story	Long	0.08	0.89	0.59	1.11
	Short	0.25	0.94	0.57	1.07
26-story	Long	0.29	0.94	0.58	1.07
	Short	0.40	0.95	0.60	1.05

4.1 Accidental torsion provisions

Like other typical modern codes, the Chilean seismic code [13] includes accidental torsion provisions. These requirements are intended to limit the torsional response of the building in the occurrence of a non-predicted eccentricity due to random variations in mass and/or stiffness properties. In the Chilean code the accidental torsion effects can be modeled by either plan (torsional) moments or eccentric floor masses. In any case, accidental torsion provisions increase the seismic demands on the walls.

Although accidental torsion requirements provide a necessary degree of safety, actual accidental torsion of significant level is unlikely, especially in buildings with regular plan floors and where the lateral stiffness is well distributed among several walls. The possible influence of accidental torsion is then assessed by reevaluating the D/C ratios without taking into account the accidental torsion when assessing the seismic demand. The resulting values of the elastic overstrength are shown in Table 4. The intention of this evaluation is not to question whether or not accidental torsion should be included in the analysis, but rather to evaluate its contribution to the elastic overstrength.

4.2 Expected concrete compressive strength

Results shown in Section 3 were obtained using the same specified concrete strength f_c’ (Table 1) that was used in the actual design process of the buildings. Concrete mixtures, however, are made in such a way that the average strength is often greater than the specified strength for a secure margin. In recognition of this reality, performance based design methodologies allow the use of a concrete compressive strength higher than f_c’. The Los Angeles Tall Buildings Structural Design Council [15], for instance, recommends that $f_c'_{expected} = 1.3\ f_c'$ be used instead of f_c’. This approach was adopted in this study to reevaluate the elastic overstrength. Results are shown in Table 4.

4.3 Expected steel yielding strength

Similar to concrete compressive strength, the yielding strength and ultimate strength of steel samples tend to be larger than the nominal values. For instance, between 2006 and 2008 Chilean steel producers reported mean test values of yielding stress that are 20% to 30% larger than the minimum required. On the other hand, the Los Angeles Tall Buildings Structural Design Council [15] recommends that $f_{y\ expected} = 1.17\ f_y$ be used instead of f_y, and once again this approach was adopted in this study to reevaluate the elastic overstrength. Results are shown in Table 4.

4.4 Cracked section of walls

It is expected that small cracks develops in RC walls due to thermal variations, service loads or small ground motions. These cracks change the effective flexural stiffness of the walls, which in turn change the stiffness of the building and in turn the seismic demands as well. Cracking effects are not accounted for in the Chilean seismic code but they are in seismic codes of other countries. For instance, ACI 318-08 [14] suggests an inertia reduction factor of 0.7, i.e. $I_{g\ expected} = 0.7\ I_g$. This approach was adopted in this study to reevaluate the elastic overstrength. Results are shown in Table 4.

4.5 Modulus of elasticity of concrete

It is currently mandatory in Chile to use the equations given by ACI 318-08 [14] to estimate the modulus of elasticity of concrete, i.e. $E_c = 4700\sqrt{f_c'}$ [MPa]. However, a previous Chilean code had a different equation, namely $E_c = 5900\sqrt{R28}$ [MPa], being R28 not the cylinder but the cubic concrete strength at 28 days. In particular, the buildings considered in this study were designed using the value of E_c given by the Chilean equation. Naturally, a different value of the modulus of elasticity of concrete changes the stiffness of the buildings and consequently the seismic demands. Possible influence on the elastic overstrength was assessed by reevaluating the D/C ratios using the value of E_c given by ACI 318-08. Results are shown in Table 4.

4.6 Expected gravity load

Results presented in Section 3 were obtained using the seismic load combinations given by the Chilean code [13]:

$$1.4D + 1.4L + 1.4E \tag{6}$$

$$0.9D + 1.4E \tag{7}$$

As these combinations are intended to represent extreme load conditions, the authors reevaluated the elastic overstrength using a load combination that better represents the expected gravity loads during normal operation of buildings. The selected combination is given by Eq. (8), and the corresponding elastic overstrength values are reported in Table 4.

$$1.0D + 0.25L + 1.4E \tag{8}$$

4.7 Rigid diaphragm constraint

A common practice in building design is to model floor levels as rigid horizontal diaphragms. In the case of concrete buildings this practice is justified on the basis that concrete floor systems have a large in-plane stiffness and essentially behave as rigid diaphragms. The main motivation of this practice is to reduce the number of degrees of freedom of the model, which in turn reduces the computational efforts. However, as computer capabilities increase, reduction of computational cost is becoming increasingly irrelevant, and it is now possible to explicitly model the floor systems. Although the slabs of the buildings considered in this study are thick (120 mm minimum), the high ratio of long-to-short dimensions and the openings might lead to non-negligible in-plane diaphragm deformations. Interestingly, rigid horizontal diaphragms were included in the computer models used in the actual design of the buildings even though the slabs were explicitly modeled with shell elements (the intention was to correctly assess member forces due to gravity loads). Hence, in order to assess the possible influence of floor modeling, the rigid diaphragm constraints were removed from the structural models and the elastic overstrength was reevaluated. Results are shown in Table 4.

4.8 Reevaluation of the Elastic Overstrength

Table 4 summarizes the percentage variation of the elastic overstrength due to each of the previously analyzed issues. The percentage variation (and the corresponding overstrength value) due to all of the previous issues combined is shown as well. It can be seen that the influence of each issue on the elastic overstrength varies strongly from one building to another, and even in a given building from one direction to another. Further, a given issue might lead to an increase of Ω_e in one building but to a reduction in another, which illustrates the complexities associated with realistic seismic evaluations.

Removal of accidental torsion demands increased the value of Ω_e in all cases, in percentages ranging for 1.5% to 16.6%. Higher concrete strength has a surprisingly negative effect in some cases. It was found that such decrease of Ω_e is due to the fact that when f_c' increases so does the modulus of elasticity, which leads to an increase in demand that is greater than the increase in capacity, thus reducing Ω_e. On the other hand, higher steel strength always has a positive effect on Ω_e because higher values of f_y leads to greater capacities without affecting the demands. Reduction of both the wall stiffness and the value of E_c have a similar effect in the sense that both lead to a reduction of the building stiffness without affecting the capacity. In turn, reduction of the building stiffness (i.e., higher fundamental period) leads to smaller demands in the case of the 17- and 26-story buildings and to greater demands in the case of the 5-story building. Such changes are logical due to the relationship between the fundamental periods of the buildings and the shape of the design response spectrum.

Table 4. Percentage variations of elastic overstrength

	5-story		**17-story**		**26-story**	
Criterion	**Long Dir.**	**Short Dir.**	**Long Dir.**	**Short Dir.**	**Long Dir.**	**Short Dir.**
No accidental torsion	1.5%	5.2%	16.6%	11.9%	5.8%	1.9%
Expected f_c'	-5.5%	8.0%	-4.3%	-7.2%	5.8%	2.3%
Expected f_y	14.8%	8.3%	8.2%	7.0%	6.5%	5.6%
Cracked section of walls	10.3%	-6.8%	9.9%	17.4%	15.3%	14.7%
E_c according ACI 318	10.9%	-2.4%	3.4%	7.4%	2.2%	1.7%
Expected gravity loads	7.5%	5.5%	10.7%	13.6%	10.8%	16.0%
No rigid diaphragm constraint	0.4%	17.8%	15.8%	15.0%	1.4%	-3.7%
All criteria combined	42.5%	34.3%	65.1%	61.7%	44.6%	37.5%
Revised Ω_e	**2.96**	**4.12**	**1.84**	**1.73**	**1.54**	**1.45**

As expected, the use of more realistic gravity loads instead of extreme loads leads to higher values of Ω_e. Nevertheless, it is interesting to point out that even though the loads given by Eq. (8) are higher than the ones given by Eq. (7), the value of Ω_e nevertheless increases because the moment capacity at axial loads due to 1.0 D + 0.25 L + 1.4 E is greater than that at axial loads due to 0.9 D + 1.4 E. Finally, removal of the rigid diaphragm constraints has different effects on different buildings, but it leads to significantly higher values of Ω_e in the 17-story building and in one direction of the 5-story building. These results indicate that the precise assessment of the influence of floor modeling might require more detailed research.

As explained in Section 3.2 the elastic overstrength is related only to the earthquake loads, therefore it can be seen as a measure of the magnitude of the design seismic load necessary to produce the first yielding in the structure. In other words, the design spectrum (divided by R) could be amplified by Ω_e and the buildings should still be able to withstand those demands with no expectation of damage. This exercise was performed by the authors and the results are presented in Fig. 6, which also includes pseudo-acceleration spectra of four records of the 2010 Chile earthquake. (Fig. 3 shows the location of the record stations in Santiago close to the analyzed buildings).

It can be seen in Fig. 6 that at the fundamental period (the discontinuous vertical line) both directions of the analyzed buildings have a (reduced) design spectrum lying below the spectra of the recorded ground motions. When the design spectrum is amplified by Ω_e, the 5-story spectra become greater than the spectra of the records, which explains why this building showed no damage. However, the situation is different for the 17- and 26-story buildings as the amplified spectra remain below the spectra of the records for a wide margin. This analysis indicate that, contrary to the observed behavior, these two buildings should have had important inelastic incursions (but they did not). The large difference between the observed behavior and the analysis indicates the need for further research.

5. Conclusions

This paper presents a methodology for the estimation of the portion of the overstrength contained within the elastic range of RC wall structures, the so-called elastic overstrength (Ω_e). This parameter measures the reserve of strength beyond that intended by code specifications using elastic calculations. Three buildings (5, 17 and 26 stories) that withstood the severe 2010 Chile earthquake with no damage were selected to evaluate their Ω_e, as they can be expected to have a large amount of elastic overstrength. Nominal demand and capacity evaluations performed to compute the Ω_e of the buildings showed that all of them have very low drift ratios when subjected to the design seismic loads. Besides, for all of the buildings the critical design requirement is flexure-compression, which has the highest D/C ratios.

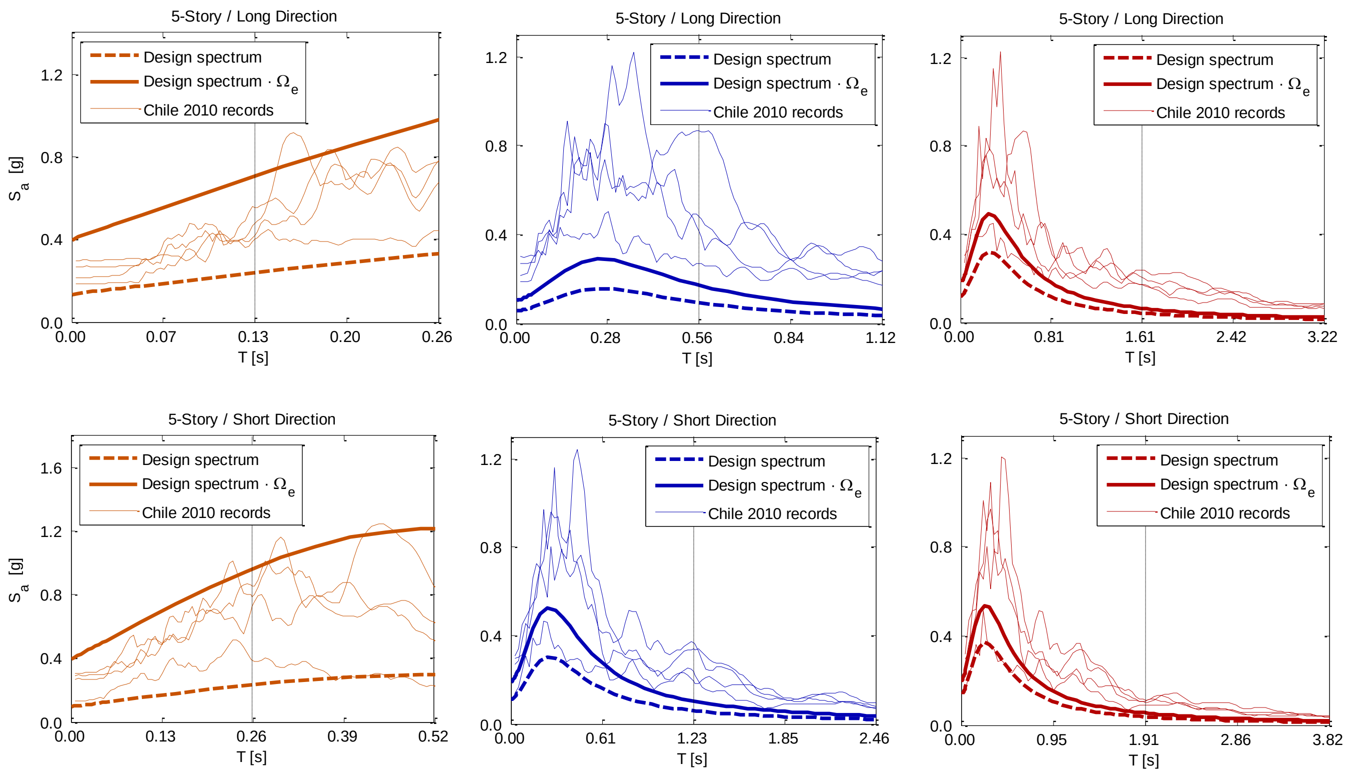


Fig. 6 – Design spectra amplified by Ω_e compared with 2010 Chile earthquake records, all centered at the fundamental period of buildings

Several modifications were done to the nominal design and analysis procedures in order to more realistically assess reflect the expected behavior of the buildings within the elastic range. The individual contribution of several issues such material strength and modeling assumptions to the value of Ω_e was evaluated. While separately each of these issues has a marginal influence on Ω_e (roughly around 10%), all of them together lead to an average 50% increase of the value of Ω_e.

It was found that the value of Ω_e of the low-rise building is large enough to explain why this structure had a good performance during the earthquake. Considering that the building is completely detailed with 15 mm thickness walls with minimum reinforcement, it is inferred that it has more walls than required by design. For buildings like this, social, architectural and functional issues determine the disposition of RC walls rather than structural criteria.

The medium and high-rise buildings, on the other hand, showed relatively low values of Ω_e, definitely not enough to explain why these buildings did not suffer damage during the 2010 earthquake. In view of this, the large values of Ω_e expected for medium- to high-rise Chilean RC wall buildings is empirically confirmed by experience but could not be verified by a nominal, linearly elastic analysis. More research on the seismic response of the buildings considered in this study by linear and nonlinear time history analysis is currently carried out as part of an on-going research project.

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