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Seismic Performance of High-rise Concrete Buildings in Chile

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Abstract

Chile is characterized by the largest seismicity in the world which produces strong earthquakes every 83±9 years in the Central part of Chile, where it is located Santiago, the capital of Chile. The short interval between large earthquakes magnitude 8.5 has conditioned the Chilean seismic design practice to achieve almost operational performance level, despite the fact that the Chilean Code declares a scope of life safe performance level. Several Indexes have been widely used throughout the years in Chile to evaluate the structural characteristics of concrete buildings, with the intent to find a correlation between general structural conception and successful seismic performance. The Indexes presented are related only to global response of buildings under earthquake loads and not to the behavior or design of individual elements. A correlation between displacement demand and seismic structural damage is presented, using the index H_o/T and the concrete compressive strain ε_c. Also the Chilean seismic design codes pre and post 2010 Maule earthquake are reviewed and the practice in seismic design vs Performance Based Design is presented. Performance Based Design procedures are not included in the Chilean seismic design code for buildings, nevertheless the earthquake experience has shown that the response of the Chilean buildings has been close to operational. This can be attributed to the fact that the drift of most engineered buildings designed in accordance with the Chilean practice falls below 0.5%. It is also known by experience that for frequent and even occasional earthquakes, buildings responded elastically and thus with "fully operational" performance. Taking the above into account, it can be said that, although the "basic objective" of the Chilean code is similar to the SEAOC VISION2000 criteria, the actual performance for normal buildings is closer to the "Essential/Hazardous objective".

Keywords: Seismic performance, High-rise, Concrete buildings, Chilean earthquake, Chilean code, Performance based design, Structural index, Stiffness index, Structural response velocity, Elastic performance spectrum

1. Introduction

Chile is located in the southern part of South America between the Andes Mountains and the Pacific Ocean. It has an average of 200 km wide and 4270 km long. Along the shore line is the Pacific trench, where the Nazca Plate subducts under the South America Plate generating frequent interplate type earthquakes some of which have been followed by destructive tsunamis.

On February 27, 2010 a Magnitude M_w 8.8 subduction interplate earthquake impacted the central part of Chile including the cities of Concepción, Viña del Mar and Santiago, affecting an area of 600 km long and 200 km wide, where 40 % of the country population lives. It is the sixth world largest magnitude earthquake recorded by mankind.

In 1985, a Magnitude M_s 7.8 earthquake affected approximately the northern area of the 2010 event. Between that year and 2010, a total of 9,974 buildings over 3 stories high were built in this area according to construction permits issued (Comité Inmobiliario CChC 2010). Of this, 20% had 9 stories or more and an estimate of 3% had over 20 stories up to 52, the tallest at the time of the earthquake.

The statistics show that among engineered buildings, there were 4 collapses (between 4 to 18 stories), and about 40 buildings were severely damaged and had to be demolished (Instituto de la Construcción, 2010). No col-

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lapses of high-rise buildings above 20 stories occurred. This represents less than 1% of the total number of new residential buildings built in this period in the area affected by the earthquake, and can be considered a successful performance from a statistical point of view. The rest suffered nonstructural damage and in some cases minor reparable structural damage.

Chilean Code provisions for the seismic design of buildings are reviewed and the global seismic performance of buildings is evaluated to establish the impact of the code prescriptions in the observed behavior.

2. Chilean Seismicity

Chile is characterized by the largest seismicity in the world which produces strong earthquakes every 83±9 years in the Central part of Chile, where it is located Santiago, the capital of Chile (Compte et al. 1986).

These earthquakes have happened 5 times in the past in 1647, 1730, 1822, 1906 and 1985.

This short interval between large earthquakes magnitude 8.5 has conditioned the Chilean seismic design practice to achieve almost operational performance level, despite the fact that the Chilean Code declares a scope of life safe performance level.

The seismicity of Chile is controlled by the convergence of the Nazca and South American plates at approximately 8 cm/year. Contact and convergence of the Nazca and South American Plates gives origin to interplate type thrust earthquakes, shallow earthquakes in the South American plate and intraplate earthquakes in the Nazca plate.

The rapid convergence of the Nazca plate over South American plate and the youth of the Nazca plate, make Chile prone to happen the largest subduction interplate thrust type earthquakes in the world. In south of Chile, in Valdivia happened in 1960 the largest magnitude earthquake M = 9.5 ever recorded by humankind.

The different types of earthquakes mean that a building can be affected severely by near source events as well as far events. An example of this is Santiago, the capital, and Valparaíso, the most populated cities of the country. They have experimented two extreme earthquakes in 25 years (1985-2010).

Therefore Chilean practice assumes that for a given building at least one large magnitude earthquake will strike it in its life span.

This large seismicity of Chile leads to a deterministic strategy to assess seismic hazard for design of buildings despite the most used probabilistic approach considered in more low or diffuse seismicity regions of the world.

3. Building Practice and Code Provisions in Chile Pre-2010

Chile has several loading and design codes, differentiated by their functionality or structural system. The loading codes are: NCh433 for residential and office buildings (1996 version in English); NCh2369 for industrial facilities and NCh2745 for base isolated buildings.

Chilean seismic code NCh433 had major changes in 1993 and 1996 (NCh433.Of96) where lessons learned after the 1985 Earthquake where incorporated. Seismic analysis procedures established in NCh433.Of96 for Modal Response Spectrum Analysis, are essentially the same as in *Uniform Building Code 1997*, except that forces from the code are allowable stress level and must be amplified for 1.4 for ultimate load level. Design requirements for RC buildings has historically followed ACI 318-95 with few exceptions, being the most notable the exclusion of the requirement for transverse reinforcement in boundary elements in walls. In 2008 with the introduction of the new Concrete Design code NCh430. Of2008, which follows ACI318-05, this exclusion was removed.

A summary of the Code NCh433.Of96 provisions for the analysis of high rise buildings under seismic forces, used in the design of most buildings affected by the 2010 Maule earthquake are:

Type of analysis: Modal spectrum linear elastic analysis, with 5% damping and CQC modal superposition method. Seismic mass taken as DL + 0.25LL.

Accidental torsion analysis: Accidental eccentricity at level k:

$e = \pm 0.10b\,(Z_k/H)$ in each principal direction

Base shear upper and lower limits: $IA_0\ P/6g \leq$ Base shear $\leq 0.35\ SIA_0P/g$.

If Base Shear is out of the range below the lower limit, forces and displacements must be scaled to the exceeded limit. If Base Shear is out of the range above the upper limit, only forces (not displacements) may be scaled to the exceeded limit.

Forces from the code are considered allowable stress level and must be amplified for 1.4 for ultimate load level.

Minimum base shear for normal buildings in seismic Zone 2 is 5% of the weight (P) and in seismic Zone 3 is 6.7%P.

Drift limitations: For stiffness and torsional plan rotation control, including accidental torsion under design spectrum forces, drift for design spectrum forces must not exceed:

- Interstory drift at Center of Mass:
  $\delta/h_{C.M.} \leq 0.002$
- Interstory drifts at any point *i* in plan:
  $(\delta/h_{C.M.} - 0.001) \leq \delta/h_{C.M.} \leq (\delta/h_{C.M.} + 0.001)$

***Earthquake Load combinations*:** Design Spectrum forces are reduced forces that must be amplified for ultimate load combinations required in ACI 318. Load combinations are:

1.4(DL + LL±E)

0.9DL±1.4E

Seismic Zoning:

Seismic Zone	Geographic Area	A_0
Zone 1	Andes Mountains area	0.20 g
Zone 2	Central strip of Chile between the Coastal Mountains and the Andes Mountains	0.30 g
Zone 3	Costal area	0.40 g

Types Soils:

Soil Type	Description	S	T_0	T'	n	p
I	Rock	0.90	0.15	0.20	1.00	2.0
II	Dense gravel, and soil with vs ≥ 400 m/s in upper 10 m	1.00	0.30	0.35	1.33	1.5
III	Unsaturated Gravel and sand with low compaction	1.20	0.75	0.85	1.80	1.0
IV	Saturated cohesive soil with $q_u < 0.050$ Mpa	1.30	1.20	1.35	1.80	1.0

Building Category: Importance factor

Building Category	Description	I
A	Governmental, municipal, public service or public use	1.2
B	Buildings with content of great value or with a great number of people.	1.2
C	Buildings not included in Category A or B	1.0
D	Provisional structures not intended for living	0.6

Design Spectrum: (Fig. 1)

Parameter	Formula	Comments
Design Spectrum	$Sa = \frac{IAo\alpha}{R^*}$	I: importance factor A_0: zone maximum effective acceleration R^*: reduction factor α: period dependent amplification factor
Amplification factor	$\alpha = \frac{1+4.5\left(\frac{Tn}{To}\right)^P}{1+\left(\frac{Tn}{To}\right)^3}$	T_n: vibration period of mode n T_0 , P: soil parameters
Reduction factor	$R^* = 1+\frac{T^*}{0.10To+\left(\frac{T^*}{Ro}\right)}$	R_0: structural system parameter (*i.e.* $R_0 = 11$ for shear wall and braced systems) T^*: period of the mode with largest translational mass in the direction of analysis

4. The Typical Chilean RC Building

High rise buildings in Chile can be classified according to their use in two main categories: residential and office buildings. The main difference is that the later requires large open spaces in plan, while the first must have partitions for occupant privacy. As a consequence the typical structural systems adopted are:

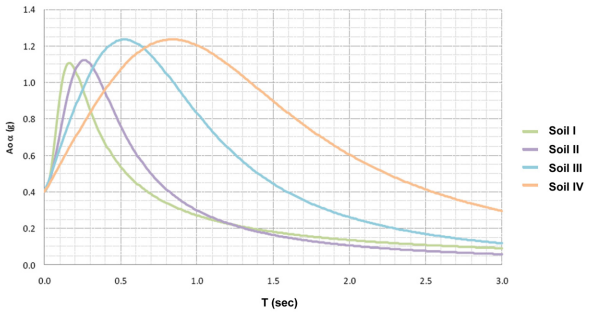


Figure 1. Chilean Code NCh433.Of96, Elastic Design Spectrum (R* = 1) for seismic Zone 3, for Soil Type I, II, III and IV.

Residential Buildings: (Fig. 2)

Floor system: flat concrete reinforced slab. Spans: 5 to 8 m., thickness: 14 to 18 cm supported on shear walls and upturned beams at the perimeter. The vertical and lateral load systems are concrete walls.

Office Buildings: (Fig. 3)

Floor system: Flat post tension slab. Spans 8 to 10 m, thickness: 17 to 20 cm. The vertical and lateral load systems are concrete core walls and a concrete special moment resisting frame at the perimeter.

The main difference between office and residential buildings is that office buildings have shorter wall length and wider thickness than residential buildings. On

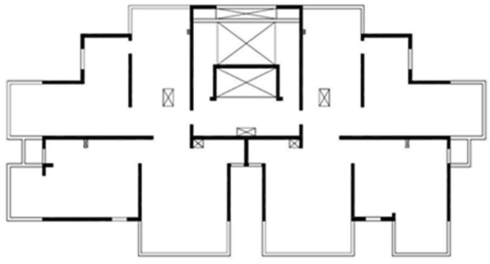


Figure 2. Typical residential building.

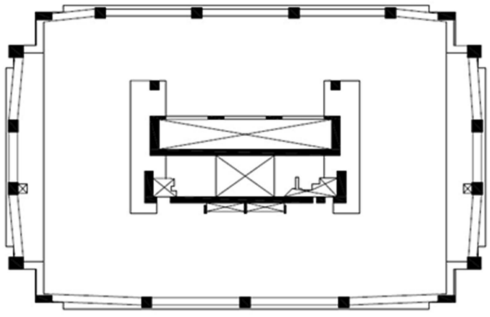


Figure 3. Typical office building.

residential buildings it is easy to turn long partitions into thin structural walls.

Parking facilities for residential and office buildings are always placed below street level requiring normally several underground levels of floor space accounting for 30 to 40 % of the total construction area. Walls at underground levels frequently present setbacks to increase parking space, generating important vertical stiffness irregularities.

At the conceptual stage, most structural engineers in Chile, when allowed by architectural requirements, selectively turn partitions into structural wall with the following simple criteria:

- Assuming the building has an average unit weight per floor area of 10 KPa (1.0 tf/m^2), the wall area in each principal direction at the base floor level, divided by the total floor area above (wall density), must be larger than 0.001. The reason for this comes from an historical code minimum base shear of 6%P, and a conservative average shear stress below 0.6 MPa (6.0 kgf/cm^2), not in the code. This criterion also implicitly limits the average compression in walls to a value less than 5.0 MPa (50 kgf/cm^2).
- The distribution of walls in plan must be as uniform as possible, generating slabs of similar sizes, placing some of the walls at the perimeter for building torsional stiffness.

The usual procedure among the local structural engineers for the definition and fine-tuning of the structural system of a high-rise building after selecting the first array of walls has been:

- Perform a preliminary response spectrum analysis (RSA) scaled to minimum base shear.
- Verification of compliance of the story drift limit at the center of mass (C.M.) at every floor. Usually with the suggested wall density this restriction is immediately achieved.
- Check for the story drift limitation at the perimeter to be within the codes requirement of 0.001 from the C.M. Normally it requires the addition of a perimeter frame formed by properly connecting piers with the upturned-beams as spandrels.
- Fine-tune the wall thickness of each wall along the height to comply with the desired shear stress.

This structuring generated very stiff system. Typical structures follow a period rule close to $T = N/20$.

These simple rules have configured what has been called the typical Chilean RC building.

4.1. Structural Indexes

Several Indexes have been widely used throughout the years in Chile to evaluate the structural characteristics of concrete buildings, with the intent to find a correlation between general structural conception and successful seismic performance. The Indexes presented are related only to global response of buildings under earthquake loads and not to the behavior or design of individual elements.

The Macro approach is the definition of the global system and is the scope of this study. The Micro approach is related to the principles behind the detailing of individual elements that is beyond the scope of this study. Both approaches must be consistent with objectives that define a successful seismic performance.

4.1.1. Wall Density Index:

The wall density measured as the wall area in the first floor on each principal direction divided by the floor area of that floor (Fig. 4), without consideration of the number of floors above, remained constant in time with average values in the range of 2~4%. On the contrary the wall density parameter, d_{np}, calculated as the wall area in the first floor on each principal direction divided by the total weight of the floor area above this level show a clear decay over the years, Fig. 5 (Gómez 2001 & Calderón 2007).

In the last 25 years the graphic shows d_{np} has a constant average value close to 0.002 m^2/tf and a constant minimum of 0.001 m^2/tf. This is consistent with the basic criteria, described previously, for the determination of the wall area required in each principal direction, assuming a unit weight per floor area of 10 KPa (1.0 tf/m^2).

The inverse of the wall density Index has units of MPa (tf/m^2) and is directly related with the average compression forces and the seismic shear forces acting on the

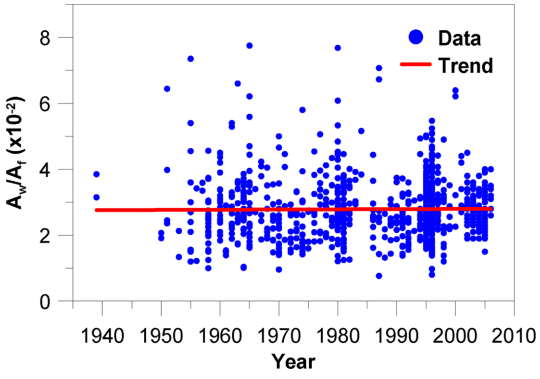


Figure 4. Wall area / Floor area at first story.

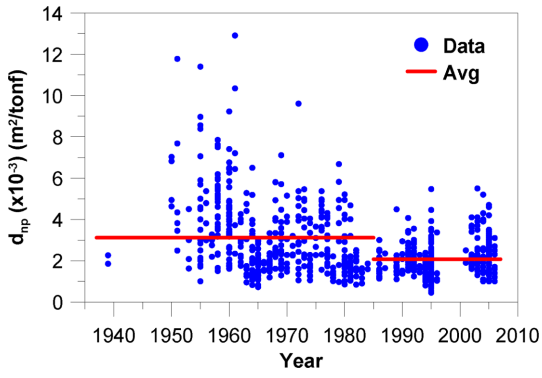


Figure 5. Wall area / Total weight above first story.

walls. A reduction in the value of the wall density Index implies a direct increase in wall compression and shear stresses. Different authors have demonstrated (Wallace et al., 2012) that the maximum roof lateral displacement is dependent of the relation c/l_w that is directly related with the axial load, the geometry and reinforcing of the wall. Walls with L or T shape and setbacks are especially vulnerable to this situation due to large compression stresses at the web when subjected to large lateral displacements. Evidence shows that an important percentage of the damaged walls fall in this category. This type of situation is usually present in modern buildings below ground level where larger spaces for parking facilities are needed.

Wall density values above 0.001 m^2/tf in each principal direction have proven to provide adequate earthquake behavior when properly designed. It becomes evident that design of shear walls must follow capacity design principles to provide individual ductile behavior in order to guarantee a global successful behavior for the building under large lateral displacements. General practice, with some exceptions, prior to 2008 did not follow these principles due to the Chilean code exclusion of the ACI 318 requirement for transverse reinforcement in boundary elements in walls. This made walls vulnerable when subjected to large displacements such as the observed on soft soils in Concepción, Viña del Mar and Santiago.

*4.1.2. Effective Spectral Reduction Factor R**:*

Figure 6 illustrate code values for the reduction factor R*, and the impact of the incorporation of the minimum base shear requirement that turns R* into R_1 (the equivalent reduction factor to reach the minimum code shear) for a single degree of freedom system (1-DOF). The Design Response Base Shear is amplified by 1.4 for evaluation at ultimate load.

The Effective Spectral Reduction Factor R** (R** = Elastic Response Base Shear / 1.4 Design Response Base Shear) is evaluated for a database of 1280 buildings in Zone 2, Soil Type 2 and for 115 buildings in Zone 3, Soil Type II (designed by René Lagos Engineers). The trend shows that for buildings with natural periods above 1.5 sec. values for R** are in the range of 1 to 4. For buildings with natural periods around 0.5 sec., the zone where minimum base shear starts to control design, R** has the highest values, in the range of 4 to 5.5.

4.1.3. Modified Displacement Ductility Ratio Index μ_Δ:*

$\mu_\Delta^* = \delta_u / 1.4\delta_d$

The maximum roof lateral displacement δ_u is defined in the current post earthquake version of the Chilean code NCh433 established in DS61 MINVU 2011 as 1.3 times the Elastic Displacement Spectrum S_{de} for the cracked translational period with the largest mass participation factor in that direction. This value can be assumed as the roof displacement for the Deterministic Maximum Considered Earthquake (MCE) due to the high frequency of large magnitude earthquakes in Chile as was commented in the Chilean Seismicity section.

The determination of the roof yield displacement δ_y (Fig. 7) normally requires a "pushover" analysis after the final design of a building is done. This procedure has been used only on special projects since in not required by the code. For this reason this displacement is seldom well established for buildings in normal projects. Values for δ_y between 2 and 3 times the design displacement δ_d of the NCh433.Of96 code have been reported in the local practice, assuming soil type remains unchanged.

On the other hand, the design displacement δ_d of the code, calculated as the elastic value based on gross inertia, reduced by R** (Fig. 7) is a well-documented value in every project.

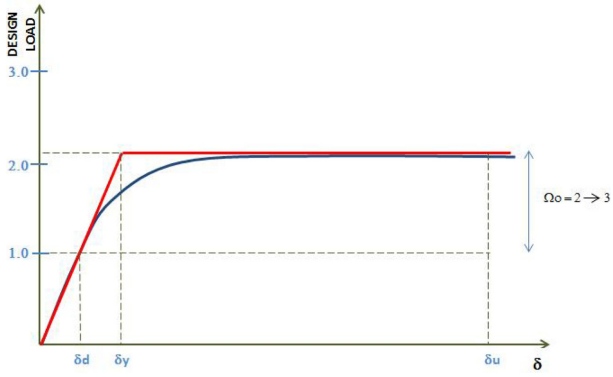


Figure 7. Capacity diagram obtained by pushover analysis.

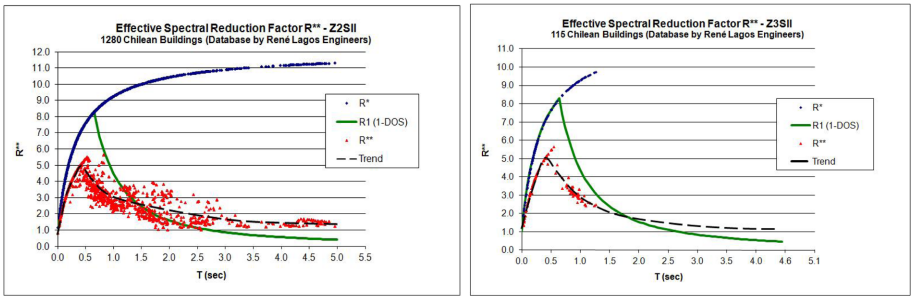


Figure 6. Effective Spectral Reduction Factor R** for 1280 buildings in Zone 2, Soil Type II, and for 115 buildings in Zone 3, Soil Type II (Database from René Lagos Engineers).

To assess the global displacement ductility demand of a building, it can be stated that this is less or equal to the individual displacement ductility demand of the fist wall to enter the inelastic range. Furthermore, this can be expected to happen anytime the roof displacement becomes larger than 1.4 times the design displacement. For this reason a Modified Displacement Ductility Ratio Index μ_Δ* is defined as the ratio between the roof displacement for δ_u at MCE and 1.4 times the design displacement δ_d of the code, in order to establish an upper limit for the global displacement ductility demand of a building. This index is evaluated for a database of 1280 buildings in Zone 2, Soil Type 2 and for 96 buildings in Zone 3, Soil Type III (designed by René Lagos Engineers). Figure 8 shows that average values of μ_Δ* decrease for increasing values of T(sec). Buildings with natural periods above 1.5 sec. have values for the index below 3. For buildings with natural periods below 0.5 sec., the index values increases rapidly (with a large dispersion) as the period decreases, presenting values in the range 2 to 8. This correlates with the evidence that shows that the majority of the damaged buildings had their uncracked first natural period around 0.6 seconds.

4.1.4. Stiffness Index or Structural Response Velocity V = H_o/T:*

It is the quotient of the Height of the building above ground level (H_o) divided by the uncracked First Translational mode period of the building calculated from spectral analysis (T). The units are meters/sec. which represents a velocity. Figure 9 show historical values from a database of 2622 Chilean buildings (Guendelman et al., 2010). Values for H_o/T are in the range of 20~160 m/sec. Values below 40 m/sec. apply to flexible mostly frame buildings; values between 40 and 70 m/sec. represent normal stiffness buildings and values over 70 m/sec. pertain to stiff buildings. Historically, Chilean buildings can be classified in the range of stiff to normal according to the Stiffness Index.

The use of the height above ground level H_o in lieu of the total height of the building H in the Index is due to the fact that H_o represents better the vibrational properties of the buildings. This is because the underground portion usually behaves as a stiff box with no significant drift under lateral loads due to the existence of large surrounding concrete retaining walls at the perimeter of the building. Additionally at ground level is where the largest curvature demand for the walls (δ_u/H_o) takes place. Figure 10 illustrates values of the maximum top-level displacement δ_u obtained for historical values of H_o from 2622 Chilean buildings (Guendelman et al. 2010).

4.1.5. Elastic performance spectrum: D = S_{de}/H_o vs V = H_o/T:*

It is directly derived from the elastic displacement

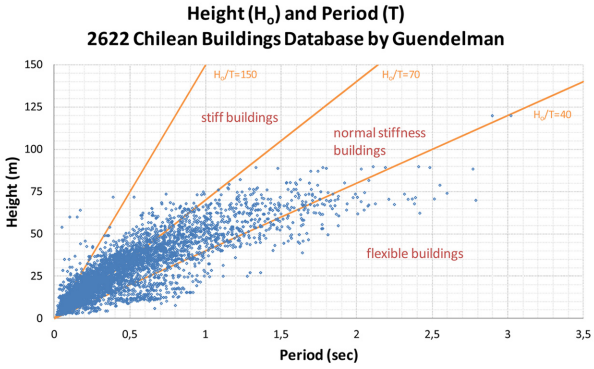


Figure 9. Stiffness Index: H_o/T. (Guendelman et al., 2010)

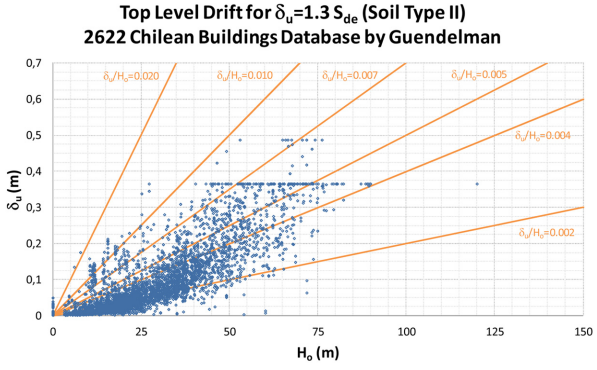


Figure 10. Top level displacement δ_u (m) vs. H_o (m) for Soil Type II. (Guendelman et al., 2010)

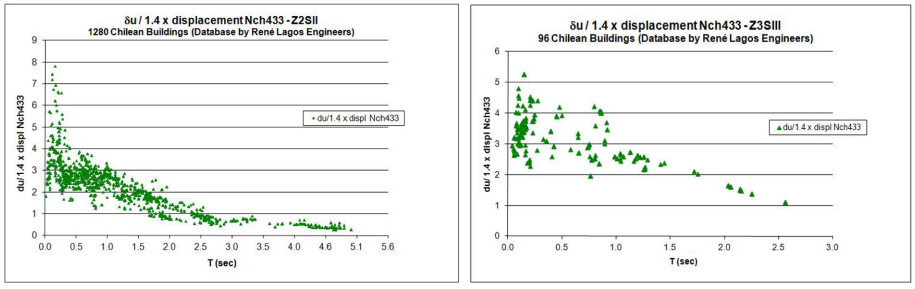
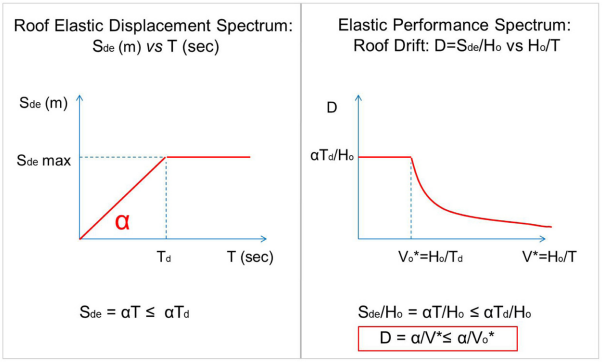


Figure 8. Modified Displacement Ductility Ratio Index μ_Δ* for 1280 buildings in Zone 2, Soil Type II, and for 96 buildings in Zone 3, Soil Type III (Database from René Lagos Engineers).



V* = H_o/T = Structural Response Velocity (m/sec)
H_o = Height of the building from ground level to the top (m)
T = Translational period with the largest mass participation factor (sec)

Figure 11. Determination of the Elastic Performance Spectrum.

spectrum as shown on Fig. 11. The Elastic Performance Spectrum shows that the roof drift is inversely related to the Structural Response Velocity V*. The parameter T_d is site dependent (seismic Zone and Soil Type). The parameter α is site dependent and also dependent of the damping coefficient of the structure β. The parameter H_o is a property of the building.

4.1.6. Performance index δ_u / H_o:

The Performance Index is the top level drift (relative to ground level) evaluated according to current post earthquake version of the Chilean code NCh433 established in DS61 MINVU 2011. The Maximum Lateral Displacement of the roof δ_u is calculated as 1.3 times the Elastic Displacement Spectrum at the top S_{de} for the cracked translational period with the largest mass participation factor in that direction. This index can also be assumed to be the curvature demand of walls at ground level for the Deterministic Maximum Considered Earthquake (MCE), due to the high frequency of large magnitude earthquakes in Chile as commented in the Chilean Seismicity section.

Figure 12 is a plot of the Elastic Performance Spectrum for 2622 Chilean buildings (Guendelman et al. 2010) that illustrate the Performance Index δ_u/H_o vs. the Structural Response Velocity (SRV) parameter V* = H_o/T. In the

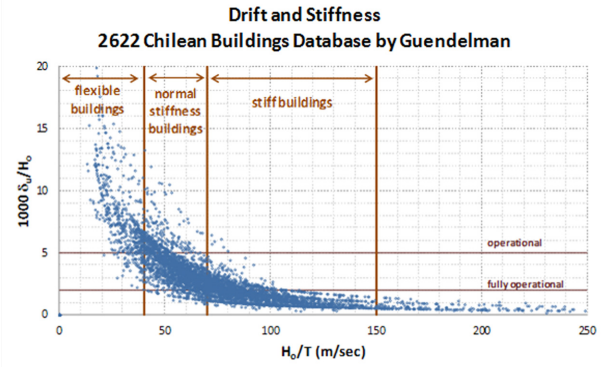


Figure 12. Performance Index: (δ_u/H_o) vs. (H_o/T). (Guendelman et al., 2010)

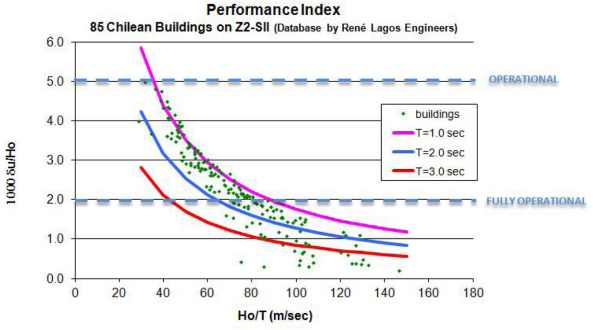


Figure 13. Performance Index: (δ_u/H_o) vs. (H_o/T). (Data base: René Lagos Engineers)

graphic, 88% of the buildings have drift values bellow 0.005 which according to Vision 2000 Performance Objectives (Fig. 14), this represents operational behavior, and 54% have drift values bellow 0.002 which represent a performance objective of fully operational behavior. Less than 2% have drift values above 0.01. It can be noticed that this value is similar to the percentage of building failures reported during the Maule earthquake.

Figure 13 is a plot of the Elastic Performance Spectrum for 85 Chilean buildings (René Lagos Engineers Database, 2012) in seismic Zone 2, Soil Type II where all buildings have roof drift values below 0.005 for the MCE.

Earthquake Design Level	Earthquake Performance Level			
	Fully Operational	Operational	Life Safe	Near Collapse
Frequent (43 years)	Basic Objective	Unacceptable	Unacceptable	Unacceptable
Occasional (72 years)	Essential/Hazardous Objective	Basic Objective	Unacceptable	Unacceptable
Rare (475 years)	Safety Critical Objective	Essential/Hazardous Objective	Basic Objective	Unacceptable
Very Rare (975 years)	Not Feasible	Safety Critical Objective	Essential/Hazardous Objective	Basic Objective

Chilean Type Shear wall buildings

Figure 14. SEAOC Vision 2000 Performance Based Design: Seismic performance objectives.

Figs. 12 and 13 are from different data sources yet they both illustrate that buildings with large values of the Index H_o/T have a better global behavior than buildings with low index values. They also show that the Operational and Fully Operational performance objectives defined in SEAOC VISION 2000 are easily met by buildings with high values of the index H_o/T. It is also evident from the graphics that the most efficient way to increase the seismic performance of a building is by increasing the Structural Response Velocity of the System same as the value of the Index H_o/T. This comparison favors the adoption of shear wall type systems instead of frame type systems as a strategy for increased earthquake performance in high-rise buildings and is consistent with the historical Chilean practice.

4.1.7. Inter-story drift index δ_i/h_s:

It is defined as the ratio between the lateral displacement δ_i between the same point i in plan, at any two consecutive floors, and the floor story height h_s. The Chilean code considers this parameter as a relevant index for stiffness and torsional plan rotation control and damage control of nonstructural components and establishes the following conditions:

- The Inter-story Drift Index must be evaluated under spectrum design forces (reduced forces) including accidental torsion.
- When evaluated at the center of mass (C.M.), the inter-story drift must not exceed the value of 0.002.
- When evaluated at any other point i in plan, the inter-story drift must not exceed 0.001 from the value at the C.M.

Studies based on inelastic models for Chilean earthquakes records (Bonelli, 2008) indicate ratios between maximum inter-story drift vs maximum roof drift between 1.2 and 2.0, the smallest values for shear wall type buildings and the largest values for frame type buildings.

5. Chilean Design Code Performance Objectives

According to historical records hazard studies Chile has seismogenic sources with typical return period ranging from between 80 and 200 years, (Fig. 15). Due to the large magnitude of the design earthquake and the shape of the seismogenic sources and country the design earthquake affects extensive areas. In practice this means that a building can be affected severely by near events as well as far events. An example of this is the capital city Santiago and Valparaiso-Viña del Mar, the two most populated cities in the country. They experienced two extreme (design) earthquakes in 25 years (1985~2010). Additionally the seismic gap in Los Vilos area, 400 km north of Santiago could affect the capital again in the near future. Another example is Concepción, with earthquakes of magnitude 9.5 and 8.8 in 1960 and 2010, a 50 year span.

For practical reasons, design engineers assume that at least one design earthquake will affect the structure in its life span. Experience has indicated that several intermediate earthquakes will also affect the structure. For this reason, the preferred design performance objective is immediate occupancy for the design earthquake.

The Chilean building code mentions three design earthquakes (frequent, intermediate and extreme) and one performance objective for each earthquake. Nevertheless there is no description of how to provide this performance



	DATE	MAGNITUDE	APROX. LOCATION
1570	Feb.8	8 a 8 ½	Concepción.
1575	Dec. 16	8 ½	Valdivia.
1604	Nov. 24	8 ½	North de Arica.
1647	May. 13	8 ½	Valparaíso
1657	Mar. 15	8	Concepción
1730	Jul. 8	8 ¾	Valparaíso
1737	Dec. 24	7 ½ a 8	Valdivia
1751	May. 25	8 ½	Concepción
1796	Mar. 30	7 ½ a 8	Copiapó
1819	Apr. 3-11	8.3	Copiapó (3 EQ).
1822	Nov. 19	8 ½	Valparaíso
1835	Feb. 20	8 a 8 ½	Concepción.
1837	Nov. 7	8	Valdivia.
1868	Aug. 13	8 ½	Arica.
1877	May. 9	8	Iquique
1880	Aug. 15	7 ½ a 8	Illapel
1906	Aug. 16	7.9	Valparaíso.
1922	Nov. 10	8.4	Vallenar
1928	Dic. 1	8.4	Talca
1939	Jan. 24	8- 8.3	Chillán
1943	Abr. 6	8.3	Illapel
1950	Dec. 9	8.0	Calama
1960	May. 22	9.5	Sur de Chile
1966	Dec. 28	8.1	Taltal
1985	Mar. 3	7.8	Zona Central
1995	Jul. 30	8.0	Antofagasta
2001	Jun. 23	8.4	South Peru
2005	Jun. 13	7.8	Tarapacá
2010	Feb. 27	8.8	Center-South

Total ±30 events in 440 years = Aprox. 1 every 15 years,
Lomnitz, Campos, Comte, Riddell, Boroschek otros

Figure 15. Major Chilean Earthquakes, 1570 to 2010.

objective or how to design for an earthquake different from the design one. The Chilean building seismic code written requirements aims only to provide "life safe" performance for the design event which is considered having a return period of 475 years.

The earthquake experience has shown that the response of the Chilean buildings has been close to operational. This can be attributed to the fact that the drift of most engineered buildings designed in accordance with the Chilean practice falls below 0.5%, as can be seen on Figs. 12 and 13. It is also known by experience that for frequent and even occasional earthquakes, buildings responded elastically and thus with "fully operational" performance. Taking the above into account, it can be said that, although the "basic objective" of the Chilean code is similar to the SEAOC VISION2000 criteria (Fig. 14), the actual performance for normal buildings is closer to the "Essential/Hazardous objective".

The Chilean industrial code NCh2369 promotes a higher performance objective than the building code. Again there is no specific reference on how to apply the procedures to provide this performance objective. The main differences are: the limitation of the reduction factor to smaller values and the consideration of different damping values according to material and structural configuration. So the design practice is to increase resistance and stiffness and to provide appropriate detailing to obtain at least limited ductility.

Other design code or recommendation, like the highway and bridge design code and the electrical facilities code provide performance objectives but no performance procedures.

6. Chilean Code Changes after the 2010 Earthquake

After the 2010 Maule Earthquake, changes have been made to the codes through government administrative procedures established in DS60 MINVU 2011 for the Design of RC Buildings and the DS61 MINVU 2011 for the Seismic Demands for Buildings.

6.1. NCh433 changes introduced by DS61 MINVU 2011 for the Seismic Demands for Buildings:

- A new Soil Type classification is introduced considering the dynamical soil properties based on Vs30 measurements below the surface level, defining soils types A, B, C, D, E and F, renaming approximately Soil Type I as A, II as B, a new type C, III as D, IV as E and a new type F.
- The existing pseudo-acceleration spectrum is multiplied by a new parameter *S*, dependent of the soil, with values 0.9 for Soil Type A, 1.0 for soil B, 1.05 for soil C, 1.20 for soil D and 1.30 for soil E. Soil type F, requires a site assessment of seismic hazard.

$$S_a = \frac{SA_0\alpha}{(R^*/I)}$$

A new Elastic Displacement Response Spectra S_{de} is introduced.

$$S_{de}(T_n) = \frac{T_n^2}{4\pi^2}\alpha A_0 C_d^*$$

The parameter C^*_d is dependent of the soil type and the natural period of the building, having values larger than 1.0 for calibration with the observed displacements at ground level under the most severe earthquake between 1985 and 2010. Conceptually this spectrum corresponds to an increase of the displacement derived from the pseudo-acceleration spectrum in the code NCh433 and is close to the Displacement Spectra from the Chilean Code NCh2745-2003 for base isolated buildings (Fig. 16B).

- For concrete buildings, the Maximum Lateral Displacement at the roof of the building δ_u is defined. This is calculated as 1.3 times the value of the Elastic Displacement Response Spectrum at the top S_{de} for the cracked translational period with the largest mass participation factor in that direction, for 5% of critical

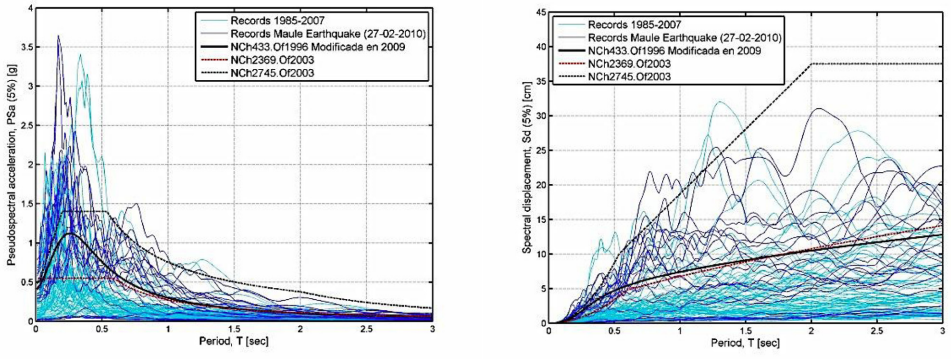


Figure 16. Response spectra at 5% percent damping for records obtained in Zone 3 and Soil Type II according to Chilean seismic codes. Elastic demands of NCh433, NCh2369 and NCh2745 are shown. A) Acceleration Spectra. B) Displacement Spectra.

damping.

6.2. NCh430 changes introduced by DS60 MINVU 2011 for the design of RC buildings:

Adoption of ACI 318-08 provisions, with some minor exceptions, for the design of concrete special structural walls. These provisions are intended to prevent crushing and spalling of concrete and buckling of vertical reinforcement at boundary regions by providing a ductile behavior to individual walls and placing a limit of 0.008 to the maximum compression strains when the building reaches the Maximum Lateral Displacement at the roof δ_u .

Changes in the design for flexure and axial force:

21.9.5.2 - The whole flange width of a flanged section T, L, C, or other cross sectional shapes must be considered. The total amount of longitudinal reinforcement present in the section must be considered when assessing the flexural strength due to combined flexural and axial loads.

Alternatively, effective flange widths of flanged sections can be considered. The effective flange width shall extend from the face of the web a distance equal to the smaller of one-half the distance to an adjacent wall web and 25 percent of the total wall height.

21.9.5.3 - Factored axial load acting on transverse section defined in 21.9.5.2, must be less or equal to *0.35fc' Ag.*

21.9.5.4 - In every wall with an aspect ratio *Ht/lw* greater or equal to *3*, in the critical section the curvature capacity, ϕ, must be greater than the demand of curvature, ϕu. Curvature capacity can be evaluated using equation (21-7a) or (21-7b). The axial load is the greatest factored axial load that is consistent with the design load combination that produces the design displacement δu. Shortening strain, ε_c, in the most compressed fiber in the critical section of a wall, shall be less or equal to *0.008*.

$$\phi_u = \frac{2\delta u}{H_t l_w} = \frac{\varepsilon_c}{c} \leq \frac{0.008}{c} \qquad (21\text{-}7a)$$

$$\phi_u = \frac{\delta_u - \delta_e}{l_p\left(H_t - \frac{l_p}{2}\right)} + \phi_e = \frac{\varepsilon_c}{c} \leq \frac{0.008}{c} \qquad (21\text{-}7b)$$

lp value in equation (21-7b) shall not be greater than *lw/2* and ϕe and δe must be justified.

The total amount of longitudinal reinforcement present in the transverse section defined in 21.9.5.2 must be considered, subjected to the axial load *Pu*. The deformation capacity must be assessed in the wall plane consistent with de direction of analysis.

Additional changes for the design for bending and axial load of shear walls in the code are:

- Slenderness: minimum wall of 1/16 of the unbraced length.
- Splices in longitudinal reinforcement: transverse reinforcement must be provided at lap splices.
- Bar buckling: spacing of transverse reinforcement must be ≤ 6 longitudinal bar diameter.

7. Correlation between Displacement Demand and Seismic Sructural Damage Using the Parameters H_o/T and the Concrete Compresive Strain ε_c

To study the effect of axial loads through the neutral axis depth in walls, compressive strain curves in the concrete are constructed, which correlates with the compression wall damage, versus neutral axis depth normalized by the wall length (c/l_w) obtained for Zone 3 and Soil Type D, as shown in Fig. 17. In this case, the value of c/l_w was limited to 0.55 that is close to the balance condition. The analysis assumes a story height of 2.5 m.

The displacement spectrum is determined assuming the "equal displacement rule" between elastic and inelastic displacement expected for a building with several degrees of freedom ($1.3S_d$) with cracked period representing flexible, moderate rigidity and rigid structural wall buildings ($T_{cr}(s) = 1.5*h_w/40$, $1.5*h_w/70$ and $1.5*h_w/100$, with h_w (m) = H_o the building height) as indicated in D.S. Nº 61 MINVU (2011).

For comparison purposes the simplified expression (21-7a) in D.S. Nº 61 MINVU (2011) for determining the strain due to roof displacement is considered (total curvature concentrated at the end of the wall within a plastic hinge), which is more conservative. Considering the elastic (yield) component would result in smaller compressive strains and in cases of tall rigid buildings might even be enough to reach the expected top lateral displacement. Furthermore, these analyzes are shown for buildings with different numbers of floors (N = 15, 20 and 25 floors).

In general, larger number of stories results in larger strain values, with the exception of flexible buildings, where due to intermediate cracked period values (1.5 to 2.5 sec.) almost identical top lateral displacement are determined (displacement spectrum plateau), resulting in lower drift levels for taller buildings, and therefore less strain values. Furthermore, higher building stiffness has the benefit of reducing displacement demands, reducing

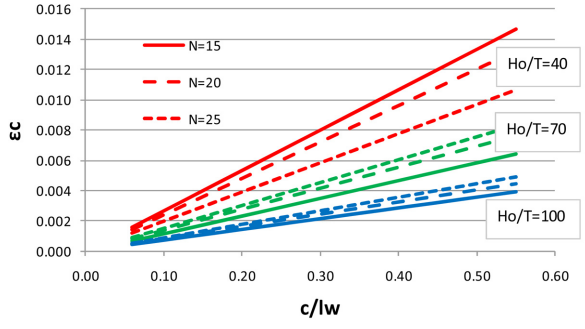


Figure 17. Compressive strain estimation in walls (compressive damage).

damage. It can be seen that for $H_o/T = 100$, strain values hardly exceeds the limit of 0.003 (limit state to require confinement in the boundary wall elements), for neutral axis depths below $0.35l_w$, independent of the number of floors. More flexible structures are more susceptible to damage with increasing axial load. It is also important to note that for low axial loads (c/l_w low), compression damage is unlikely to occur, independently of the stiffness of the structure and number of floors. The incorporation in the Chilean code of a limit compressive strain of 0.008 (compression damage limit state), can be understood as a reduction in axial load levels (or neutral axis depth), but equally it can be understood as favoring more rigid structures as is the usual Chilean practice, since as shown in Fig. 17, this level is exceeded practically only for $H_o/T = 40$ with levels of c/l_w above 0.3. In the case of structures with moderate rigidity ($H_o/T = 70$), it is exceeded only at high levels of c/l_w (about 0.5).

8. The Chilean Practice vs Performance based Design

8.1. Conceptual framework

The short interval between large magnitude earthquakes has conditioned the Chilean seismic design practice to aim an objective of almost operational performance level, despite the fact that the Chilean Code declares a scope of life safe performance level.

Nevertheless the design of a structure should consider eventual non-linear behavior, by providing to it adequate capacity and ductility. To do that, seismic design normally establishes a single reduction factor of the spectral accelerations, dependent on the period of the main translational mode in the direction of the analysis, and consequently, allows performing a linear dynamic analysis for that demand. But the demand reduction has to be proven for large earthquakes, in order to confirm that capacity and ductility provided are adequate. To do that, a more rigorous non-linear response method should be used, such as a time-history procedure, but it produces various difficulties, only partially resolved to date by the best known computer programs. On the other hand, nonlinear equivalent static methods (Chopra and Goel-1999; FEMA 356-2000; FEMA 450-2003; among many others) have emerged forcefully in the past 15 years.

The use of these procedures requires to estimate the capacity of the structure with incremental techniques referred to as "pushover", consisting in the application of a set of lateral monotonically increasing forces acting over the structure, associated with a fixed pattern, generally of inverted triangular type. Degradation of stiffness is the result of plastic hinges generated at the ends of the critical elements. The load pattern is considered closely related to the predominant mode of vibration, and additionally, it is assumed to be independent of the stiffness degradation.

Although results generally show a reasonable degradation mechanism, it is important to investigate in greater depth the rectifications of this mono modal pushover, because the pattern of forces that exclude higher order mode effects might ignore plastic behavior of elements located in the upper stories of the building,

The developments of the past decade have allowed the extension of the mono modal pushover to the so called "modal pushover" (Chopra and Goel, 2005), whose scope is much broader than the first one, though still not exempt from questioning.

The combination of the capacity diagrams, obtained with mono or multi modal pushover, with the equivalent demand diagram associated with inelastic spectra, has generated a very practical methodology named "Demand-capacity Procedure", which is available in many computer programs, both, private or commercial, of vast international use.

The above synthesizes, in general terms, the procedures actually used in some engineering offices in Chile when involved in the design of special or complex high-rise buildings.

As it was mentioned before, the most important practical difficulty in the use of this procedure is reflected in the fact that the structure has to be completely designed prior to determining its capacity. Therefore, this methodology has more merit when used for reviewing rather than for designing purposes.

That is the Chilean practice in Performance Based Design and, examples of the use of the "Demand-capacity Procedure" are among others, the buildings Territoria 3000, Costanera Center Tower 2 and Telefónica Tower, located in Santiago Chile (Figs. 18, 19 and 20). Their studies concluded that the buildings would have elastic behavior during an extreme event such as the 27F 2010 Chilean earthquake (M_w 8.8). All three buildings have values of the Index $H_o/T > 40$ with T calculated from uncracked sections for vertical elements and fully cracked sections for coupling beams. Both, Telefónica Tower and

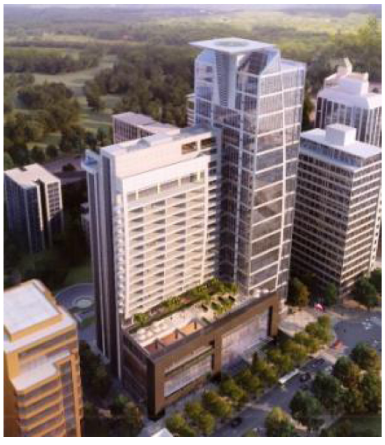


Figure 18. Territoria 3000.



Figure 19. Costanera Center.



Figure 20. Telefonica Tower.

Territoria 3000 had fully operational performance under the February 27th 2010 earthquake. At that time Costanera Center Tower 2 was under construction, with the concrete structure approximately at mid-high.

8.2. Application to structures

Design values that sets the Chilean Code must be obtained from a linear dynamic seismic analysis, for seismic elastic demand reduced by a the factor R*, function of the Modification Factor of Response R_o, and of the period of vibration of the main mode, in the direction of analysis. The modal results are then combined and scaled to accomplish with either the minimum base shear, via an amplification factor, or to the maximum base shear, via a reduction factor. The amplification or the reduction of R* defines the Effective Reduction Factor R**.

Statistics over near 2,000 buildings show that R** values fall below 7, with a high concentration around 4 (Fig. 6). Accordingly, Chilean practice has established the following criteria for performance reviewing of a building design:

- For values of R** equal to or below 3, verification of structural performance is not needed.
- For values of R** between 3 and 7, it is suggested the use of the Demand-Capacity Procedure, with a demand defined by the elastic acceleration spectra.
- For values of R** higher than 7, it is mandatory to perform a nonlinear "Time-History" analysis.

Seismic demands come out from Seismic Risk studies, or in its absence, by an approximate criterion such as:

Earthquake	Demand
Frequent (50% in 30 years)	Elastic spectra / R*
Ocassional (50% in 50 years)	1.4 elastic spectra / R*
Rare (10% in 50 years)	Elastic spectra
Very Rare (10% in 100 years)	1.2 elastic spectra

The expected performance shares the values of Vision 2000 for the basic objectives, as shown below:

Performance Objective	Drift
Fully operational	$d/H_o < 0.002$
Operational	$d/H_o < 0.005$
Life Safe	$d/H_o < 0.015$

The above criteria have been applied in numerous buildings of reinforced concrete of various heights and types (walls, wall-frame, and frameworks). Additionally, it has been applied in several ports structured primarily with steel piles and steel or concrete beams. Many of these structures, both in Chile and Peru, have successfully overcome severe earthquakes.

9. Conclusions

Chile is characterized by the largest seismicity in the world, which produces strong earthquakes every 83±9 years in the central part of country. The different types of earthquakes mean that a building can be affected severely by near source events as well as far events. An example of this is Santiago, the capital, and Valparaíso, the most populated cities of the country. They have experimented two extreme earthquakes in 25 years (1985-2010).

Therefore Chilean practice assumes that for a given building at least one large magnitude earthquake will strike it in its life span.

This short interval between large earthquakes magnitude 8.5 has conditioned the Chilean seismic design practice to achieve almost operational performance level, despite the fact that the Chilean Code declares a scope of life safe performance level.

This large seismicity of Chile leads to a deterministic strategy to assess seismic hazard for design of buildings despite the most used probabilistic approach considered in more low or diffuse seismicity regions of the world.

High-rise concrete buildings constructed in Chile in the past 25 years performed well during the 2010 earthquake. Nevertheless, the earthquake produced significant structural damage on some new mid- rise shear wall buildings never seen on previous earthquakes.

The level of performance observed for the majority of RC high-rise buildings designed according to modern codes such as the ACI 318 was successful when the seismic code provided a reasonable estimate of the displace-

ment demand.

The historical Chilean practice of using high-density shear wall lateral load systems instead of frame type systems has favored the good global performance of high-rise buildings during the 2010 earthquake.

The Structural Response Velocity Index H_o/T has a good correlation with the performance objectives defined as δ_u/H_o according to SEAOC VISION 2000. In buildings with values of $H_o/T > 70$ studies indicate that global elastic response could be expected in firm soils, nevertheless at individual elements level, inelastic behavior may occur. To take advantage of a well-conceived lateral load system, it becomes apparent that the design and detailing of individual elements must be done following capacity design and ductility principles.

Recognizing that the building performance is governed by displacement demand rather than strength, the code NCh433.Of96 drift limitations under reduced design forces with a minimum base shear, led to the adoption of stiff lateral structural systems with high values of H_o/T. This indirectly contributed to the successful performance of high-rise buildings observed during the 2010 earthquake.

Performance Based Design procedures are not included in the Chilean seismic design code for buildings, nevertheless the earthquake experience has shown that the response of the Chilean buildings has been close to operational. This can be attributed to the fact that the drift of most engineered buildings designed in accordance with the Chilean practice falls below 0.5%, as can be seen on Figs. 12 and 13. It is also known by experience that for frequent and even occasional earthquakes, buildings responded elastically and thus with "fully operational" performance. Taking the above into account, it can be said that, although the "basic objective" of the Chilean code is similar to the SEAOC VISION2000 criteria (Fig. 14), the actual performance for normal buildings is closer to the "Essential/Hazardous objective".

The new provisions introduced in the Chilean Codes after the earthquake, continue to move into this direction.

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