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Seismic Performance of Concrete Buildings in Chile

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Abstract

Chile is characterized as the country with the largest seismicity in the world. This produces strong earthquakes every 83±9 years in the Central part of the country, where 60% of the population lives and those of Magnitude 8 or higher occur every 10 years. The short interval between large earthquakes of magnitude 8.0 or larger (3 in the last 5 years) has shown that the Chilean seismic design practice has achieved almost "operational" performance level, despite the fact that the Chilean Code uses “life safety” performance level. This article presents a review of the current Chilean practice for the seismic design of concrete buildings based on the displacement demand records of the last three major earthquakes: Maule 2010 (Mw=8.8), Iquique 2014 (Mw=8.2) and Illapel 2015 (Mw=8.4). Conclusions are drawn for the definition and calibration of the Earthquake Design Levels and the Performance Levels for the new Chilean Code of Performance Based Seismic Design under development.

Keywords: Chilean buildings, Structural Index, Seismic performance, High-Rise, Concrete

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 American Plate, generating frequent interplate type earthquakes some of which have been followed by destructive tsunami, and a similar number of intraplate events.

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 60 % of the country population lives. According to the USGS [20] it is the sixth world largest magnitude earthquake recorded by mankind.

In 1985, a Magnitude M_w 8.0 earthquake affected approximately the northern area of the 2010 event. Between these two earthquakes, a total of 9,974 buildings over 3 stories high were built in this area according to construction permits issued [1]. 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 [1]. No collapses 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.

In April 1st 2014, an M_w 8.2 earthquake affected Iquique, a city with a large number of high-rise buildings in the north of Chile. In September 16th 2015, an M_w 8.4 earthquake followed by a tsunami affected Los Vilos and Coquimbo located in the central and northern part of the country (73 years after the Mw 8.2 Ovalle 1943 earthquake). On these two earthquakes, no collapses and very limited damage were reported on engineered buildings.

2. Chilean Seismicity

Chile is characterized as the country with the largest seismicity in the world which produces strong earthquakes every 83 ± 9 years in the Central part, where Santiago, the capital is located [2].

These earthquakes have happened 6 times in the past, in the years 1647, 1730, 1822, 1906, 1985 and 2010.

This short interval between large earthquakes of magnitude 8.0 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 safety performance level.

The rapid convergence of the Nazca plate under the South American plate and the youth of the Nazca plate, make Chile prone to largest subduction interplate thrust type earthquakes in the world, shallow earthquakes in the South American plate, and intraplate earthquakes in the Nazca plate. In Valdivia, south of Chile, we experienced in 1960 the largest magnitude earthquake, Mw = 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 by far events. An example of this is Santiago, the capital, and Valparaíso/Viña del Mar, the most populated cities of the country. They have experimented two extreme earthquakes in 25 years (1985-2010).

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

This large seismicity has led to a nearly 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 [3]; NCh2369 for industrial facilities [4] and NCh2745 for base isolated buildings [5]. 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 [6], 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 [7] 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 [8], which follows ACI318-05 [9], this exclusion was removed.

A summary of the Code NCh433.Of96 [3] 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 typically taken as: DL + 0.25LL.No Soil-Structure Interaction restriction.
- ***Accidental torsion analysis:*** Accidental eccentricity at level k:

  $e = \pm 0.10\ b\ (Z_k / H)$ 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 by 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: $(\delta/h_{C.M.} - 0.001) \leq \delta/h_{i.} \leq (\delta/h_{C.M.} + 0.001)$
- ***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)

  0.9 DL ± 1.4 E
- ***Spectrum (Fig. 1 and Fig. 2)***

Parameter	Formula	Comments
Design Spectrum	$Sa = \frac{I Ao \alpha}{R^*}$	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

- ***Seismic Zoning:***

Seismic Zone	Geographic Area	A_0
Zone 1	Andes Mountains area	0.20 g
Zone 2	Central strip of Chile between Coastal Mountains and 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 $\geq$ 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***

Build. 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

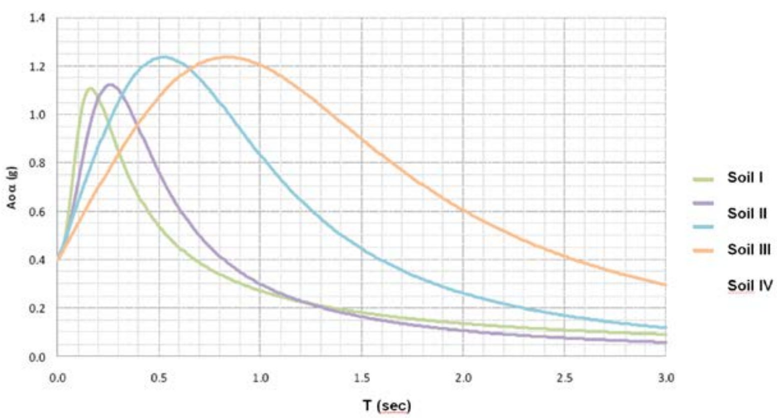


Fig. – 1 Chilean Code NCh433.Of96, Elastic Design Spectrum (R*=1) for seismic Zone 3

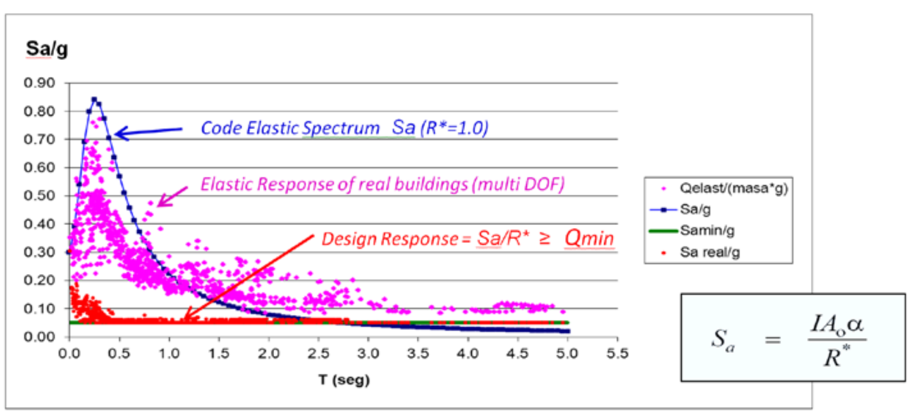


Fig. – 2 Chilean Code NCh433.Of96, Elastic Design Spectrum (R*=1) for seismic Zone 2, Soil Type II and Design Response of 1280 real buildings built in Santiago between 1985 and 2009 (René Lagos Engineers)

4. Characteristics of Chilean Buildings

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

Residential Buildings: (Fig. 3)

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 with no interior beams. The vertical and lateral load systems are RC walls.

Office Buildings: (Fig. 4)

Floor system: Flat post tension slab. Spans 8 to 10m., 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. Office buildings usually have shorter wall length and wider thickness than residential buildings. On residential buildings it is easy to turn long partitions into thin structural walls.

Parking facilities in all 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. In the case of residential buildings they are sometimes located at the first level as well.

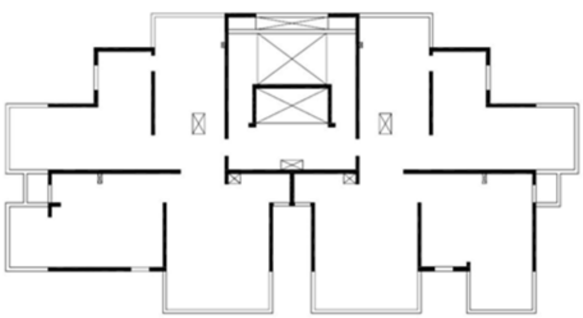


Fig. 3 – Typical residential building

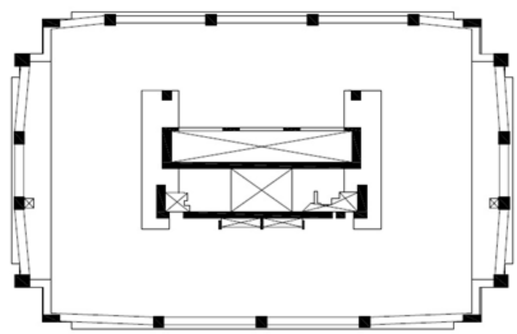


Fig. 4 – Typical office building

At the conceptual stage, most structural engineers in Chile, when allowed by architectural requirements, selectively turn partitions into structural wall, resulting in complex cross-sections with large redundancy, with the following simple criteria:

- Assuming that the building has an average unit weight per floor area of 10 KPa (1.0 tf/m2), the ratio of wall area in each principal direction at the base floor level (can be extended to any floor level), to the total floor area above, must be larger than 0.1% (wall density). The reason for this comes from an historical code minimum seismic base shear of 6%P, and a conservative average shear stress in walls below 0.6 MPa (6.0 kgf/cm2), not included in the code. This criterion also implicitly limits the average compression in walls to a value less than 5.0 MPa (50 kgf/cm2).
- 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 procedure generates very stiff lateral load systems. Typical structures follow a period rule close to T= N/20, with N = number of stories.
These simple rules have configured what has been called the **Chilean Building**.

5. Structural Indexes

Several indexes have been widely used throughout the years in Chile to evaluate the structural characteristics of RC 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 general structural conception approach is the definition of the global structural system and is the scope of this study. The principles for the detailing of individual elements, has not been considered within the scope of this study due to space limitations. Both approaches must be consistent with objectives that define a successful seismic performance.

- ***Wall Density Index***:

The Wall Density Index, 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 as seen in Figure 6, [10,11].

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 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 [12] that the maximum roof lateral displacement is dependent on 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 during the 2010 earthquake falls 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.

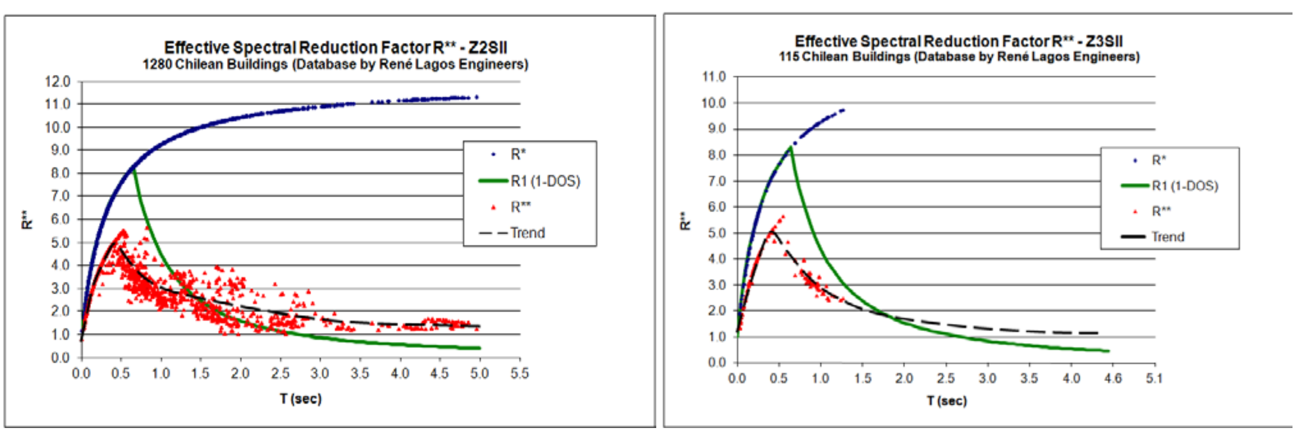


Fig. – 5 Effective Spectral Reduction Factor R** for 1280 buildings in Zone 2, Soil Type II, and for 115 buildings in Zone 3, Soil Type II (Database from René Lagos Engineers).

- ***Effective Spectral Reduction Factor R**:***

Figure 5 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.

- ***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) corresponding to a far subduction mega-earthquake like Maule 2010, Mw=8.8, due to the high frequency of occurrence of large magnitude earthquakes in Chile, as was commented in the Chilean Seismicity section. This earthquake is not the same considered for the resistant design.

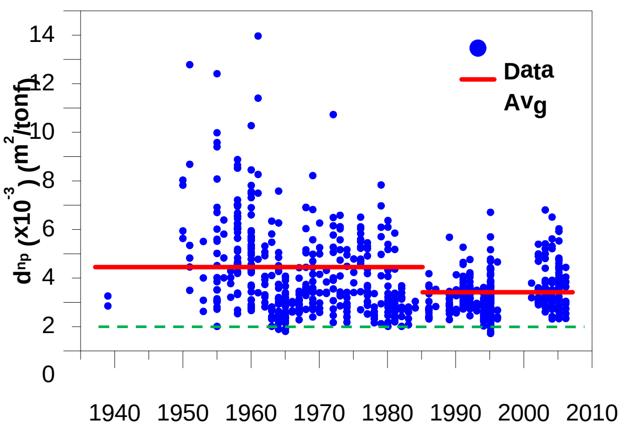


Fig. – 6 Historical Wall Density Index

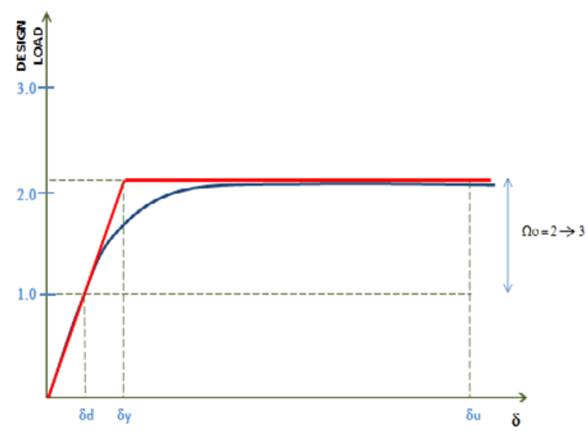


Fig – 7 Capacity diagram obtained by pushover analysis

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 it is not required by the code. For this reason this displacement is seldom well established for buildings in typical 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.

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.

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 first 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. Although many buildings with these characteristics behaved well, it can be noted that this correlates with the evidence that shows that the majority of the damaged buildings had their uncracked first natural period around 0.6 seconds during the 2010 event, showing that large values of the modified displacement ductility ratio produce buildings more susceptible to damage.

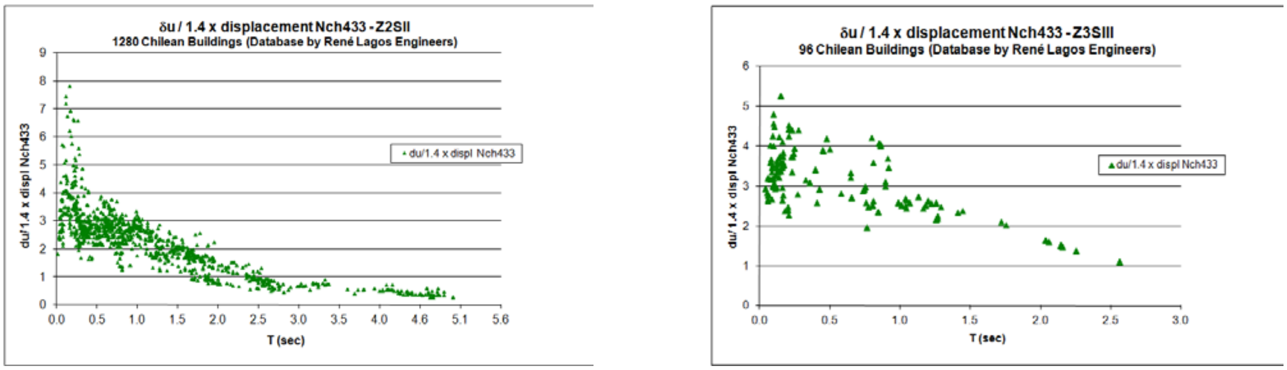


Fig. – 8 Modified Displacement Ductility Ratio Index μ_Δ* for 1280 buildings in Zone 2, Soil Type II, and for 96 buildings in Zone 3, Soil Type III (Database from René Lagos Engineers).

- ***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 (T). The units are meters/sec. which represents a velocity. Figure 9 show historical values from a database of 2622 Chilean buildings [13] [14]. 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 with only frames as structural system; 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 [14].

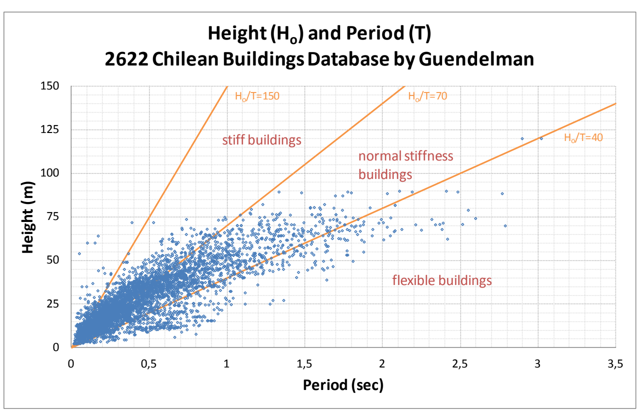


Fig. – 9 Stiffness Index: H_o/T [14]

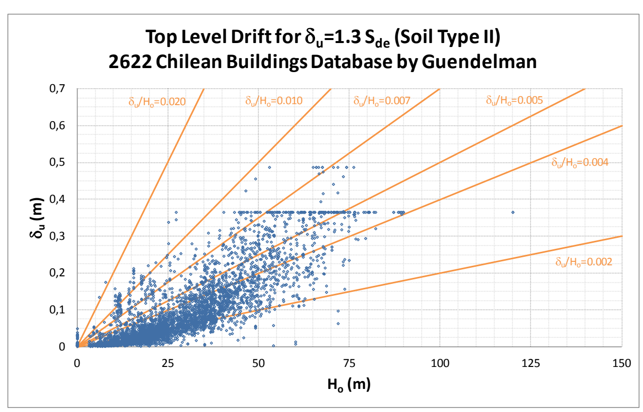
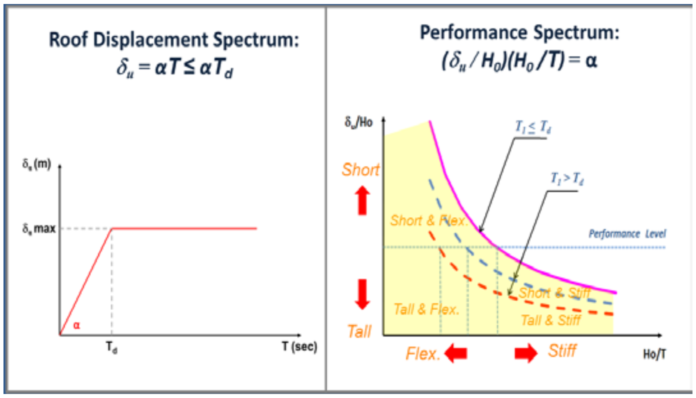


Fig. – 10 Top level drift for Soil Type II. [14]

- ***Performance Spectrum: (δ_u /H_o) vs (H_o/T):***

The Performance Spectrum is the **Roof Displacement Spectrum** plotted as ***(δ_u /H_o) vs (H_o/T)***, as shown on Figure 9. The Performance Spectrum shows that the roof drift is inversely related to the parameter ***(H_o/T)***. 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.
In the Performance Spectrum Graphic, buildings with $T_1 \leq T_d$ are located in the curve $(\delta_u /H_o)(H_o/T) = \alpha$ while buildings with $T_1 > T_d$ are located below the curve, in the shaded area as shown in figures 9 and 10.



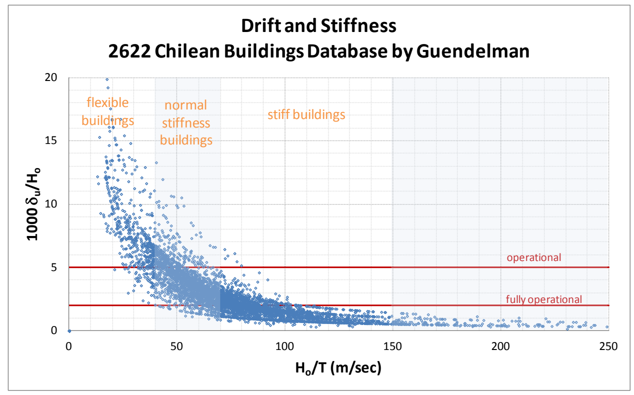


Fig – 9 Performance Spectrum concept. Fig.– 10 Performance Index for 2622 Chilean buildings [14]

- ***Performance Index δ_u / H_o:***

The Performance Index is the top level drift (relative to ground level) evaluated for the maximum roof lateral displacement δ_u. Figure 10 is a plot of the Performance Index δ_u/H_o vs. the Stiffness Index H_o/T for 2622 Chilean buildings [14] that illustrates the concept of the Performance Spectrum for Chilean Type buildings. In the graphic, 88% of the buildings have drift values bellow **0.005** which according to Vision 2000 Performance Objectives [18], represent **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.

- ***Inter-story Drift Index δ_i / h_s:***

It is defined as the ratio between the lateral displacement $\boldsymbol{\delta_i}$ between the same point ***i*** in plan, at any two consecutive floors, and the floor story height $\boldsymbol{h_s}$. 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:
- 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, must not exceed 0.001 from the value at the C.M.

Studies based on inelastic models for Chilean earthquakes records [15] indicate ratios between maximum roof drift vs. maximum inter-story drift between 1.2 and 2.0, the smallest values for shear wall type buildings and the largest values for frame type buildings.

6. Code Changes after Chile 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 [16] and the DS61 MINVU 2011 for the Seismic Demands for Buildings [17].

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 V_{s30} 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 *S*a is multiplied by a new parameter *S*, dependent of the soil, with values of 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. A new Elastic Displacement Response Spectra S_{de} is introduced

$$S_a = \frac{SA_0\,\alpha}{(R^*/I)} \qquad\qquad S_{de}(T_n) = \frac{T_n^{\,2}}{4\pi^2}\,\alpha A_0 C_d^{\,*}$$

  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.
- 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 damping.

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 . This limit indirectly reduces the axial load (because the neutral axis depth is limited), when large compressive strains are expected, such as in walls with large amount of reinforcing bars in tension as it is common in sections with flanges in tension (i.e., L, T). Thus, characteristics that increase the compressive strain at wall base, such as, buildings in soft soils or flexible buildings, which increase the displacement demand, and large axial load in walls or large longitudinal reinforcement bars in tension, that increase the compressive zone (deeper neutral axis), are controlled.

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, $\boldsymbol{\phi}$, must be greater than the demand of curvature, $\boldsymbol{\phi_u}$. 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 $\boldsymbol{\delta_u}$. Shortening strain, $\boldsymbol{\varepsilon_c}$, in the most compressed fiber in the critical section of a wall, shall be less or equal to ***0.008***.

$$\emptyset_u = \frac{2\delta_u}{H_t l_w} = \frac{\varepsilon_c}{c} \leq \frac{0.008}{c} \quad (21\text{- }7a)$$

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

$\boldsymbol{l_p}$ value in equation (21-7b) shall not be greater than $\boldsymbol{l_w /2}$ and $\boldsymbol{\phi_e}$ and $\boldsymbol{\delta_e}$ 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. Conclusions

Chile is characterized by the largest seismicity in the world, which produces strong earthquakes every 83 ± 9 years in the central part of the country. The different types of earthquakes mean that a building can be affected severely by near source events as well as by far events. An example of this is Santiago, the capital, and Valparaíso/Viña del Mar, 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 of magnitude 8.0 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 safety 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 displacement 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 Figures 10. 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**".

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

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