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PERFORMANCE OF HIGH RISE BUILDINGS UNDER THE FEBRUARY 27TH 2010 CHILEAN EARTHQUAKE

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ABSTRACT: During the earthquake (Mw 8.8), high rise shear wall buildings in the affected area performed well. Less than 1% of these were severely damaged or had to be demolished. Nevertheless, the earthquake produced significant structural damage on some new high rise shear wall buildings in Santiago, Viña del Mar, Chillán and Concepción. Lateral loads systems and design criteria commonly used in concrete buildings in Chile are reviewed. Explanations for the successful seismic performance of concrete buildings and lessons learned from the observed failures are discussed.

Key Words: Maule earthquake, earthquake engineering, seismic design, high rise buildings, shear wall, damage, Chile

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 penetrates under the South America Plate generating frequent subduction type earthquakes usually followed by 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 500 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 same area of the country. 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 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 only suffered nonstructural damage and in some cases minor reparable structural damage.

The majority of the damaged buildings had their first natural period of vibration with values around 0.6 seconds. This corresponds to mid-rise buildings. Evaluation studies of the buildings severely damaged indicate that in 50% of them, failure can be attributed to sub-classification of foundation soil, resulting in larger displacements demands than those anticipated. The other 50% had failures attributed to structural reasons that could have been avoided.

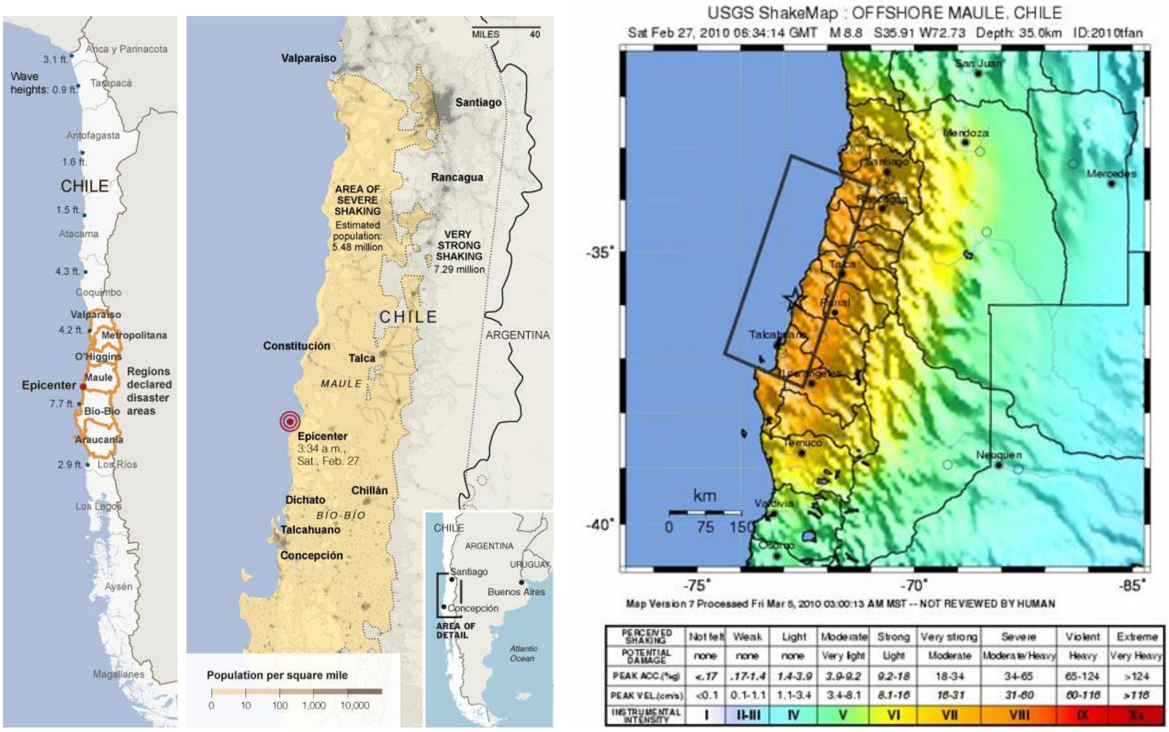

Fig. 1 Geographical area affected by the February 27th 2010 Chilean earthquake.

On RC buildings, observed damage was concentrated near the base, on L or T shape walls, presenting crushing and spalling of concrete and buckling of vertical reinforcement at boundary regions, extending horizontally deep into the wall length. Vertically the observed damage only affects a small portion of the story high. Shear failures at the web were also observed. Damage in slabs was observed, due to wall rotation at doorways in upper stories.

This paper presents a study of the global response parameters of typical RC buildings historically designed in Chile to characterize the attributes of their structural systems. Also the prescriptions in the seismic code for buildings, NCh433.Of96 are reviewed to determine their influence in these attributes and in the observed acceptable performance of the majority of buildings during this extreme event.

BUILDING 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; NCh2369 for industrial installations 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 ASCE7-10, 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. Building mass from DL + 0.25LL.

Accidental Torsion Analysis: Accidental eccentricity at level k:
$e = \pm 0.10\, b\, (Z_k / H)$ in each principal direction

Base shear limitations: $IA_0\, P/6g \leq$ Base shear $\leq 0.35\, SIA_0P/g$ for concrete buildings.
If Base Shear is out of the range, forces and displacements must be scaled to the exceeded limit.
Forces from the code are 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%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: $\delta_{cm} \leq 0.002$
- Interstory drift at any point *i* in plan: $(\delta_{cm} - 0.001) \leq \delta_i \leq (\delta_{cm} + 0.001)$

***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

Seismic Zoning:

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

Types of foundation 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 $v_s \geq 400$ 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

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	$Sa = \frac{I A o \alpha}{R^*}$	I : importance factor A_0 : zone maximum effective acceleration R^*: reduction factor α : amplification factor

Amplification factor	$\alpha = \dfrac{1+4.5\left(\dfrac{Tn}{To}\right)^{P}}{1+\left(\dfrac{Tn}{To}\right)^{3}}$	T_n : vibration period of mode n T_0 , P : soil parameters
Reduction factor	$R^* = 1+\dfrac{T^*}{0.10To+\left(\dfrac{T^*}{Ro}\right)}$	R_0 : structural system parameter R_0 = 11 for shear wall and braced systems T^* : period of mode with largest translational mass in the direction of analysis

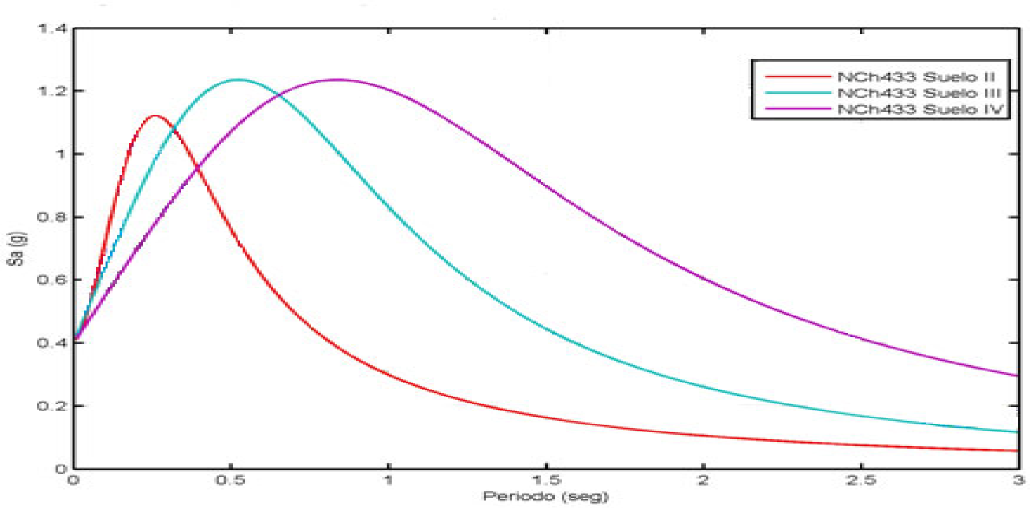

Fig. 2 Chilean Code NCh433.Of96, Design Spectrum for seismic zone 3, for soil type I, II and III

STRUCTURAL SYSTEMS FOR HIGH RISE BUILDINGS IN CHILEAN PRACTICE

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:

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. The vertical and lateral load systems are concrete 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.

The main difference between office and residential buildings is that office buildings 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 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.

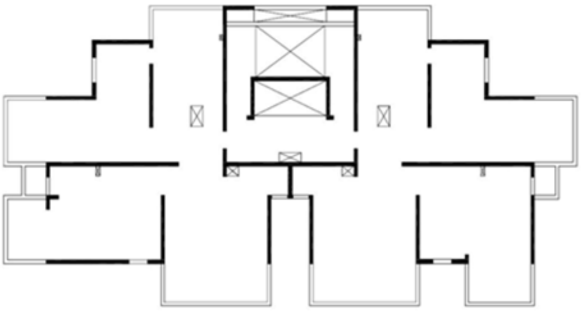

Fig. 3 Typical residential building

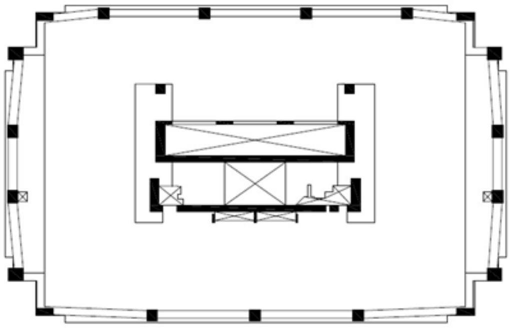

Fig. 4 Typical office building

At the conceptual stage, historical practice in Chile indicates that structural engineers, 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 ton/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 kg/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 kg/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 (CM) at every floor. Usually with the suggested wall density this restriction is immediately achieved.
- Scheck for the story drift limitation at the perimeter to be within the codes requirement of 0.001 from the CM. 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.

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

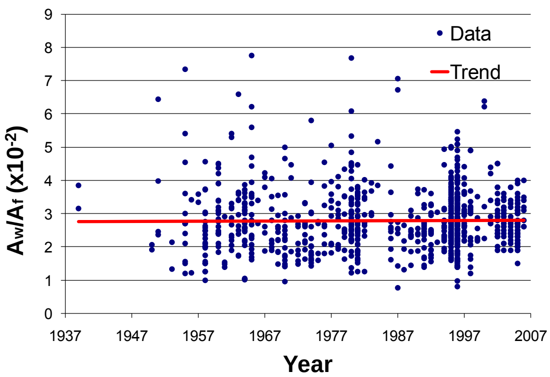

Fig. 5 Wall Area / Floor Area at first story

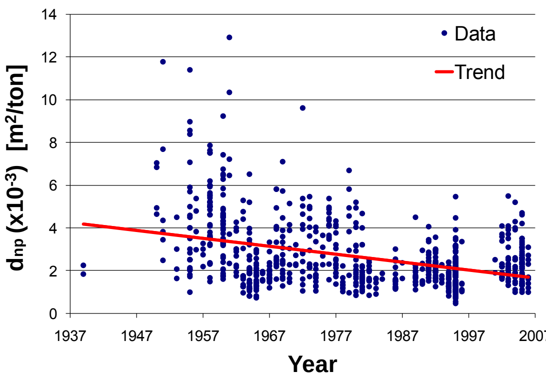

Fig. 6 Wall Area / Total Weight above first story

Structural Indicators

Several indicators 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 indicators presented are related only to global response of buildings under earthquake loads and not to the behavior of individual elements. To achieve a successful performance strategy, the adoption of an adequate general structural system must be properly complemented with the adoption of capacity design principles for the design and detailing of individual elements. The Macro approach is the definition of the global system, and the Micro approach are the principles behind the detailing of individual elements. They must be consistent with objectives that define a successful seismic performance.

Wall Density Indicator d_{np}:
Figure 5 shows the wall area in the first floor on each principal direction divided by the floor area of that floor, without consideration of the number of floors above. This ratio remains constant in time with values in the range of 2-4%. Figure 6 illustrate the evolution of the Wall Density parameter d_{np}. This parameter 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 with units of (m^2/ton), (Gómez 2001 & Calderón 2007).

In the last 60 years there has been trend to a continuous reduction of the average wall density indicator. Nevertheless a constant minimum of 0.001 is observed. 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 ton/m^2).

The inverse of this indicator has units of MPa (ton/m^2) and is directly related with the average compression forces and the seismic shear forces acting on the walls. A decreased value of the wall density indicator implies a direct increase in wall compression and shear stresses. The use of sophisticated structural analysis software and the use of higher strength concrete have led to the use of relatively thin walls, resulting in brittle behavior and poor performance when subjected to large lateral displacements. Different authors have demonstrated (Massone, 2011) 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 0.001m^2/ton 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 an 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.

Stiffness Indicator H / T:
Proposed by Guendelman (2000) it is the quotient of the Total Height of the building (H) divided by the First Translational mode period of the building calculated from spectral analysis (T). The units are meters/sec. which represents a velocity. Figure 7 show historical values from a database of 2622 Chilean buildings (Guendelman, 2010). Values of H/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 indicator.

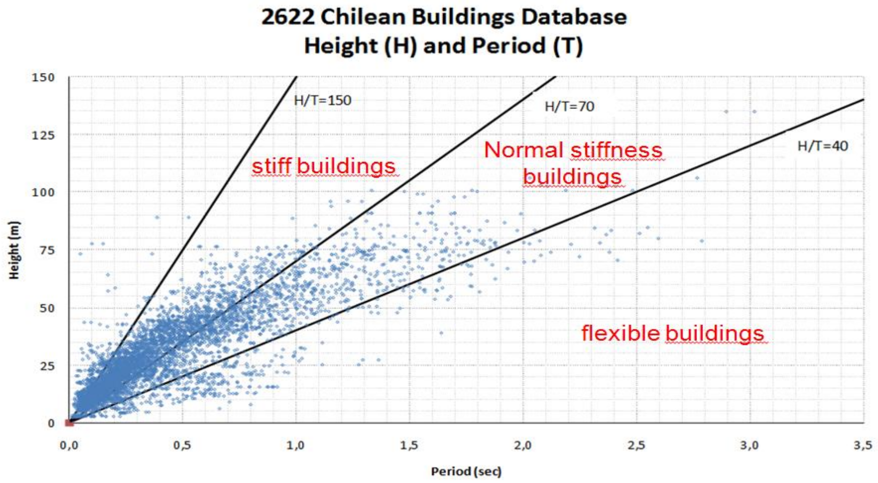

Fig. 7 Stiffness Indicator = H/T (Guendelman, 2010)

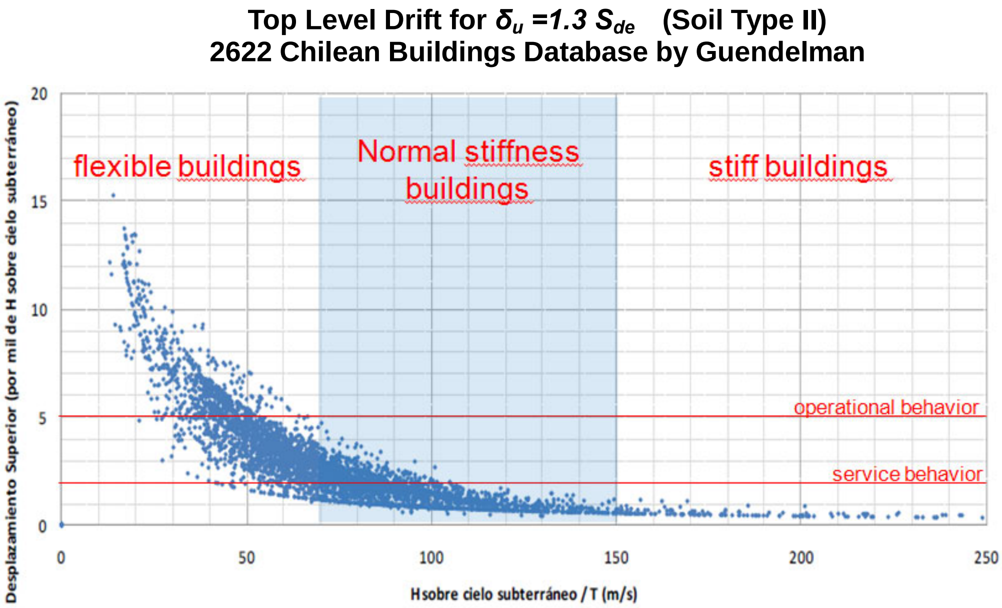

Fig. 8 Performance Indicator: (Top displacement δ_u /H) / vs. H/T parameter. (Guendelman, 2010)

Performance Indicator δ_u / H:

The Performance indicator 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 value can be assumed as the roof displacement for the Deterministic Maximum Considered Earthquake (MCE). Figure 8 illustrate the Performance Parameter δ_u / H vs. the parameter H/T. In the graphic, 88% of the buildings have drift values bellow 0.005 which according to Vision 2000 Performance Objectives, this represents **operational behavior**, and 54% have drift values bellow 0.002 which represent a performance objective of **service 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 8 also illustrate that stiff buildings evaluated under the parameter H/T have a better global behavior than low stiffness systems according to performance objectives defined in SAOC VISION 2000. 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.

Ductility Demand Indicator or Effective Spectral Reduction Factor R**:

R** = Elastic Response Base Shear / 1.4 Design Response Base Shear

Figure 9 illustrate code values for the reduction factor R* multiplied by a factor 1.4 for ultimate load level, and the impact of the incorporation of the minimum base shear requirement that turns R* into R_1 for a single degree of freedom system (1-DOF).

The Ductility Demand indicator R** 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 the ductility demand 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, the ductility demand indicator has the highest values, in the range of 4 to 5.5. This correlates with the evidence that shows that the majority of the damaged buildings had their first natural period around 0.6 seconds. It also shows the importance of the use of capacity design and ductility principles in the design and detailing of walls.

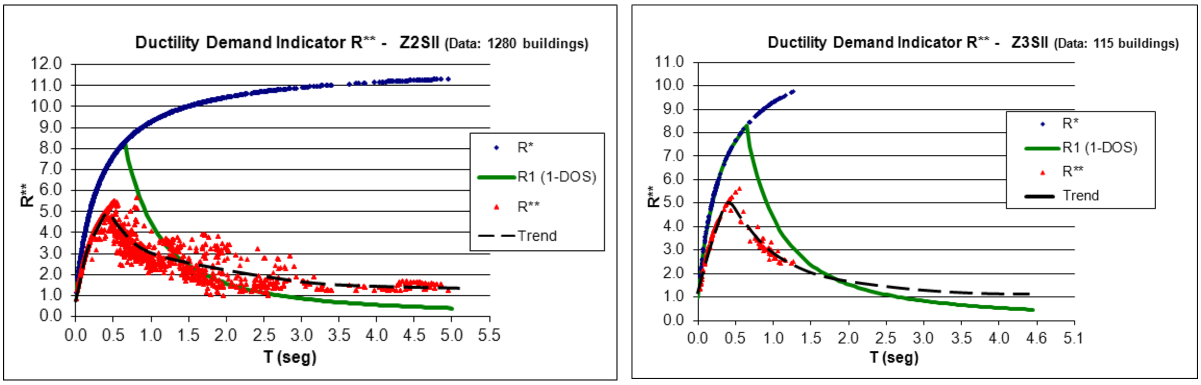

Fig. 9 Ductility Demand Indicator R** for 1280 buildings in zone 2, soil type II, and for 115 buildings in zone 3, soil type II (Database from René Lagos Engineers).

Required Displacement Ductility Ratio Indicator μ_Δ*:

μ_Δ* = δ_u / 1.4 roof design displacement according to NCh433.Of96

The Required Displacement Ductility Ratio Indicator μ_Δ* 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 10 shows that buildings with natural periods above 1.5 sec. have values for the displacement ductility demand ratio below 3. For buildings with natural periods below 0.5 sec., the displacement ductility demand ratio increases rapidly (with a large dispersion) as the period decreases, presenting values in the range 2 to 8. The existence of values around 2 in all the range of periods evaluated suggest that the natural period is not an appropriate parameter for the evaluation of the displacement ductility demand ratio, regardless the observed tendency that average values of μ_Δ* decrease for increasing values of T(sec). Studies to evaluate how the indicator μ_Δ* varies with the parameter H/T are in progress in order to evaluate the influence of the lateral load structural system on the indicator.

Figure 10 also shows the importance of the use of capacity design and ductility principles in the design and detailing of walls.

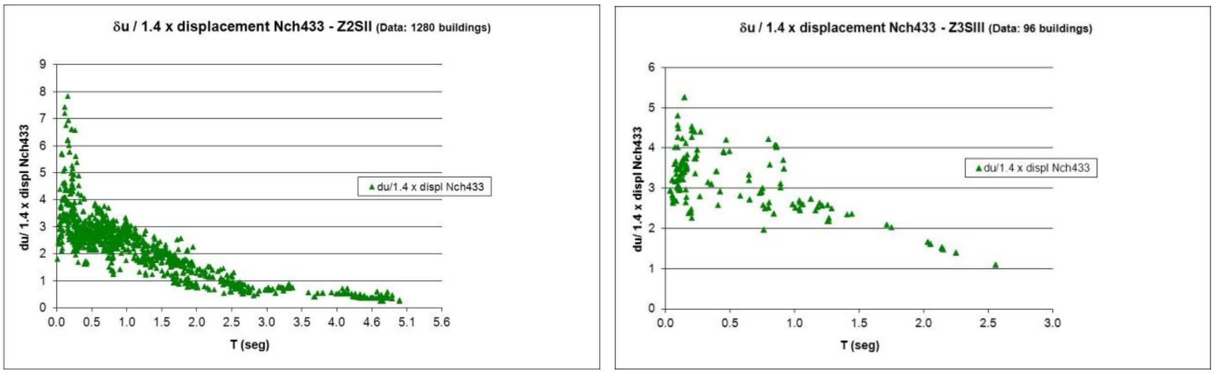

Fig. 10 Required Displacement Ductility Ratio Indicator μ_Δ* for 1280 buildings in zone 2, soil type II, and for 96 buildings in zone 3, soil type III (Database from René Lagos Engineers).

DAMAGE IN RC BUILDINGS IN CONCEPCION AND VIÑA DEL MAR AFTER THE 2010 EARTHQUAKE IN CHILE

The earthquake presented some unexpected characteristics such as the low frequency content, not seen before in Chile. This affected considerably the response of high rise buildings. Records obtained in Concepción by the University of Chile (Boroschek, 2010) show high destructive potential and are similar to others obtained in Viña del Mar and Santiago in soft soils (figure 10).

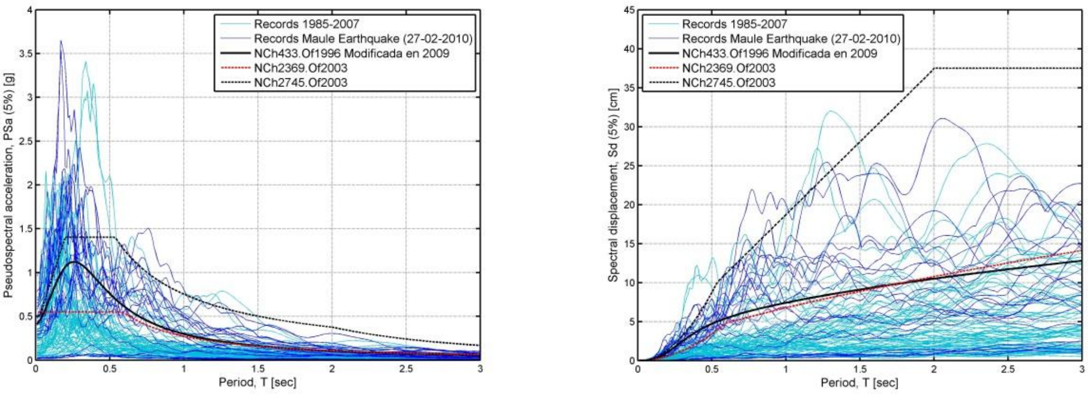

Fig. 10 Response spectra at 5% percent damping for records obtained in zone 3 and soil type II according to Chilean seismic codes. Elastic demands of NCh433, NCh2369 and NCh2745 are shown. A) Acceleration Spectra. B) Displacement Spectra.

The earthquake produced important structural damage in a number of high rise shear wall buildings in Santiago, Viña del Mar, Chillán and Concepción. Damage was concentrated at the base, on L or T shape walls with compression failures and buckling in vertical reinforcement at poorly detailed boundary elements. Shear failures at the web were also observed.

Figures 11 and 12 show typical plan configurations of damaged buildings (Bonelli, 2010). The architecture of this buildings present similar characteristics such as a central corridor with transverse walls of rectangular L or T shape. The walls are continuous from top to foundation. At street and underground parking levels the walls frequently present penetrations or length reduction to facilitate parking. This represents a common structural irregularity on modern high rise buildings.

 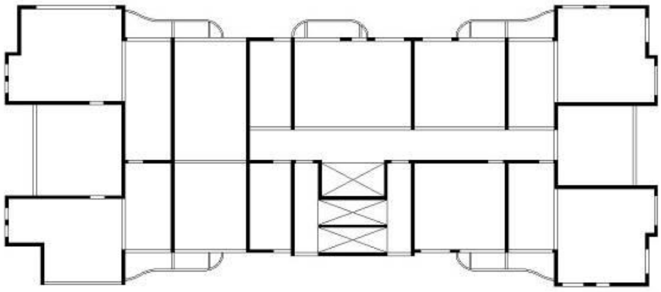

Fig. 11 Building B, Concepcion: Compression, tension and shear failure on first and second floors

 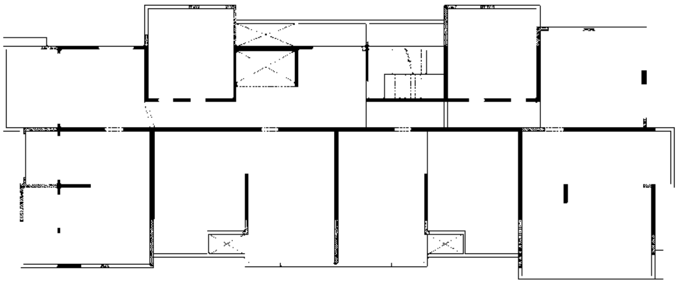

Fig. 12 Building C, Viña del Mar: general wall failure at first floor: 40 cm out of plumb at the top.

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 Design of Buildings.

NCh433 changes introduced by DS61 MINVU 2011 for the Seismic Design of Buildings:

- A new soil type classification is introduced defining soils types A, B, C, D and F, renaming soil type I as A, II as B, a new type C, III as D 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.
- 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 in real buildings under the February 27th 2010 earthquake. Conceptually this spectrum corresponds to a more severe earthquake than the assumed for the respective pseudo-acceleration spectrum in the code.
- 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. This value can be assumed as the roof displacement for the Deterministic Maximum Considered Earthquake (MCE).

NCh430 changes introduced by DS60 MINVU2011 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.

The use of capacity design has been incorporated to prevent shear failures.

The most important changes for the design for bending and axial load of shear walls are:

- The level of axial stress allowed:
  An explicit limit is established for the maximum value of: $P_u \leq 0.35f'_c$
  An implicit limit is established by requiring that the maximum concrete compression strains ε_c at the critic zone in walls must not exceed 0.008 when the Maximum Lateral Displacement at the roof δ_u occurs. This requirement also limits compression damage in the wall at the MCE.
- Slenderness: wall thickness must be greater than 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.

CONCLUSIONS

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 Maule earthquake.

The parameter H/T has a good correlation with the performance objectives defined as $\delta u/H$ according to SAOC VISION 2000. In buildings with values of H/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, 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 codes 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/T. This indirectly contributed to the successful performance of high-rise buildings observed during the 2010 earthquake.

The new provisions introduced in the Chilean Codes after the earthquake, represent an important step in the performance and the damage control objectives. These provisions are intended to prevent crushing and spalling of concrete and buckling of vertical reinforcement at boundary regions, and at the same time prevent shear failures when the buildings are subjected to large displacement demands.

REFERENCES

American Concrete Institute (2005), Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05), Farmington Hills, MI., 2005, 430pp.

American Society of Civil Engineers (2010). ASCE/SAI 7-10: Minimum Design Loads for Buildings and Other Structures, Reston, Va., 650pp.

Bonelli P., 2010. Evaluación de Daños y Normativa Terremoto en Chile Central-2010 (in Spanish), Presentación Congreso ACHISINA, Santiago Chile 2010.

Boroschek R., Soto P., and Leon R., 2010. Registros del Terremoto del Maule Mw=8.8 27 de Febrero de 2010, *Red Nacional de Acelerógrafos del Departamento de Ingeniería Civil,* Facultad de Ciencias y Matemáticas, Universidad de Chile, *Informe RENADIC 10/05*, 100pp. (p://www.terremotosuchile.cl/)

Calderón JA, 2007. Update on Structural System Characteristic used in RC Building Construction in Chile (in Spanish), *Civil Engineering Thesis*, University of Chile, 76 pp.

Comisión de Diseño Estructural en Hormigón Armado y Albañilería, Edificios Chilenos de Hormigón Armado, Cámara Chilena de la Construcción, pp. 117, June 2002

Comité Inmobiliario (real state committee). CChC, 2010. Communication based on INE data, Instituto Nacional de Estadisticas (National Institute of Statistics), http://www.ine.cl/

D.S. Nº 60 MINVU (2011), Reglamento que fija los requisitos de diseño y cálculo para el hormigón armado y deroga Decreto Nº 118, de 2010 (in Spanish), Ministerio de Vivienda y Urbanismo. Diario Oficial 13 de Diciembre del 2011

D.S. Nº 61 MINVU (2011), Reglamento que fija el diseño sísmico de edificios y deroga Decreto Nº 117, de 2010 (in Spanish), Ministerio de Vivienda y Urbanismo. Diario Oficial 13 de Diciembre del 2011

Gómez C.E., 2001. Structural System Characteristics used in RC and Reinforced Masonry Building Construction in Chile (in Spanish), *Civil Engineering Thesis*, University of Chile.

GUC seismological services at the University of Chile, http://www.sismologia.cl/

Instituto de la Construcción, Comisión Provisoria Terremoto 2010, 30 Propuestas relativas al terremoto 27-F 2010 (in Spanish), Anexo 2, March 2010

Guendelman T., Lindenberg J., Perfil bio-sísmico de edificios. Un instrumento de calificación sísmica (in Spanish), Revista BIT, v.7:n.17, pp.33, March 2000

Guendelman T., Lindenberg J., Cambio en Dolicitaciones Sísmicsd en Edificios, Seminario Efectos del Terremoto en el Nuevo Diseño Sísmico y Estructural en Chile, ICH Nov. 2010.

Massone L, Bonelli P, Lagos R, Lüder C, Wallace JW, 2011, RC Building Damage and Code Implications, EERI, Earthquake Spectra – Chile Special Issues.

Official Chilean Standard, NCh433.Of96, Earthquake Resistant Design of Buildings (in English), Instituto Nacional de Normalización, INN, 43pp, 1996

Official Chilean Standard, NCh430.Of2008, Reinforced Concrete – Design and calculation requirements (in Spanish), Instituto Nacional de Normalización, INN, 17pp., 2008

Official Chilean Standard, NCh2745.Of03, Analysis and Design of Buildings with Base Isolation (in Spanish)), Instituto Nacional de Normalización, INN, 98pp, 2003

Official Chilean Standard, NCh2369.Of03, Earthquake Resistant Design of Industrial Structures and Facilities (in Spanish)), Instituto Nacional de Normalización, INN, 120pp, 2003

Structural Engineers Association of California, Performance Based Seismic Engineering of Buildings VISION2000, 1995.