Exceptions to ACI 318-08 in the Americas

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Introduction

The Exceptions to ACI 318 in the Americas is intended to present in a easy to read way the major differences between the reinforced concrete design codes in use in the Americas and the most current version of the ACI 318 document.

The following member countries have submitted documents stating the major differences. Some editing is still needed in order to make the final version easier to read. In some cases no changes are made to the entire ACI 318 document which is accepted completely. However, not all member countries have updated their documents to the newer editions of ACI 318. Most of the submitted documents use ACI 318-02 and one has adopted ACI 318-05. This is quite an interesting finding since starting in 2005, ACI has published a Spanish version of the document. It is concluded that the Spanish version has not had an official impact in the code writing bodies in the Americas. Perhaps more important than this is the possibility that practicing engineers may be using the most current Spanish edition without paying too much attention to the possible gaps between the newer and the approved editions.

Enclosed are the contributions from the member countries listed below.

Argentina
Chile
Costa Rica
Mexico
Panama
Puerto Rico
Venezuela

Argentina

THE STRUCTURAL CONCRETE CODE IN ARGENTINA CIRSOC 201-2005. DIFFERENCES RESPECT ACI 318

Alberto Giovambattista

1 - INTRODUCTION [1]

1.1 - Regulation system for Civil Engineering Projects.

Today's regulation system for Civil Engineering Projects consists of Standards and Codes.

Codes establish general requirements and are of mandatory application. CIRSOC [2] reports to the national government and is responsible for writing and approving the Codes, which are then applied by the national, provincial and municipal governments.

The standards specify material characteristics. They are issued by IRAM [3], a private institution acknowledged by the national government, which runs like similar institutions in other countries.

The CIRSOC 201 Code lays down the requirements for the design and construction of concrete structures, and it also references IRAM standards applicable to materials.

1.2 - Brief history of Structural Concrete Codes in Argentina.

The first code dates from 1930. It was based on German standards.

PRAEH [4] was issued in 1964. Its first part dealt with ACI concrete technology. It included specifications on materials, strength properties, durability and construction processes contained in recommendations other than ACI 318. The second part specified requirements for structural design; it was based on the state of the art at that time and acknowledged the influence of CEB **[5].**

CIRSOC was created in 1979. The first version of CIRSOC 201 was approved in 1982 [6]. It is based on the German Code [7]. However, concrete technology continues to be ACI's and all PRAEH contents have been included in it.

In 1996 a second version of CIRSOC 201 was issued to be of use in Buenos Aires City. This version is similar to the previous one, but a new order is established in the chapters related to Materials and Constructions and a Classification of Exposure Environments based upon Eurocodes is added.

The present version of CIRSOC 201, issued as CIRSOC 201-2005 is based on ACI 318-05.

2 - DIFFERENCES BETWEEN ACI 318 AND CIRSOC 201-2005

2.1 - General considerations

When CIRSOC 201-05 was written, the same order as ACI 318-02 was followed for chapters from 7 to 22.

Chapters 1 to 6 were reordered to maintain the approach set by previous national codes. They include concrete technology specifications applicable to projects set in ACI recommendations other than ACI 318. These chapters are:

Chapter 1-General requirements and definitions;
Chapter 2-Concrete quality, durability and strength requirements;
Chapter 3-Materials;
Chapter 4-Assesment criteria;
Chapter 5-Proportioning, mixing, placing and curing;
Chapter 6-Formwork, embedded pipes and tolerances.

New chapters were also added

Chapter 23-Prestressed concrete. Injection of ducts.
Chapter 24- Finished Structures Approval.

2.2 – Main differences between ACI 318-08 and CIRSOC 201-05

Differences between both Codes are shown in Table A. Also, in the following sections main differences are detailed.

3 - DURABILITY.

3.1 - Durability Requirements

They are included in Chapter 4 of ACI 318-08 and in Chapter 2 of CIRSOC 201-2005. Both chapters are equivalent, with some differences.

CIRSOC 201-05 considers that the environment is an action that affects the structure. It also establishes that service life should be fixed by the owner and assumes it to be equal to or older than 50 years.

ACI 318-05 did not present any Exposure Categories or Classes; as a result, CIRSOC 201-05 adopted a classification based upon EU 2 and complementary standards. On later revisions, ACI 318-08 included a classification of Exposure Categories or Classes, which in some aspects is similar to that of CIRSOC, but also presents significant differences.

Within this picture, CIRSOC 201-05 durability design specific requirements are taken from ACI 318 and recommendations included in the ACI Manual of Concrete Practice, referred to in the Safety Code. Some of the important subjects treated here are: aggressive substances in water and soil in contact with the structure, sulphate contents in concrete, chloride contents in concrete, freezing and thawing resistance in concrete, alkali-silica reaction.

CIRSOC presents two tables to classify the natural environment at the sites taking into account whether they produce rebar corrosion or concrete degradation due to effects other than corrosion. Protection requirements are set for each environment. All these requirements are shown in Tables B, C and D.

In a separate section, Chapter 2 deals also with alkali-silica reaction (see section 4 of this presentation).

Table A. Main differences between ACI 318-08 and CIRSOC 201-05

Art.		Differences
ACI 318-08	CIRSOC 201-05	
3.1.1 a 3.1.2	---	Testing materials requirements are presented for each material individually in CIRSOC
3.2	3.1; 3.5	ACI specifications on cementitious materials are based on ASTM Standards. CIRSOC uses IRAM Standards which are based on European Community Standard EN 197-1
3.3	3.2	ACI refers to ASTM aggregates standards. CIRSOC refers to IRAM standards. Both standards are similar.
3.4	3.3	ACI refers to ASTM aggregates standards. CIRSOC refers to IRAM standards. They have minor differences.
3.6	3.4	CIRSOC include additive specifications with reference to IRAM Standards that are similar to ASTM C 494 and ASTM C 260
3.5	3.5	Steel reinforcement. The bars used in both codes are different in size and mechanical characteristics.
4.1 / 4.5	2.0 / 2.2.12.2	Durability requirements. Differences are detailed in section 3 of this presentation
5.1.6	---	Steel fiber reinforced concrete use is not considered by CIRSOC
5.3.2	5.2.2	In CIRSOC, f¨cr shall be determined according to different conditions at the concrete production site (see section 6 of this presentation), using the following expressions: a) Control Mode 1: $f'cr = f'c + 1{,}34\ sn$ $f'cr = f'c + 2{,}33\ sn - 3{,}5$ b) Control Mode 2: $f'cr = (f'c + 5) + 1{,}34\ sn$ $f'cr = f'c + 2{,}33\ sn$
5.6	Chapter 4	Concrete evaluation and acceptance. Two different criteria are introduced by CIRSOC in order to establish compliance with specified resistance. See further explanation in section 6 of this presentation
5.8 / 5.9 / 5.10	5.3 / 5.4 / 5.5 / 5.6 / 5.7	CIRSOC introduced the highlight issues of the Recommendation ACI 304R Guide for Measuring, Mixing, Transporting and Placing Concrete and ACI 309R Guide for Consolidation of Concrete.
5.11	5.10	CIRSOC included the highlight issues of the Recommendation ACI 308R Standard Practice for Curing Concrete.
5.12	5.11	CIRSOC included the highlight issues of the Recommendation ACI 306R Cold Weather Concreting
5.13	5.12	CIRSOC included the highlight issues of Recommendation ACI 305R Hot Weather Concreting
Chapter 6	Capítulo 6	CIRSOC included the highlight issues of the Recommendation ACI 347R Guide for Formwork for Concrete

Table B - Durability in CIRSOC
General exposure classes that lead to rebar corrosion
(CIRSOC 201-05; Table 2.1)

Type	Class		Description of the environment
A1	Non-aggressive		No condensation inside the building. Exterior elements protected. Structural massive concrete. Rural environments and desert climates, average annual rainfall below 250 mm.
A2	Normal environment. Moderate and cold temperature, no freezing. High and average humid conditions or cyclic wet and dry.		Interiors exposed to air, RH≥65%, or to condensation. Exteriors exposed to average annual rainfall < 1,000 mm. Elements buried in damp soils or submerged.
A3	Tropical and sub-tropical climates.		Exteriors exposed to average annual rainfall ≥ 1,000 mm. Average monthly annual temperature for over 6 months a year ≥ 25ºC.
CL	Humid or submerged, containing chlorides from sources other than marine environment		Concrete surfaces exposed to chlorides from water spray or due to fluctuations in water level. Concrete exposed to natural waters polluted by industrial drains.
M1	Marine	Outdoors	At more than 1 km from the high tide level and unusual contact with airborne salt.
M2		Outdoors	At less than 1 km from the high tide level and in continuous or frequent contact with airborne salt.
		Submerged	Submerged in seawater, below the minimum tide level.
M3		Submerged	In the zone of tide fluctuation or exposed to sea splash

Table C - Durability in CIRSOC
Specific exposure classes that can cause degradation
other than rebar corrosion
(CIRSOC 201-05; Table 2.2)

Type	Class	Subclass	Description of the environment
C 1	Freezing and Thawing	Without deicing salts	Elements commonly in contact with water or with an average ambient relative humidity above 75 % in winter, with more than 50% probability of reaching, at least once, temperatures below - 5 ºC.
C 2		With deicing salts	Structures for vehicles or pedestrians in areas with more than 5 snowfalls per year or with an average minimum temperature below 0 ºC in the winter months.
Q 1	Chemical environments	Moderately aggressive	Concrete alteration at low velocity.
Q 2		Highly aggressive	Concrete alteration at medium velocity. Seawater exposure.
Q 3		Extremely aggressive	Concrete alteration at high velocity.

Table D - CIRSOC durability requirements depending on exposure conditions
(CIRSOC 201-05; Table 2.5)

Requirements	Concrete	Types of exposure for structures									
		A 1	A 2	A 3 M 1	CL M 2	M 3	C 1 (2)	C 2 (2)	Q 1	Q 2	Q 3 (3)
Maximum wc ratio (1)	Plained	------	------	------	0.45	0.45	0.45	0.40	0.50	0.45	0.40
	Reinforced	0.60	0.50	0.50	0.45	0.40	0.45	0.40	0.50	0.45	0.40
	Prestressed	0.60	0.50	0.50	0.45	0.40	0.45	0.40	0.50	0.45	0.40
Minimum f´c (MPa)	Plained	------	------	------	30	35	30	35	30	35	40
	Reinforced	20	25	30	35	40	30	35	30	35	40
	Prestressed	25	30	35	40	45	30	35	35	40	45

(1) - If cement and cementitious materials are used, wc shall be replaced by [w(c+x)].
(2) - Concrete shall be air-entrained with the air content required by this Code.
(3) In addition, the structure shall be protected with an aggression-resisting waterproof membrane

Table E - Durability in CIRSOC
Limiting values for aggressive substances in contact waters
(CIRSOC 201-05; Table 2.3)

Degree of attack	Soluble sulphates (SO_4^{2-}) (1)	Magnesium (Mg^{2+}) (2)	pH (3)	Lime dissolution by attack with carbonic acid (CO_3^{2-}) (4)	Ammonium (NH_4^+) (5)
	mg/liter	mg/liter	-----	mg/liter	mg/liter
Moderate	150 a 1.500	300 a 1.000	6,5 a 5,5	15 a 40	15 a 30
Strong	1.500 a 10.000	1.000 a 3.000	5,5 a 4,5	40 a 100	30 a 60
Very strong	Greater than 10.000	Greater than 3.000	Less than 4,5	Greater than 100	Greater than 60
(1); (2); (3) y (5) IRAM 1872:2004. (4) IRAM 1708:1998.					

Criteria to determine the degree of attack of contact waters Table E.

- Table values apply to moderate climates, with average annual temperatures equal to or below 25 ºC, and still or slowly moving waters (velocity equal to or less than 0.8 m/s).
- If water contains a single aggressive substance, such substance determines the degree of attack.
- If water contains two (2) or more aggressive substances, the degree of attack will be determined by the most severe concentration of aggressive agents present in the water. If all the concentrations correspond to the same degree of attack, with values greater than 0.75 of the upper limit or 0.25 of the lower limit for acidity (pH), the next biggest degree of attack must be taken.

Table F - Durability in CIRSOC.
Limiting values for aggressive substances in contact soils
(CIRSOC 201-05; Table 2.4)

Degree of attack	Soluble sulphate (SO_4^{2-}) (1)	Acidity degree by Baumann – Gully Modified (2)
	% in mass	Nº
Moderate	0,10 a 0,20	Greater than 20
Strong	0,20 a 2,00	-----
Very Strong	Greater than 2,00	------
(1) IRAM 1873:2004; (2) IRAM 1707-1:1998		

Criteria to determine the degree of attack of contact soils using Table F.

- Limiting values for aggressive substances in soils are valid for structures that are frequently or continuously in contact with water-saturated soils.
- When the soils have low permeability, K less than 10^{-5} m/s, the degree of attack can be reduced to the next smaller degree.

4 – DURABILITY. ALKALI-SILICA REACTIONS IN CIRSOC 201-05.

The specifications for Alkali-Silica Reactions (ASR) follow PCA and ACPA recommendations [8] and consider local experience.

ASR is prevented by field performance data and laboratory tests.

When field data on existing structures that have no evidence of ASR and meet the following conditions are available:

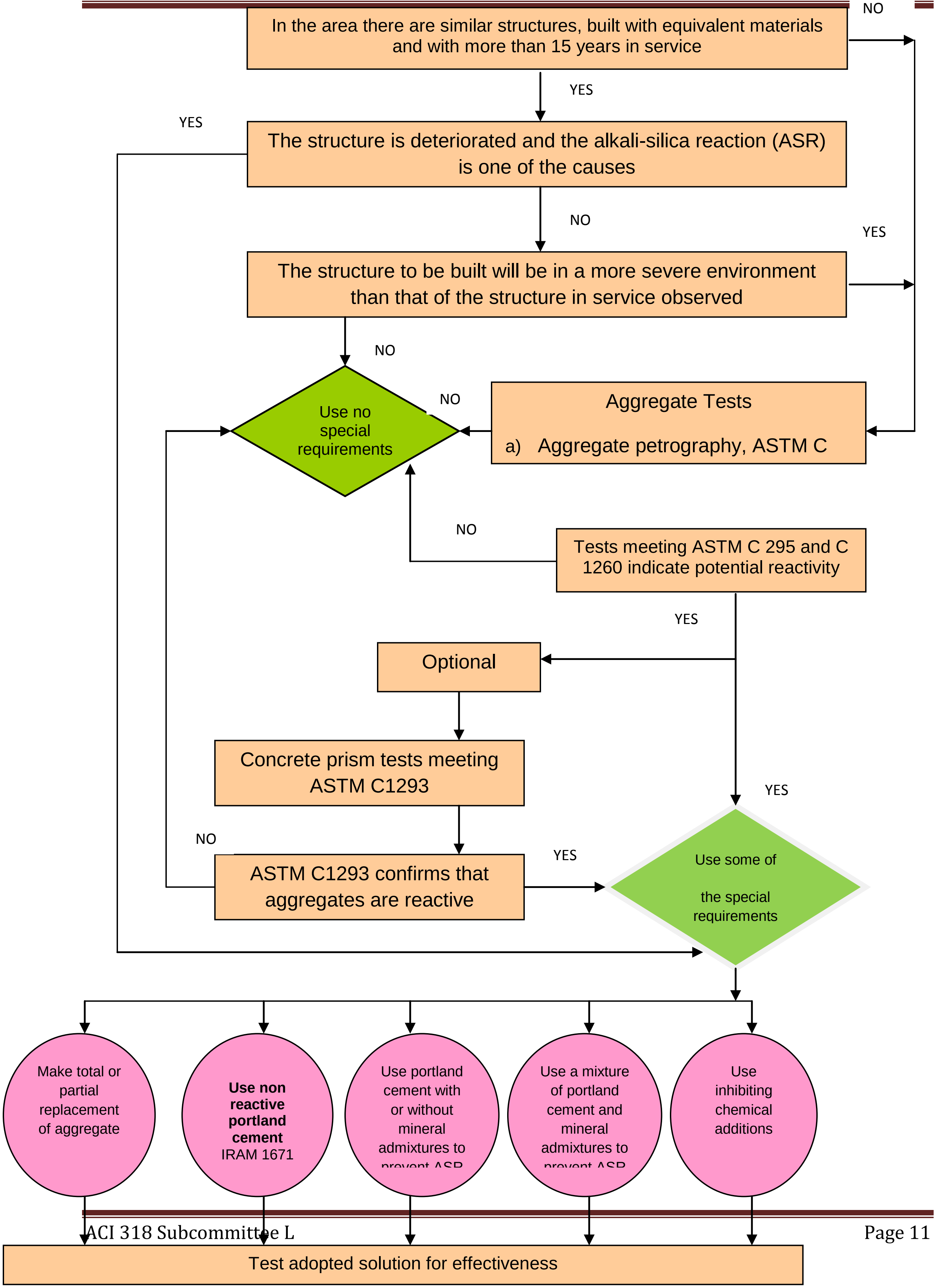
- they have been built with a set of materials (aggregates, cement, cementitious materials and mineral additions) similar to those intended to be used in the project under study
- they are of the same structural type,
- they are located in the same area,
- exposure conditions are similar and
- they have been in service for more than fifteen years,

the materials proposed shall be considered as not causing ASR.

When using aggregates with no field performance history or when their potential alkaly reactivity is uncertain, they will be assessed by the following methods:

- Petrography ASTM C 295 (IRAM 1649-68).
- Mortar-bar test ASTM C 1260 (IRAM 1674-97).
- Concrete-prism test ASTM C 1293 (IRAM 1700-97

A Flow chart is added to show how to use previous methods.


In the area there are similar structures, built with equivalent materials and with more than 15 years in service
NO
YES
The structure is deteriorated and the alkali-silica reaction (ASR) is one of the causes
YES
NO
The structure to be built will be in a more severe environment than that of the structure in service observed
YES
NO
Use no special requirements
NO
Aggregate Tests
a) Aggregate petrography, ASTM C
NO
Tests meeting ASTM C 295 and C 1260 indicate potential reactivity
YES
Optional
Concrete prism tests meeting ASTM C1293
YES
NO
ASTM C1293 confirms that aggregates are reactive
YES
Use some of the special requirements
Make total or partial replacement of aggregate
Use non reactive portland cement IRAM 1671
Use portland cement with or without mineral admixtures to
Use a mixture of portland cement and mineral admixtures to
Use inhibiting chemical additions
Test adopted solution for effectiveness

5 - CONFORMITY CRITERIA FOR STRENGTH.

CIRSOC strength-related conformity criteria follow modern quality concepts and seek to reduce consumer's risk.

CIRSOC has two types of conformity criteria which correspond to different conditions at the concrete production site. In both cases, concrete acceptance is based on tests performed on samples taken at the site.

Type 1 criteria are similar to those of ACI 318 and apply to manufacturers that have a quality system in the place of production.

Type 2 criteria apply to concrete supplied by a plant that has not implemented a quality system. According to these criteria there is a 20% probability of accepting a lot, subjected to 5 tests, from a population with a standard deviation of 5 MPa and where 20% of the concrete gives strength values below f´c.

In Type 2 criteria, the strength level is considered satisfactory if both of the following requirements are met:

a) Every arithmetic average of any three (3) consecutives strength test results is

$$f'_{cm3} \geq f'_{c} + 5\ MPa$$

b) Any individual test result (two cylinders on average) is

$$f'_{ci} \geq f'_{c}$$

6 - CHAPTER 23-PRESTRESSED CONCRETE. INJECTION OF DUCTS.

In our country, differences about methods and grouting for duct injections existed. CIRSOC 201.05 included this chapter to show how to do this task carefully and with detail.

7 - CHAPTER 24- APPROVAL OF FINISHED STRUCTURES.

This chapter was existent in previous versions of CIRSOC 201-05.

8 - REFERENCES

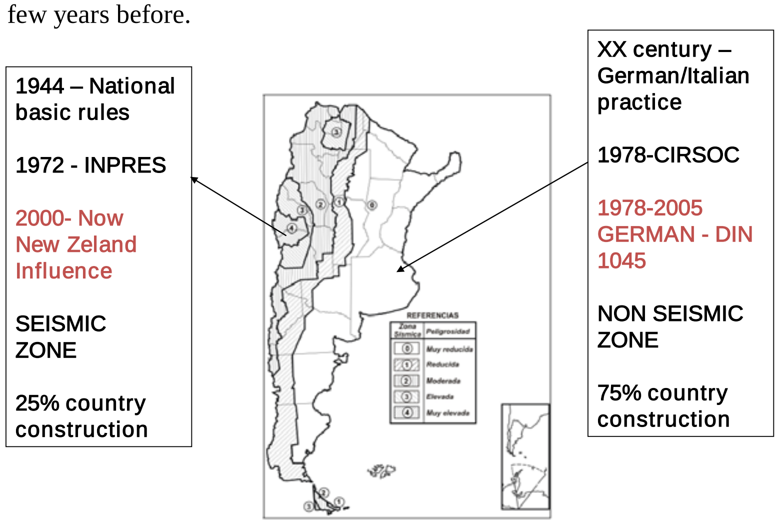
[1] Giovambattista, A; “Structural Concrete Code in Argentina”. Structural Concrete for the Americas, Workshop Sponsored by ACI-International, Phoenix, USA, October 25-26, 2002
[2] CIRSOC. Centro de Investigación de los Reglamentos Nacionales de Seguridad para las Obras Civiles. Instituto Nacional de Tecnología Industrial. Gobierno de la República Argentina.
[3] IRAM. Instituto Argentino de Normalización.
[4] Proyecto de Reglamento Argentino de Estructuras de Hormigón (P.R.A.E.H.), 1964.
[5] Comité Europeén du Beton. Bulletis d’Information Nos. 31, 35, 39.
[6] Proyecto de Reglamento CIRSOC 201. Proyecto, Cálculo y Ejecución de Estructuras de Hormigón Armado y Pretensado. (1980). CIRSOC, INTI, 459 p. July 1980.
[7] DIN 1045 -1978. Bestimnungen für Ausführung von Bauwerken aus Stahlbeton.
[8] Durability Subcommittee, Portland Cement Association. (1998). Guide Specification for Concrete Subject to Alkali-Silica Reactions. PCA Concrete Information, September 1998.

Comparison between the ACI 318-02 and the Argentine Code for the Design of Concrete Structures CIRSOC 201

Prepared by Raul D. Bertero for ACI Committee 318L. April 2010

1. General Comments related to the Argentine Code

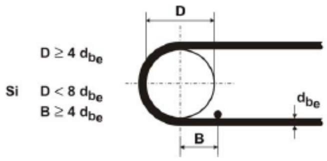
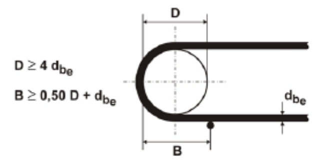
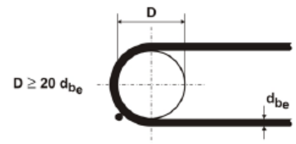
- Due to the strong influence of German and Italian engineers at the beginning of the XX century, Argentina used the German concrete code, DIN 1045, along that century.
- In the 90's it became clear that it was necessary to replace the old DIN 1045 by a code based on one of the two main world concrete code systems: Eurocode or ACI.
- After a long discussion, in 2002 the CIRSOC ("Centro de Investigación de los Reglamentos Nacionales de Seguridad para las Obras Civiles") adopted ACI 318 -2002 as the base code for the concrete building code in Argentina.
- However, since the seismic zones of the country are under regulation of other agency, the "Instituto Nacional de Prevención Sísmica – INPRES", the seismic code still remains using the New Zealand based approach that had been adopted few years before.



- A committee of experts was designated to adapt the ACI Code to the European construction practice prevalent in Argentina. The policy of the committee was to change as few items as possible. However, some changes had to be done mainly due to the European tradition of using smaller bar diameters (6 mm and 8 mm are common for ties and stirrups) and bar spacing.
- The structural segment of the code (Chapter 7 to Chapter 23) uses the same sections numbers that the ACI 318-2002. Chapter 21 does not exist and seismic regulations are covered by INPRES-CIRSOC 103 code.
- Other changes are related with the thickness cover since more detailed specifications are incorporated in the code for environmental and fire protection.

2. ACI 318 Changes to be used in Argentina

<table>
<tr><th>ACI 318</th><th>CIRSOC 201</th></tr>
<tr><td>7.2.1 — Diameter of bend measured on the inside of the bar, other than for stirrups and ties in sizes No. 3 (10mm) through No. 5 (16mm), shall not be less than the values in Table 7.2.

TABLE 7.2 — MINIMUM DIAMETERS OF BEND<table><tr><th>Bar size</th><th>Minimum diameter</th></tr><tr><td>No. 3 through No. 8</td><td>$6d_b$</td></tr><tr><td>No. 9, No. 10, and No. 11</td><td>$8d_b$</td></tr><tr><td>No. 14 and No. 18</td><td>$10d_b$</td></tr></table></td><td>Introduces a new requirement for longitudinal main reinforcement (7.2.2.) bend to act as frame joints reinforcement or shear reinforcement. In these cases the diameter of bend of Table 7.2 is increased 50 % .</td></tr>
<tr><td>7.2.3 — Inside diameter of bend in welded wire reinforcement for stirrups and ties shall not be less than 4db for deformed wire larger than D6 and 2db for all other wires. Bends with inside diameter of less than 8db shall not be less than 4db from nearest welded intersection.
</td><td>Introduces new requirements (7.2.4) regarding the distance from bends to nearest welded intersection when transversal bar is external. The requirements are illustrated in the Code with the following figures:

</td></tr>
<tr><td>7.6.5 — In walls and slabs other than concrete joist construction, primary flexural reinforcement shall not be spaced farther apart than three times the wall or slab thickness, nor farther apart than 450 mm</td><td>7.6.5 — In walls and slabs other than concrete joist construction, primary flexural reinforcement shall not be spaced farther apart than three times the wall or slab thickness, nor farther apart than 300 mm</td></tr>
<tr><td>7.7.1 – Concrete cover, mm
(a) Concrete cast against and permanently exposed to earth................................75
(b) Concrete exposed to earth or weather:
No. 19 through No. 57 bars................................50
No. 16 bar, W31 or D31 wire, and smaller........40
(c) Concrete not exposed to weather or in contact with ground:
Slabs, walls, joists:
No. 43 and No. 57 bars 40
No. 36 bar and smaller 20

Beams, columns:
Primary reinforcement, ties, stirrups, spirals..40
Shells, folded plate members:
No. 19 bar and larger 20
No. 16 bar, W31 or D31 wire, and smaller .....13</td><td>7.7.1 – Concrete cover, mm
(a) Concrete cast against and permanently exposed to earth................................50
(b) Concrete exposed to earth or weather:
>16 mm bars................................35
≤ 16 mm bars and wires…….......30
(c) Concrete not exposed to weather or in contact with ground:
Slabs, walls, joists:
> 32 mm bars 30
≤ 32 mm bars and wires......................................20
…………………………………….(≥ bar diameter)

Beams, columns:
Primary reinforcement….. ≥ bar diameter, but not less than 20 mm and need not exceed 40 mm

Ties, stirrups ……………………………….20
spirals... 40
Shells, folded plate members:
> 16 mm bars 20
≤ 16 mm bars and wires.......................... 15

Note: According to the ambient condition the values above are increased 0%, 30% or 50%</td></tr>
<tr><td>7.7.2 – Concrete cover, mm
(a) Concrete cast against and</td><td>7.7.2 – Concrete cover, mm
(a) Concrete cast against and</td></tr>
</table>

ACI 318	CIRSOC 201
permanently exposed to earth ....75 (b) Concrete exposed to earth or weather: Wall panels, slabs, joists....................................25 Other members..40 (c) Concrete not exposed to weather or in contact with ground: Slabs, walls, joists.. 20 Beams, columns: Primary reinforcement..................................40 Ties, stirrups, spirals .... 25 Shells, folded plate members: No. 5 bar, W31 or D31 wire, and smaller .... 10 Other reinforcement .... db but not less than 20	permanently exposed to earth ....50 (b) Concrete exposed to earth or weather: Wall panels, slabs, joists....................................25 Other members..30 (c) Concrete not exposed to weather or in contact with ground: Slabs, walls, joists.. 20 Beams, columns: Primary reinforcement..................................40 Ties, stirrups, spirals .... 25 Shells, folded plate members: > 16 mm bars .... 20 ≤ 16 mm bars and wires.......................... …. .15 Other reinforcement .... d_b but not less than 20 **Note: According to the ambient condition the values above are increased 0%, 30% or 50%**
7.7.3 – Concrete cover, mm (a) Concrete exposed to earth or weather: Wall panels: No. 14 and No. 18 bars, prestressing tendons larger than 1-1/2 in. diameter ....40 No. 11 bar and smaller, prestressing tendons 1-1/2 in. diameter and smaller, W31 and D31 wire and smaller...................... 20 Other members: No. 14 and No. 18 bars, prestressing tendons larger than 1-1/2 in. diameter ....50 No. 6 through No. 11 bars, prestressing tendons larger than 5/8 in. diameter through 1-1/2 in. diameter............................40 No. 5 bar and smaller, prestressing tendons 5/8 in. diameter and smaller, W31 and D31 wire, and smaller........................... 30 (b) Concrete not exposed to weather or in contact with ground: Slabs, walls, joists: No. 14 and No. 18 bars, prestressing tendons larger than 1-1/2 in. diameter .... 30 Prestressing tendons 1-1/2 in. diameter and smaller..20 No. 11 bar and smaller, W31 or D31 wire, and smaller... 16 Beams, columns: Primary reinforcement .... db but not less than 16 mm and need not exceed 40 mm Ties, stirrups, spirals ....10 Shells, folded plate members: Prestressing tendons.......................................20 No. 6 bar and larger .... 16 No. 5 bar and smaller, W31 or D31 wire, and smaller......................................10	**7.7.3** – Concrete cover, mm (a) Concrete exposed to earth or weather: Wall panels: Bars > 32mm and prestressing tendons larger than 40 mm.diameter ....40 Bars and wires ≤ 32 mm , prestressing tendons 40mm diameter and smaller.......................... 20 Other members: Bars > 32mm prestressing and tendons larger than 40 mm.diameter ....40 16mm < Bars and wires diameter ≤ 32 mm and 15 mm < prestressing tendons diameter ≤ 40mm diameter……………………………………..….30 Bars ≤ 16mm and prestressing tendons smaller than 15 mm.diameter .... 20 (b) Concrete not exposed to weather or in contact with ground: Slabs, walls, joists: Bars > 32mm prestressing and tendons larger than 40 mm.diameter........... 30 Prestressing tendons 40 mm diameter and smaller..20 Bars and wires ≤ 32mm................................. 15 Beams, columns: Primary reinforcement .... db but not less than 15 mm and need not exceed 40 mm Ties, stirrups, spirals ....20 Shells, folded plate members: Prestressing tendons.......................................20 Bars > 16 mm... 15 Bars and wires ≤ 16 mm..................................10 **Note: According to the ambient condition the values above are increased 0%, 30% or 50%**
7.7.5 — Corrosive environments In corrosive environments or other severe exposure conditions, amount of concrete protection shall be suitably increased, and the pertinent requirements for concrete based on applicable exposure categories in Chapter 4 shall be met, or other protection shall be	**7.7.5. Classes of environmental exposure** According to the enviromental exposure several classes are defined. The minimum concrete covers defined in 7.7.1., 7.7.2., 7.7.3. are established for classes A1 and A2. For classes A3, Q1 y C1 the values must be increased 30

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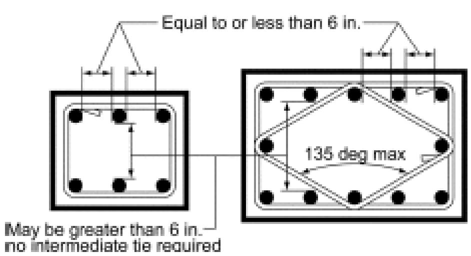
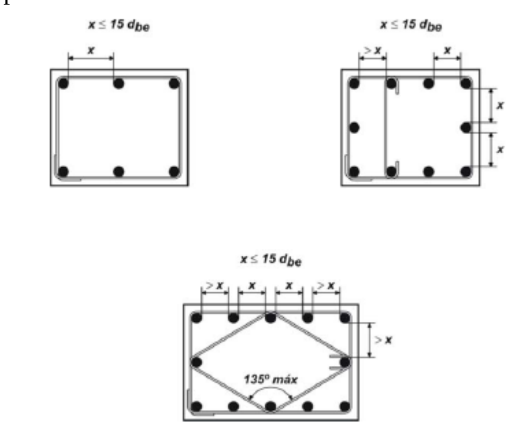
provided	% and for classes CL, M1, M2, M3, C2, Q2 y Q3 must be increased 50 % .
7.7.8 — Fire protection If the general building code (of which this Code forms a part) requires a thickness of cover for fire protection greater than the concrete cover in 7.7.1 through 7.7.7, such greater thicknesses shall be specified**.**	**7.7.8 —** Fire protection Several tables are introduced with the minimum cover thickness to satisfy fire resistance from 1 hr to 4 hr for slabs, walls, beams (prestressed and nonprestressed) and columns.
7.10.5.1 — All nonprestressed bars shall be enclosed by lateral ties, at least 10 mm in size for longitudinal bars 32 mm or smaller, and at least 13 mm in size for 36 mm and larger and bundled longitudinal bars. Deformed wire or welded wire reinforcement of equivalent area shall be permitted.	**7.10.5.1 —** **Longitudinal bars, d_b** — **Minimum diameter lateral ties, d_{be} (mm)** $d_b \leq 16$ mm — 6 16 mm < $d_b \leq 25$ mm — 8 25 mm < $d_b \leq 32$ mm — 10 $d_b > 32$ mm, bundled long.bars — 12
7.10.5.2 — Vertical spacing of ties shall not exceed **16 longitudinal bar diameters**, 48 tie bar or wire diameters, or least dimension of the compression member.	**7.10.5.2 —** Vertical spacing of ties shall not exceed **12 longitudinal bar diameters**, 48 tie bar or wire diameters, or least dimension of the compression member.
7.10.5.3 — Ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees and no bar shall be farther than **150 mm** clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie shall be permitted. 	**7.10.5.3** — Ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees and no bar shall be farther than **15 d_{be}** (15 times the tie bar diameter) clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie shall be permitted. 
7.12.2.2 — Shrinkage and temperature reinforcement shall be spaced not farther apart than **five times** the slab thickness, nor farther apart than **450 mm**	**7.12.2.2** — Shrinkage and temperature reinforcement shall be spaced not farther apart than **three times** the slab thickness, nor farther apart than **300 mm**
9.2.1 — Required strength U shall be at least equal to the effects of factored loads in Eq. (9-1) through (9-7). The effect of one or more loads not acting simultaneously shall be investigated.	**9.2.1 —** Required strength U shall be at least equal to the effects of factored loads in following equations. The effect of one or more loads not acting simultaneously shall be investigated.

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$U = 1.4(D+F)$ (9-1) $U = 1.2(D+F+T) + 1.6(L+H) + 0.5(L_r \text{ ó } S \text{ ó } R)$ (9-2) $U = 1.2D + 1.6(L_r \text{ ó } S \text{ ó } R) + (1.0L \text{ ó } 0.8W)$ (9-3) $U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ ó } S \text{ ó } R)$ (9-4) $U = 1.2D + 1.0E + 1.0L + 0.2S$ (9-5) $U = 0.9D + 1.6W + 1.6H$ (9-6) $U = 0.9D + 1.0E + 1.6H$ (9-7)	$U = 1.4(D+F)$ (9-1) $U = 1.2(D+F+T) + 1.6(L+H) + 0.5(L_r \text{ or } S \text{ or } R)$ (9-2) $U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W)$ (9-3) $U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R)$ (9-4) $U = 0.9D + 1.6W + 1.6H$ (9-6) **The seismic load combinations are not in this Code. The INPRES/CIRSOC 103 argentine seismic code has to be used.**
9.3.4 — For structures that rely on special moment frames, or special structural walls to resist earthquake effects, E, φ shall be modified as given in (a) through (c):	**9.3.4 —** For load combination including seismic load φ values are not in this Code. T**he INPRES/CIRSOC 103 argentine seismic code has to be used.**

9.5.3.4 — Slab thickness less than the minimum required by 9.5.3.1, 9.5.3.2, and 9.5.3.3 shall be permitted where computed deflections do not exceed the limits of Table 9.5(b). Deflections shall be computed taking into account size and shape of the panel, conditions of support, and nature of restraints at the panel edges. The modulus of elasticity of concrete, E_c, shall be as specified in 8.5.1. The effective moment of inertia, I_e, shall be that given by Eq. (9-8); other values shall be permitted to be used if they result in computed deflections in reasonable agreement with results of comprehensive tests. Additional long-term deflection	**9.5.3.4 —** A Table with the minimum thickness of slabs with interior beams is added to the code.
	
	Using:
	
	α_f = ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panels (if any) on each side of the beam
	
	β = ratio of long to short dimensions: clear spans for two-way slabs,
	
	Slabs with interior beams — αf / β / Min h:
	≤ 0,2 / ≤ 2 / ln / 33
	1 / 1 / ln / 36
	  / 2 / ln / 40
	≥ 2 / 1 / ln / 41
	  / 2 / ln / 49

10.5.4 - For structural slabs and footings of uniform thickness, $A_{s,min}$ in the direction of the span shall be the same as that required by 7.12.2.1. Maximum spacing of this reinforcement shall not exceed **three times** the thickness, nor **450 mm**	**10.5.4** - For structural slabs and footings of uniform thickness, $A_{s,min}$ in the direction of the span shall be the same as that required by 7.12.2.1. Maximum spacing of this reinforcement shall not exceed **2.5 times** the thickness, nor **300 mm**, nor **25 times the smaller bar diameter**
11.5.5.1 — Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed d/2 in nonprestressed members or 0.75h in prestressed members, nor **600 mm.**	**11.5.5.1 —** Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed d/2 in nonprestressed members or 0.75h in prestressed members, nor **400 mm**
11.10.9.3 — Spacing of horizontal shear reinforcement shall not exceed the smallest of lw/5, 3h, and **450 mm**, where lw is the overall length of the wall.	**11.10.9.3 —** Spacing of horizontal shear reinforcement shall not exceed the smallest of lw/5, 3h, and **300 mm**., where lw is the overall length of the wall.
11.10.9.5 — Spacing of vertical shear reinforcement shall not exceed the smallest of $l_w/3$, 3h, and **450 mm**., where lw is the overall length of the wall.	**11.10.9.5 —** Spacing of vertical shear reinforcement shall not exceed the smallest of lw/3, 3h, and **300 mm**., where l_w is the overall length of the wall.
14.3.5 — Vertical and horizontal reinforcement shall not be spaced farther apart than three times the wall	**14.3.5 —** Vertical and horizontal reinforcement shall not be spaced farther apart than three times the wall

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thickness, nor farther apart than **450 mm**	thickness, nor farther apart than **300 mm**
CHAPTER 21- EARTHQUAKE-RESISTANT STRUCTURES	**CHAPTER 21- EARTHQUAKE-RESISTANT STRUCTURES** This Chapter is not in the CIRSOC 201 Code. The Code drives to INPRES- CIRSOC 103, the Argentine Seismic Code based on New Zealand and previous Argentine Codes.

Chile

APPENDIX F

ACI 318-02 CODE ADJUSTMENTS TO BE USED IN CHILE

F.1- Reinforced concrete elements and structures must be designed and built according to ACI 318-02 Code requirements, using the modifications indicated in sections F.2 to F.30 of this Appendix.

This code replaces the use of the Chilean Standards: NCh 429. Of1957 and NCh 430. Of1961.

> ***RF.1-*** *Standards NCh 429 Of. 1957 and NCh 430 Of. 1961 are old and elemental for designing reinforced concrete elements. These standards are no longer in use (designs considering permissible stresses) and they have been improved by more complete and modern standards like the ACI 318 – 02 Code, that relies on a wide experimental background, an extended use of international level and it is bringing up to date continuously. This Appendix modifies the ACI 318 Code and presents some adjustments to the Chilean environment and special characteristics.*

F.2- ACI 318, section 1.1.3: add the following paragraph:

Chilean standards govern over other standards referenced in the ACI 318-02 Code, unless otherwise specifically indicated in this appendix.

> ***RF.2-*** *In general, the designer should use the most restricting Chilean Standards and those that correspond to the ones mentioned in the ACI 318 Code.*

F.3- ACI 318, section R1.1.3: add the following paragraph:

The ACI 318 Code forms a unit, so the requirements mentioned in it cannot be partially used, nor used in conditions different from those indicated in the code, unless a research or a requirement referenced in the appendix specifies something different.

F4- ACI 318, section R2.1: add the following paragraph to the “Load, Dead” definition:

The corresponding Chilean Standard is NCh1537 “Diseño estructural de edificios - Cargas permanentes y sobrecargas de uso” (Structural Design for Buildings – Dead Loads and Overload due to Use).
F.5- ACI 318, section R3.2: add the following paragraph:

The corresponding Chilean Standard is NCh148 “Cemento – Terminología, clasificación y especificaciones generales” (Cement – Terminology, Classification and General Specifications).

F.6- ACI 318, section R3.3: add the following paragraph:

The corresponding Chilean Standards are NCh163 “Áridos para morteros y hormigones – Requisitos generales” (Aggregates for Mortars and Concrete) and NCh170 “Hormigón – Requisitos generales” (Concrete – General Requirements).
F.7- ACI 318, section R3.4: add the following paragraph:

The corresponding Chilean Standard is NCh1498 "Hormigón – Agua de amasado – Requisitos" (Concrete – Batch Water – Requirements).

F.8- ACI 318, section R3.5.3: add the following paragraph:

The corresponding Chilean Standard is NCh204 "Acero – Barras laminadas en caliente para hormigón armado" (Steel – Hot-Rolled Bars for Reinforced Concrete).

> ***RF.8-*** *NCh 204. Of1977 establishes that the quotient f_u/f_y must be > to 1,33 compared to 1,25 required by the ASTM A 706M Standard. This greater hardening grade required in Chile improves reinforced concrete elements ductility and must be maintened if it does not mean a great increase on steel cost. However, NCh 204 Standard does not fix the yield strength upper limit. So it is neccesary to respect maximum values for the yield strengths established in the ASTM A 706M.*

F.9- ACI 318, section R3.6: add the following paragraph:

The corresponding Chilean Standard is NCh2182 "Hormigón y mortero – Aditivos – Clasificación y requisitos" (Concrete and Mortar – Additives – Classification and Requirements).

F.10- ACI 318, section 3.8.1: add the following Chilean Standards:

NCh148.Of1968	"Cemento – Terminología, clasificación y especificaciones generales" (Cement – Terminology, Classification and General Specifications).
NCh163.Of1979	"Áridos para morteros y hormigones – Requisitos generales" (Aggregates for Mortars and Concrete)
NCh170.Of1985	"Hormigón – Requisitos generales" (Concrete – General Requirements).
NCh204.Of1977	"Acero – Barras laminadas en caliente para hormigón armado" (Steel – Hot-Rolled Bars for Reinforced Concrete).
NCh433.Of1996	"Diseño sísmico de edificios" (Seismic Design for Buildings).
NCh1498.Of1982	"Hormigón – Agua de amasado – Requisitos" (Concrete – Batch Water – Requirements).
NCh1537.Of1986	"Diseño estructural de edificios - Cargas permanentes y sobrecargas de uso" (Structural Design for Buildings – Dead Loads and Overload due to Use).
NCh1934.Of1992	"Hormigón preparado en central hormigonera" (Concrete Prepared at a Mixing Plant).
NCh2182.Of1995	"Hormigón y mortero – Aditivos – Clasificación y requisitos" (Concrete and Mortar – Additives – Classification and Requirements).

F.11- ACI 318, section R4: add the following paragraph:

According to the subjects described in this Chapter, the corresponding Chilean Standards are NCh170 "Hormigón – Requisitos generales" (Concrete – General Requirements) and NCh163 "Áridos para morteros y hormigones – Requisitos generales" (Aggregates for Mortars and Concrete).

F.12- ACI 318, section 5.1.2: add the following paragraph:

Alternately, cube tests may be used, however the design equations must consider the f_c' value given by the following equivalence table established by the NCh170.Of1985 Standard:

f_c' (Mpa)	Concrete class (NCh170 with 10% of defective fraction)
16	H20
20	H25
25	H30
30	H35
35	H40

RF.12*- The following mathematic expression associates f_c' strain with Concrete Class (R_{28}) :*

$f_c' = 0{,}8\ R_{28}$ for $R_{28} \leq 25$ *MPa* *and*
$f_c' = R_{28} - 5$ for $R_{28} \geq 25$ *MPa*

F.13- ACI 318, section R5.8.2: add the following paragraph:

The corresponding Chilean Standard is NCh1934 "Hormigón preparado en central hormigonera" (Concrete Prepared at a Mixing Plant).

F.14- ACI 318, section 7.6.6: add the following paragraph:

7.6.6.6 - Bundled bars must not be used in elements where steel may pass to a plastic range or where it may undergo irreversible seismic stresses (tension – compression).

RF.14*- Bars that form the bundle are very exposed to buckle once the cover fails (spallings).*

F.15- ACI 318, section R7.7: add the following remark:

There are three reasons for requiring the minimum coverings:

1.- The strain transfer from the reinforcing bar towards the concrete.
2.- Reinforcing bars protection from corrosion.
3.- Reinforcing protection against fire effects.

Considering this three effects, different coverings from those required by the ACI 318 Code have been defined.

The following environmental and concrete cast conditions must be considered:

a.- Severe environmental conditions:

(i) Interior building zones with high humidity (industrial kitchens, saunas, laundries; do not apply to housing).
(ii) Water draining zones (flower stands or boxes, balconies).
(iii) Industrial or sea atmospheric conditions.

b.- Normal environmental conditions:

Conditions not included in the severe condition category, unless the experience indicates that special protection measures are required.

In case of severe environmental conditions, excepting some of them, the covers mentioned in the main body of ACI 318 have been maintained. In case of normal conditions, smaller covers are proposed.

Concrete casting conditions. There are three casting conditions:

(i) Cast in place of non prestressed elements.
(ii) Cast in place of prestressed elements.
(iii) Cast in plant (pre-cast elements).

F.16- ACI 318, section 7.7.1: replace all section by the following:

7.7.1- Cast-in-place concrete (non prestressed)

The following minimum concrete cover shall be provided for reinforcement, but not be less than required by sections 7.7.5 and 7.7.7:

	Minimum cover, mm Normal conditions	Severe conditions
(a) Concrete cast against earth and permanently exposed to earth ………….......	50	70
(b) Concrete exposed to earth or weather:		
Bars ø18 through ø56	40	50
Bars ø16 and smaller ……..	30	40
(c) Concrete not exposed to weather or in contact with ground:		
Slabs, walls, joists:		
Bars ø44 and ø56	40	40
Bars ø16 to ø36	20	20
Bars ø12 and smaller.......	15	20
Beams, columns:		
Primary reinforcement	30	40
ties, stirrups, spirals......	20	30
Shells and folded plates members:		
Bars ø18 and bigger.........	20	20
Bars ø16, 16 mm diameter wires and smaller ……...…	15	15
(d) Masonry confinement elements:		
Primary reinforcement ø10 and smaller	20	30
Ties, stirrups, spirals ø8 and smaller	15	20

F.17- ACI 318, section 7.7.2: replace all section by the following:

7.7.2- Cast-in-place concrete (prestressed)

The following minimum concrete cover shall be provided for prestressed and non prestressed reinforcement, ducts and end fittings, but shall not be less than required by sections 7.7.5, 7.7.5.1 and 7.7.7.

	Minimum cover, mm Normal conditions	Severe conditions
(a) Concrete placed against and permanently exposed to earth	60	70
(b) Concrete exposed to earth or weather:		
Walls panels, slabs and joists	25	25
Other members	40	40
(c) Concrete not exposed to weather or in contact with ground:		
Slabs, walls, joists	20	20
Beams, columns:		
Primary reinforcement	30	40
Ties, stirrups, spirals ……	20	25
Shells and folded plates members:		
Bars ø16, 16 mm diameter wires and smaller…	10	10
Other reinforcements .... ***db*** but not less than…	20	20

F.18- ACI 318, section 7.7.3: replace all section by the following:

7.7.3- Precast concrete (manufactured under plant control conditions)

The following minimum concrete cover shall be provided for prestressed and non prestressed reinforcement, ducts and end fittings, but shall not be less than required by sections 7.7.5, 7.7.5.1 and 7.7.7.

	Minimum cover, mm Normal conditions	Severe conditions
(a) Concrete exposed to earth or weather		
Wall panels:		
Bars ø44 and ø56	40	40
Bars ø36 and smaller.......	20	20
Other membres:		
Bars ø44 and ø56	50	50
Bars ø18 to 36	30	40
Bars ø16, 16 mm diameter wires and smaller …..	20	30

(b) Concrete not exposed to weather or in contact with ground:

Slabs, walls, joists:		
Bars ø44 and ø56	30	30
Bars ø36 and smaller.......	15	15
Beams, columns:		
Primary reinforcement ..... . ***db*** but not less than …	15	15
and not bigger than …..	40	40
Ties, abutments, hoop reinforcements	10	10
Shells and folded plates:		
Bars ø18 and bigger.........	15	15
Bars ø16, 16 mm diameter wires and smaller ….	10	10

F.19- ACI 318, section 9.1.3: add the following remark:

Use of load factors combinations and strength reduction factors mentioned in section 9.2.1, load combination, in cases that include earthquake effects, shall be done replacing the earthquake load factor 1.0 by the 1.4 factor, according to 9.2.1 (c), where earthquake E requirement shall be determined according to the NCh433 Standard Seismic Design for Buildings.

F.20- ACI 318, section R9.5.2.1: add the following remark:

The limits stipulated in Table 9.5(a) may be insufficient to control deformations, especially for cantilevers.

F.21- ACI 318, section 10.5.3: add the following remark:

This requirement is not applicable to flexural members. The minimum amount mentioned in 10.5.1 and 10.5.2 must be provided.

> ***RF.21****- To obtain the flexural performing, it is important that the yielding force of the reinforcement in tension be greater than the concrete area resistance exposed to tension, the same required by 10.5.1 and 10.5.2.*

F.22- ACI 318, section 21.2.1.1: add the following:

Not necessarily columns and wall chains of confined masonry shall fulfill the requirements mentioned in Chapter 21.

> ***RF.22-*** *Columns and wall chains of confined masonry are members of a composite system that work connected with the internal masonry panel. They do no act as frame members; their function is to confine the panel. The details of these elements must be designed according to the NCh 2123 Standard.*

F.23- ACI 318, section 21.2.1.2: add the following paragraph:

For the seismic design of reinforced concrete elements, all regions of the Chilean territory must be considered as high seismic risk.

> ***RF.23-*** *To resist to the forces induced by earthquakes, the structural systems mentioned in NCh433 Standard on seismic design for buildings or the NCh2369 Standard on seismic design for industrial buildings, shall be used.*

F.24- ACI 318, section 21.2.1.4: add the following paragraphs:

Frame structured buildings shall comply with the requirements mentioned in Chapter 21.

In composite structures with continuous walls and frames in all building depth, where walls extend at least a 5% of the shear in the bottom, or when frames will be able to reach the corresponding displacement that is obtained with the elastic spectrum of NCh433 Standard without applying the response reduction factors, the frames can be designed according to section 21.12 of this Chapter.

> ***RF.24-*** *For reinforced concrete buildings with walls in the two main analysis directions, it is assumed that the displacement requirements will be less than those for a structure without walls. Frames present in those composite structural systems would have low flexural requirements. Chapters 1 to 18 and Chapter 22 provide the appropriate toughness to structures not exposed to severe seismic movements, mentioned as normal in the ACI 318, and it is not necessary to applied the requirements of Chapter 21. However, it is advisable to give the appropriate details in order to answer to unexpected displacements, which requirements are mentioned in section 21.12.*

> *The answer displacement of the structure to the seismic design contained in the Standard is a little greater than R times the displacement calculated with the reduced spectrum. When applying the displacement requirements it is necessary to know the actual displacement of the expected answer. A frame that is able to answer elastically, inside the system, to a displacement equal to the calculated with the elastic spectrum, without applying the reduction factor R, will support greater displacements without developing great ductibility, and it shall be suitable to apply the recommendations for intermediate frames of this chapter, as they give some ductibility.*

F.25- ACI 318, section 21.2.5: replace the original text of letter (b) by:

(b) The ratio between ultimate actual tensile strength to the actual yielding strenght must not be less than 1.33, according to NCh204.

F.26- ACI 318, section 21.2.5: add the following paragraph after letter (b):

AT56-50 Steel or other that does not fulfill the requirements of section 21.2.5 (b) shall be used provided that:

i) It is not used in critical zones where steel may be induced to plastic range.
ii) It is not used as shear mesh in walls, unless the design is made for a capacity that assures that the element performance will not be controlled by a failure shear.

RF.26- *It is expected that the reinforcement yields only in critical zones leading to great deformations. It has been observed that in wall boundaries the reinforcement has been broken (failure). For this reason, there are some requirements for a minimum amount of reinforcement and for confinement reinforcement to avoid failure of the reinforcement. So it is necessary to use high toughness steel in those areas. However, as the other reinforcements rest in the lineal range, it is possible to use steel of less toughness.*

F.27- ACI 318, section 21.2.7.1: replace the original text by:

Welded splices are not permitted for member reinforcements in zones where the reinforcement is likely to occur inelastic displacements.

RF.27- *Bars welding is related with embrittlement failures. The arguments that justify this article recommendations are expressed in Commentaries of the article R26.*

F.28- ACI 318, section 21.4.2.2: add the following paragraph:

Upper floor ceiling connections of frame buildings shall not conform the requirements of Eq.(21-1).

RF28- *The presence of a hinge joint at the upper end of a column at the last floor does not risk building a weak floor. It is different that the hinge joist is formed in the column or in the ceiling girder of the last floor. Some engineers prefer that the hinge joist be in the column because they are afraid of alter the resistance capacity to axial loads. However, it is important to keep in mind that the theory allows joisting at the columns base, where demands for the axial loads resistances are greater.*

.

F.29- ACI 318, section 21.7.2.2: replace the original text by:

All walls resisting earthquake loads shall be built using two layers of reinforcement.

RF.29- *It exists a general consent that using two reinforcement layers assures the stability out of the plane and presents some practical advantages like concrete casting in place and a well confinement of the concrete center inside the wall panel zone, even though it is possible to use one layer.*

F.30 - ACI 318, section 21.7.6.2: add the following paragraph to letter (a):

In Eq. (21-8), δ_u shall be used for the corresponding displacement obtained with the elastic spectrum of NCh433 Standard, without applying answer reduction factors, amplifying by 1.3.

RF.30- *The maximum lateral displacement in the building ceiling shall be calculated from the displacement spectrums, but it is necessary to correct the calculated value for the fundamental period. In a system of different degrees of freedom, the upper modes contribute to the lateral displacement according to the corresponding participation factor. In general, the participation factor of the first mode of a wall building shall be of 85%. Moreover it is necessary to consider that the response in front of a severe earthquake shall be in a non lineal system. Different analysis made on reinforced concrete buildings shall be adopted the factor 1.3 as a statistic value.*

Mexico

Comparison between the ACI 318-08 and the Mexico City Building Code (MCBC) 2004 for the Design of Concrete Structures

Prepared by Mario E. Rodriguez for ACI Committee 318L. May 2010

(In metric units otherwise as indicated)

1. **Specified compressive strength of concrete**

   For the MCBC the specified compressive strength of concrete f_c^* is

$$f_c^* = 0.8\, f_c^{'}$$

2. **Types of concrete**
   Two types of concrete are specified by the MCBC: class 1 and class 2. The former has a $f_c^{'}$ value equal or larger than 25 MPa and is produced with a coarse aggregate having a unit weight equal or larger than 2.6 ton/m3. The specified concrete strength f_c' of concrete class 2 is smaller than 25 MPa and larger than 20 MPa.

3. **Modulus of rupture**

   ACI section 9.5.2.3: $f_r = 0.62\, \lambda\sqrt{f_c'}$

   MCBC section 1.5.1.3:

   -concrete class 1 $f_r = 0.47\,\sqrt{f_c'}$
   -concrete class 2 $f_r = 0.38\sqrt{f_c'}$

4. **Modulus of elasticity**

   - ACI section 8.5.1: for normal weight concrete $E_c = 4700\,\sqrt{f_c'}$

   - MCBC section 1.5.1.4, for concrete class 1 $E_c = 4400\,\sqrt{f_c'}$ and $E_c = 3500\,\sqrt{f_c'}$, depending of the type of coarse aggregate

     For concrete class 2 $E_c = 2500\,\sqrt{f_c'}$

5. **Shrinkage and temperature reinforcement**

ACI section 7.12 (in psi)

(a) Slabs where Grade 40 or 50 deformed bars are used 0.0020

(b) Slabs where Grade 60 deformed bars or welded wire reinforcement are used..................................0.0018

(c) Slabs where reinforcement with yield stress exceeding 60,000 psi measured at a yield strain of 0.35 percent is used .... $\frac{0.0018 \times 60,000}{f_y}$

MCBC section 5.7

The area of shrinkage and temperature reinforcement shall be at least equal to

$$A_s = \frac{660\ x_1}{f_y\ (x_1\ + 1000)}$$

where A_s is the area of reinforcement per unit of width (mm^2/mm) and x_1 is the width of member perpendicular to shrinkage and temperature reinforcement.

In liu of the above expression the ratio of A_s to *bd* is specified as 0.002 for elements in not severe exposures otherwise 0.003.

6. Shear strength provided by concrete for nonprestressed members

ACI section 11.2.2 (in psi)

11.2.2 — V_c shall be permitted to be computed by the more detailed calculation of 11.2.2.1 through 11.2.2.3.

11.2.2.1 — For members subject to shear and flexure only,

$$V_c = \left(1.9\lambda\sqrt{f_c'} + 2500\rho_w \frac{V_u d}{M_u}\right) b_w d \qquad (11\text{-}5)$$

but not greater than $3.5\lambda\sqrt{f_c'}\,b_w d$. When computing V_c by Eq. (11-5), $V_u d/M_u$ shall not be taken greater than 1.0, where M_u occurs simultaneously with V_u at section considered.

MCBC section 2.5.1

For beams with *M/(V d)* not smaller than 5

if $\rho_w < 0.015$ $\quad V_{cR} = 0.3\ F_R bd\ (0.2 + 20\rho_w)\sqrt{f_c^*}$ (2.19)

if $\rho_w \geq 0.015$ $\quad V_{cR} = 0.16\ F_R bd\ \sqrt{f_c^*}$ (2.20) where F_R is the strength reduction factor

if *M/(V d)* is smaller than 4, V_{cR} is obtained with Eq 2.20 multiplied by

$$3.5 - 2.5\frac{M}{V\,d} > 1.0$$

where $V_{cR} \leq 0.47 F_R bd\sqrt{f_c^*}$

For values *M/(V d)* between 4 and 5, V_{cR} varies linearly using Eqs 2.19 and 2.20.

7. **Earthquake resistant structures. General Requirements**

ACI Section 21.1.1.6. Structures assigned to SDC D, E, or F shall satisfy 21.1.2 through 21.1.8 and 21.11 through 21.13

Commentary R21.1.1

> Structures assigned to SDC D, E, or F may be subjected to strong ground shaking. It is the intent of Committee 318 that the seismic-force-resisting system of structural concrete buildings assigned to SDC D, E, or F be provided by special moment frames, special structural walls, or a combination of the two. In addition to 21.1.2 through 21.1.8, these structures also are required to satisfy requirements for continuous inspection (1.3.5), diaphragms and trusses (21.11), foundations (21.12), and gravity-load-resisting elements that are not designated as part of the seismic-force-resisting system (21.13). These provisions have been developed to provide the structure with adequate toughness for the high demands expected for these SDCs.

MCBC

Intermediate moment frames are allowed for seismic areas subjected to strong ground shakings. That is structures designed with a ductility behavior factor, Q, equal to 2.

Panama

Panamá, April 29th 2010

Doctor
Guillermo Santana
Chairman Committee ACI-318L
E. S. D.

Dear Dr. Santana,

This is to inform you that the “Reglamento Estructural Panameño, REP-2004”, makes no exceptions to the application of ACI-318 in Panama.

Sincerely,

Oscar M. Ramírez, Ph.D.
Miembro ACI-318L
Panama

Puerto Rico

Angel Herrera, PE

March 31, 2010

MEMORANDUM

TO : Guillermo Santana, PhD, FACI
Chairman, Committee 318-L

FROM: Angel Herrera & José M. Izquierdo

RE : Adoption of Building Code (ACI-318-08)

In Puerto Rico the ACI 318 Code has been used traditionally for the design of building structures.

On 1999, with an effective date of December 8, 1999, the Regulations and Permits Administration formally adopted the UBC 1997 Edition, as the Puerto Rico Building Code, with amendments and additions.

The UBC 1997 adopted the ACI 318-95 and Commentary ACI 318-R 95, so that is the legally adopted code for Puerto Rico. However, it is common practice to use the latest version of the Code, although it is not officially adopted.

It is expected that this summer the Regulations and Permit Administration will adopt the IBC 2009 including the ACI-318-08 in full.

Venezuela

Comparison between the ACI 318-08 and the Venezuelan Code for the Design of Concrete Structures 1753-2006

Prepared by Luis B. Fargier Gabaldon for ACI Committee 318L. April 2010

General Comments related to the 1753-2006 Code

- Approximately 3/4 of Venezuelan territory is earthquake prone. Specified peak design ground accelerations range between 0.1-0.4g.Thus all Code chapters include seismic detailing provisions (Equivalent to those presented in Chapter 21 of the ACI-318).
- ACI 318 Chapter 13, Structural Systems or Elements, was simplified. The Equivalent Frame Method and Direct Design Method are not adopted. Any method for slab design that satisfies equilibrium and compatibility is permitted.
- Chapter 16, Precast Construction; 18, Prestressed Concrete and 19, Shells and Folded Plate Members of the ACI 318-08 Code are not adopted in 1753-2006 because they are uncommon in building built in Venezuela. Thus none of the provisions related to these chapters are adopted in the 1753-2006 code.
- Venezuelan materials meet the ACI 318, ASTM, and ISO requirements.
- Cement, concrete and steel industries are modern industries with high-quality control standards.
- Use of slabs and plates without beams is not permitted in regions of moderate to high seismicity.
- U shape hoops made of two pieces are not permitted. In all cases, 135 degree hooks must be used in a single hoop.
- S and R Loads are not used in design because Venezuela is a tropical country. R shall comply with the provisions in COVENIN 3400-98 Code: Impermeabilization.
- In the load combinations, H shall comply with the provisions in Chapter 11 of the Seismic Code 1756:98
- In the load combinations, E includes horizontal and vertical components acting simultaneously.
- A single formula for calculating development length of tension bars is adopted, based on the expressions proposed in the ACI-318-08 Code.
- Minimum wall thickness requirement are specified for seismic design.
- Chapter 15, Foundations, introduces a limit state design philosophy.

ACI 318-08	VENEZUELA 1753-2006
1.1.9.1 The seismic design category of a structure shall be determined in accordance with the legally adopted general building code of which this Code forms a part, or determined by other authority having jurisdiction in areas without a legally adopted building code *This section was not adopted.*	**Comment: Section 1.4** The Venezuelan code is based on design levels according to current seismic code 1756-1998. The use of seismic design categories is not adopted.
3.5.5 Headed studs and headed stud assemblies shall conform to ASTM A1044M.	**Comment**: The use of headed studs is not adopted.
3.5.8 Steel discontinuous fiber reinforcement for concrete shall be deformed and conform to ASTM A820M. Steel fibers have a length-to-diameter ratio not smaller than 50 and not greater than 100.	**Comment**: The use of steel discontinuous fiber reinforcement is not adopted.
3.5.9 — Headed deformed bars shall conform to ASTM A970M and obstructions or interruptions of the bar deformations, if any, shall not extend more than $2d_b$ from the bearing face of the head	**Comment**: The use of Headed deformed bars is not adopted.
4.2.1 The licensed design professional shall assign exposure classes based on the severity of the anticipated exposure of structural concrete members for each exposure category according to Table 4.2.1	**Comment**: Code recommendations are based on ACI -318-02. The concept of exposure classes is not adopted.
5.3.1.1 Where a concrete production facility has strength test records not more than 12 months old, a sample standard deviation, shall be established.	**Comment**: No time limit is set with respect to the age of the strength test records.
5.6.2.4 A strength test shall be the average of the strengths of at least two 150 by 300 mm cylinders or at least three 100 by 200 mm cylinders made from the same sample of concrete and tested at 28 days or attest age designated for determination of f_c'	**Comment**: 100 by 200 mm cylinders are not permitted.
5.6.6 Steel fiber-reinforced concrete	**Comment**: Steel fiber-reinforced concrete is not permitted.

ACI 318-08	VENEZUELA 1753-2006
8.4.1 Except where approximate values for moments are used, it shall be permitted to decrease factored moments calculated by elastic theory at sections of maximum negative or maximum positive moment in any span of continuous flexural members for any assumed loading arrangement by not more than $1000\,\varepsilon_t$ percent, with a maximum of 20 percent.	**Comment: Section 8.4** is similar except that redistribution of positive moments is not permitted.
8.8 Effective stiffness to determine lateral deflections.	**Comment**: This section is not adopted.
9.2.1 The following load combinations were not adopted: $U = 0.9\,D + 1.6\,W + 1.6\,H$ (9.6) $U = 0.9\,D + 1.0\,E + 1.6\,H$ (9.7)	**Comment: Section 8.4.** The following load combinations replace 9-6 and 9-7 of ACI 318-08: $U = 0.9\,D \pm 1.6\,W$ $U = 0.9\,D \pm E$ $U = 0.9\,D \pm 1.6\,H$
9.3.2.2 "…Compression-controlled sections, as defined in 10.3.3 (a) Members with spiral reinforcement conforming to 10.9.3……... $\phi = 0.75$…"	**Comment: Section 9.4.** Is similar expect that for member with spiral reinforcement $\phi = 0.70$
9.3.5 In Chapter 22, ϕ shall be 0.60 for flexure, compression, shear, and bearing of structural plain concrete.	**Comment: Section 9.4.** Is similar expect that for member made of plain concrete $\phi = 0.55$
11.6.5 For normal weight concrete either placed monolithically or placed against hardened concrete with surface intentionally roughened as specified in 11.6.9, V_n shall not exceed the smallest of $0.20\,f_c'\,A_c, (3.3 + 0.08 f_c')A_c$ and $11\,A_c$ where A_c is area of concrete section resisting shear transfer. For all other cases, V_n shall not exceed the smaller of $0.20\,f_c'\,A_c$ or $5.5\,f_c'$. Where concretes of different strengths are cast against each other, the value of f_c' used to evaluate V_n shall be that of the lower-strength concrete.	**Comment: Section 11.6** is similar, expect that for normal weight concrete either placed monolithically or placed against hardened concrete with surface intentionally roughened as specified in 11.6.9 (ACI-318-08), V_n shall not exceed the smallest of $0.20\,f_c'\,A_c$, and $5.5\,A_c$ where A_c is area of concrete section resisting shear transfer.

ACI 318-08	VENEZUELA 1753-2006
10.6.7. Skin reinforcement is needed when the beam depth exceeds 90 cm.	**Comment: Section 10.3.2.3.** Skin reinforcement is needed when the beam depth exceeds 75 cm.
10.10 ¨…It shall be permitted to consider compression members braced against sidesway when bracing elements have a total stiffness, resisting lateral movement of that story, of at least 12 times the gross stiffness of the columns within the story..¨	**Comment: Section 9.4** is similar expect that the provision on the left column is no adopted.
10.10.2.1 Total moment including second-order effects in compression members, restraining beams, or other structural members shall not exceed 1.4 times the moment due to first-order effects" not adopted.	**Comment: Section 10.10.2.1 (ACI-318-08) is not adopted. Section 10.6.3.2 1753-2006)** requires the following checks for slender compression members (similar to the provisions of ACI 318-05) If an individual compression member has $$\frac{l_u}{r} > \frac{35}{\sqrt{\frac{P_u}{f_c' \cdot A_g}}}$$ it shall be designed for the factored axial load P_u and the moment $M_c$calculated using 10.10.6 in of the ACI 318-08 Code. In addition to load cases involving lateral loads, the strength and stability of the structure under factored gravity loads shall be considered. **(a)** When $d_s M_s$ is computed from a second order analysis, the ratio of second-order lateral deflections to first order lateral deflections for 1.4 dead load and 1.7 live load plus lateral load applied to the structure shall not exceed 2.5; **(b)** When $d_s M_s$is computed according to $d_s M_s = M_s/(1-Q)$the value of Q computed using $\Sigma P_u$for 1.4 dead load plus 1.7 live load shall not exceed 0.60; **(c)** When $d_s M_s$ is computed from $d_s M_s = M_s/(1-\Sigma P_u/0.75\Sigma P_c)$, $d_s$computed using ΣP_u and ΣP_c corresponding to the factored dead and live loads shall be positive and shall not exceed 2.5.

ACI 318-08

10.10.4.1 It shall be permitted to use the following properties for the members in the structure,

(a) Modulus of elasticity .... E_c from 8.5.1

(b) Moments of inertia, ***I***

Compression members:
Columns............................. $0.70\ I_g$
Walls-Uncracked................ $0.70\ I_g$
Cracked....................... $0.35\ I_g$

Flexural members:
Beams................................ $0.35\ I_g$
Flat plates and flat slabs .... $0.25\ I_g$

(c) Area .... $1.00\ A_g$

Alternatively, the moments of inertia of compression and flexural members, ***I***, shall be permitted to be computed as follows:

Compression members

$$I = \left(0.80 + 25 \cdot \frac{A_{st}}{A_g}\right) \cdot \left(1 - \frac{M_u}{P_u} - 0.5 \cdot \frac{P_u}{P_o}\right) \cdot I_g$$

$$I < 0.875 \cdot I_g \dots \dots (10.8)$$

where P_u and M_u shall be determined from the particular load combination under consideration, or the combination of P_u and M_u determined in the smallest value of I. I need not be taken less than $0.35\ I_g$

Flexural members

$$I = (0.10 + 25 \cdot \rho) \cdot \left(1.2 - 0.2 \cdot \frac{b_w}{d}\right) \cdot I_g$$

$$I < 0.50 \cdot I_g \dots \dots (10.9)$$

For continuous flexural members, ***I*** shall be permitted to be taken as the average of values obtained from Eq. (10-9) for the critical positive and negative moment sections I need not be taken less than $0.25\ I_g$. The cross-sectional dimensions and reinforcement ratio used in the above formulas shall be within 10 percent of the dimensions and reinforcement ratio shown on the design drawings or the stiffness evaluation shall be repeated.

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10.6 Comment: Slenderness effects are evaluated using the properties of the gross concrete section to be consistent with the requirements of the seismic design code 1756-2001. Thus Section 10.10.4.1 is not adopted.

ACI 318-08	VENEZUELA 1753-2006
13.2.5 When used to reduce the amount of negative moment reinforcement over a column or minimum required slab thickness, a drop panel shall: (a) project below the slab at least one-quarter of the adjacent slab thickness; and (b) extend in each direction from the centerline of support a distance not less than one-sixth the span length measured from center-to-center of supports in that direction.	**Comment**: **Section 13.1** Structural system without beams, or when all beams have the same thickness as the slab are not permitted in earthquake prone areas
13.2.6 When used to increase the critical condition section for shear at a slab-column joint, a shear cap shall project below the slab and extend a minimum horizontal distance from the face of the column that is equal to the thickness of the projection below the slab soffit.	**Comment**: Section 12.2.6 of the ACI 318-08 Code is not adopted
14.5 Empirical Design Method for Walls.	**Comment**: Empirical Design Method is not adopted.
14.8 Alternative Design of Slender Walls.	**Comment**: Alternative Design of Slender Walls is not adopted.
	Section 14.2.1 sets minimum dimensions for shear walls, Minimum thickness shall be taken as the largest of a and b: $a = 10\ cm$ and the value resulting from $b =$ The lesser of $(L_u/25\ y\ (L_w/25)$. Where L_u is the height of the wall between lateral supports and L_w is the length parallel to shear plane. **Section 14.2.1** When web reinforcement of shear walls consists of two steel meshes, they shall be tied together through the use of alternate single leg ties connecting vertical and horizontal reinforcement.

ACI 318-08	VENEZUELA 1753-2006
15.4.4.2 For reinforcement in short direction, a portion of the total reinforcement, $\gamma_s A_s$, shall be distributed uniformly over a band width (centered on centerline of column or pedestal) equal to the length of short side of footing. Remainder of reinforcement required in short direction, $(1-\gamma_s)A_s$, shall be distributed uniformly outside center band width of footing $\gamma_s = \frac{2}{\beta+1}$ where β is ratio of long to short sides of footing.	**Comment: Section 15.5.4.1** is similar expect that longitudinal reinforcement shall be provided with uniform spacing in the long and short directions.
Chapter 16 Precast Concrete	**Comment:** Precast Concrete Chapter is not adopted
Chapter 18 Prestressed Concrete	**Comment:** Prestressed Concrete Chapter is not adopted
Chapter 19 Shells and Folded Plate Member	**Comment:** Shells and Folded Plate Member Chapter is not adopted
20.3.2 The total test load (including dead load already in place) shall not be less than the larger of (a), (b), and (c): $(a)\ 1.15D + 1.5L + 0.4(L_r \text{ or } S \text{ or } R)$ $(b)\ 1.15D + 0.9L + 1.5(L_r \text{ or } S \text{ or } R)$ $(c)\ 1.3D$ The load factor on the live load L in (b) shall be permitted to be reduced to 0.45 except for garages, areas occupied as places of public assembly, and all areas where L is greater than $4.8\ kN/m^2$ It shall be permitted to reduce L in accordance with the provisions of the applicable general building code	**Comment: Section 17.5.4** Specifies that the total test load (including dead load already in place) shall not be less than $0.85\ (1.4\ D + 1.7\ L)$. It shall be permitted to reduce L according to the use of the structures and specification of the code on minimum loads.

ACI 318-08

21.5.2.3 — Lap splices of flexural reinforcement shall be permitted only if hoop or spiral reinforcement is provided over the lap length. Spacing of the transverse reinforcement enclosing the lap-spliced bars shall not exceed the smaller of $d/4$ and $100\ mm$.

Lap splices shall not be used:

- **(a)** Within the joints;
- **(b)** Within a distance of twice the member depth from the face of the joint; and
- **(c)** Where analysis indicates flexural yielding is caused by inelastic lateral displacements of the frame.

21.5.4.2 — Transverse reinforcement over the lengths identified in 21.5.3.1 shall be proportioned to resist shear assuming $V_c = 0$ when both (a) and (b) occur:

- **(a)** The earthquake-induced shear force calculated in accordance with 21.5.4.1 represents one-half or more of the maximum required shear strength within those lengths;
- **(b)** The factored axial compressive force, P_u, including earthquake effects is less than $\boldsymbol{A_g f'_c/20}$

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18.2.3.2 Comment: This section is similar expect that the $2 \cdot d$ length where splices shall not be used specified in ACI 318-08, Section 21.5.2.3b is replaced by a confinement length L_{cf} as a function of the beam aspect ratio:

Table 18.3.4

L_n/h	L_{cf}
≤ 4	h
$4\ and\ 10$	$\frac{h}{6}\left[\frac{L_n}{h}+2\right]$
> 10	$2h$

18.3.2.b.2 The height of the beam shall be at least 15 times the diameter of the largest column bar that supports it**.**

18.3.2.b.5 When beam width exceeds the width of the supporting column the depth of the column shall be at least 24 times the diameter largest beam longitudinal bar.

Comment: Section18.3.5 (1753-2006) requires that transverse reinforcement over the lengths identified in 21.5.3.1 of the ACI-318-08 code shall be proportioned to resist shear assuming $V_c = 0$ when the factored axial compressive force, P_u, including earthquake effects is less than $A_g f'_c/20$.

The criterion specified in 21.5.4.2a is not adopted.

ACI 318-08

INTERMIDIATE MOMENT FRAMES

21.3.3 ϕV_n of beams and columns resisting earthquake effect, ***E***, shall not be less than the smaller of
(a) and (b):

(a) The sum of the shear associated with development of nominal moment strengths of the member at each restrained end of the clear span and the shear calculated for factored gravity loads;
(b) The maximum shear obtained from design load combinations that include ***E***, with ***E*** assumed to be twice that prescribed by the legally adopted general building code for earthquake-resistant design.

21.3.5.2 At both ends of the column, hoops shall be provided at spacing s_o over a length l_o measured from the joint face. Spacing s_o shall not exceed the smallest of (a), (b), (c), and (d):
- **(a)** Eight times the diameter of the smallest longitudinal bar enclosed;
- **(b)** 24 times the diameter of the hoop bar;
- **(c)** One-half of the smallest cross-sectional dimension of the column;
- **(d)** 300 mm.

21.4 Intermediate precast structural walls

SPECIAL MOMENT FRAMES

21.5.1.4 Width of member, b_w shall not exceed width of supporting member, c_2, plus a distance on each side of supporting member equal to the smaller of (a) and (b):
(a) Width of supporting member, c_2, and
(b) 0.75 times the overall dimension of supporting member, c_1.

VENEZUELA 1753-2006

INTERMIDIATE MOMENT FRAMES

Comment Section 18.7.5.1: ϕV_n of beams and columns resisting earthquake effect, ***E***, shall not be less than,

(a) The sum of the shear associated with development of nominal moment strengths of the member at each restrained end of the clear span and the shear calculated for factored gravity loads. The confined zone is defined in Table 18.3.4 (of the 1753-2006 Code).

The criterion specified in 21.3.3b of the ACI 318-08 is not adopted

Comment Section 18.8.5, is similar expect for the spacing limits:

- **(a)** Eight times the diameter of the smallest longitudinal bar enclosed;
- **(b)** 150 mm.

Comment Intermediate precast structural walls section is not adopted.

SPECIAL MOMENT FRAMES

18.3.2.b.6 Width of member, b_w, shall not exceed width of supporting member, c_2, plus a distance on each side of supporting member equal to 0.75 times the overall dimension of supporting member.

The criterion specified in 21.5.1.4a of the ACI 318-08 is not adopted.

ACI 318-08	VENEZUELA 1753-2006
21.5.3.6 Hoops in flexural members shall be permitted to be made up of two pieces of reinforcement: a stirrup having seismic hooks at both ends and closed by a crosstie. Consecutive crossties engaging the same longitudinal bar shall have their 90-degree hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the crossties are confined by a slab on only one side of the flexural frame member, the 90-degree hooks of the crossties shall be placed on that side.	**Comment:** Hoops made up of two pieces of reinforcement are no permitted by the Code.
21.6.2.3 If 21.6.2.2 is not satisfied at a joint, the lateral strength and stiffness of the columns framing into that joint shall be ignored when determining the calculated strength and stiffness of the structure. These columns shall conform to 21.13. **Note:** Section 21.6.2.2 requires $\Sigma M_{nc} > 1.20\ \Sigma M_{nc}$	**Comment: Section 18.4.3.2 (1753-2006)** includes an alternative procedure to verify the minimum strength of columns by comparing summation of all column strength and beam strength within a story. $\overbrace{\Sigma M_{cn}}^{\substack{all\ columns \\ below\ the\ story}} > 1.2\ \overbrace{\Sigma M_{cn}}^{\substack{all\ beams \\ in\ a\ story}}$ Section 21.6.2.2 of the ACI 318-08 is kept as an option to verify strong-column/weak-beam criterion.
21.7.2.3 Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall not be less than 20 times the diameter of the largest longitudinal beam bar for normal weight concrete. For lightweight concrete, the dimension shall be not less than 26 times the bar diameter.	**Comment: Section 18.4.2 (1753-2006)** requires that longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall satisfy: $\frac{h}{d_b} > \frac{\alpha F_y}{f'_c}$ α varies between 0.08 and 0.1 For lightweight concrete the ratio should be multiplied by 1.3.
21.7.5.4 If epoxy-coated reinforcement is used, the development lengths in 21.7.5.1 through 21.7.5.3 shall be multiplied by applicable factors in 12.2.4 or 12.5.2	**Comment:** Epoxy-coated reinforcement is not adopted by the code in seismic regions.

ACI 318-08

21.8 Special moment frames constructed using precast concrete.

BOUNDARY ELEMENTS OF SPECIAL STRUCTURAL WALLS

21.9.6.3 Structural walls not designed to the provisions of 21.9.6.2 shall have special boundary elements at boundaries and edges around openings of structural walls where the maximum extreme fiber compressive stress, corresponding to load combinations including earthquake effects, *E*, exceeds $0.20\,f_c'$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15\,f_c'$ Stresses shall be calculated for the factored forces using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in 21.9.5.2 shall be used.

21.9.6.4c The boundary element transverse reinforcement shall satisfy the requirements of 21.6.4.2 through 21.6.4.4, except Eq. (21-4) need not be satisfied and the transverse reinforcement spacing limit of 21.6.4.3(a) shall be one-third of the least dimension of the boundary element.

21.9.7.2 Coupling beams with $l_n/h < 2$ and with V_u exceeding $0.33\,\lambda\sqrt{f_c'}A_w$ shall be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan, unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the vertical load carrying ability of the structure, the egress from the structure, or the integrity of nonstructural components and their connections to the structure.

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Comment: Special moment frames constructed using precast concrete are not permitted by the Code.

BOUNDARY ELEMENTS OF SPECIAL STRUCTURAL WALLS

14.6.1 The thickness of the boundary element for walls reinforced with two steel meshes shall be taken greater than the unsupported length divided by 16.

Two procedures are given for determining the need of boundary elements on structural walls.

Procedure 1 is identical to section 21.9.6.2 of the ACI 318-08 Code, based on the length of the compression zone (c).

Procedure 2 replaces section 21.9.6.3:

The need for boundary elements is establish if the factored axial load P_u exceeds the values shown in Table 14.6

Table 14.6 (1753-2006)

Wall shape	Axial load P_u
Symmetric	$\leq 0.10\ A_g\ f_c'$
Asymmetric	$\leq 0.05\ A_g\ f_c'$

If procedure 2 is used, the length of the boundary element will vary linearly between 0.30 to 0.15 L_w when the axial forces ranges from 0.35 to 0.15 P_u, but not less than 0.15 L_w and 450 mm.

Comment: Section 14.6: The boundary element transverse reinforcement shall satisfy the requirements of the ACI 318-08, 21.6.4.2 through 21.6.4.4, except Eq. (21-4) need not be satisfied.

Note: The relaxation of the spacing for transverse reinforcement from one fourth to one third is not adopted.

Comment: Section 14.7.1 The section is identical to Section 21.9.7.2 of the ACI 318-08, except for the limit of the coupling beams aspect ratio requiring diagonal reinforcement. The Code requirement (1753-2006) is more strict than ACI-318-08 and specifies and upper limit of $l_n/h < 4$.

ACI 318-08	VENEZUELA 1753-2006
21.11.7.2 Related to bonded tendons.	**Comment**: Provisions for Pre/Post tension members are not adopted.
21.11.7.6. Longitudinal reinforcement for collector elements at splices and anchorage zones shall have either: **(a)** A minimum center-to-center spacing of three longitudinal bar diameters, but not less than 40 mm, and a minimum concrete clear cover of two and one-half longitudinal bar diameters, but not less than 50 mm; or **(b)** Transverse reinforcement as required by 11.4.6.3, except as required in 21.11.7.5.	**Comment**: Recommendations related to reinforcement for collector elements at splices and anchorage zones are not adopted by the code.
21.11.9.3 Above joints between precast elements in noncomposite and composite cast-in-place topping slab diaphragms, V_n shall not exceed $V_n = A_{vf} \cdot F_y \cdot \mu$ (21.11) where A_{vf} is total area of shear friction reinforcement within topping slab, including both distributed and boundary reinforcement, that is oriented perpendicular to joints in the precast system and coefficient of friction, μ, is 1.0λ, where λ is given in 11.6.4.3. At least one half of A_{vf} shall be uniformly distributed along the length of the potential shear plane. Area of distributed reinforcement in topping slab shall satisfy 7.12.2.1 in each direction.	**Comment**: This section is not adopted. Cast-in-place topping slab diaphragms are not common in Venezuela. Joist construction is dominant.
MEMBERS NOT DESIGNATED AS PART OF THE SEISMIC-FORCE-RESISTING SYSTEM	**MEMBERS NOT DESIGNATED AS PART OF THE SEISMIC-FORCE-RESISTING SYSTEM**
21.13.4 If the induced moment or shear under design displacements, δ_u, exceeds ϕM_n or ϕV_n of the frame member, or if induced moments are not calculated, the conditions of 21.13.4.1, 21.13.4.2, and 21.13.4.3 shall be satisfied.	**Comment: Section 18.10.2.2 (1753-2006)** requires that moments and shears shall not exceed the design moment and shear strength of the frame member.

ACI 318-08	VENEZUELA 1753-2006
21.13.5 Precast concrete frame members assumed not to contribute to lateral resistance	**Comment:** Precast member are not adopted.
21.13.6 For slab-column connections of two-way slabs without beams	**Comment:** Two-way slabs without beams are not permitted in seismic regions. .

VENEZUELA 1753-2006

Section 18.4.2 (1753-2006) Axial load in a column of a special moment resisting frame shall not exceed $0.75\, A_g f_c'$

Section 18.4.5.3 (1753-2006) When core of the column can carry the load combination resulting from EQ effects, it is not necessary to provide confinement requirements according to equations 10.5 and 21.5 of the ACI 318-08 Code.

Section 18.9 (1753-2006) Joint shear of intermediate moment frames shall be determined assuming stresses in longitudinal beam and column reinforcement equal to $1.1\, F_y$ and F_y, respectively.

Acknowledgments: The material prepared by José Grases, Arnaldo Gutiérrez, Denis Rodríguez and Gustavo Parra for the ACI International Workshop Phoenix Oct. 2002 and kindly made available to the writer of this comparison is greatly appreciated.