CHAPTER 1

OBJECTIVES, SCOPE AND RESPONSABILITIES

1.1 OBJECTIVES

The present Code establishes the minimum criteria for the analysis and design of buildings located in areas where seismic motions can occur. Nevertheless they can be modified provided that results of special studies approved by an ad-hoc authority are presented, with design values not less than eighty percent (80%) of those specified in Chapters 4 and 5.

The prescriptions of this Code, are aimed to protect lives, and reduce expected damages in designed buildings. At the same time they seek to keep the essential buildings operative. For the latter, additional studies will be made to ensure its functionability in case of extreme quakes.

This Code substitutes the preceding 1756-82 version and it applies in conjunction with other COVENIN-MINDUR, project, design and building construction codes such as: 2002, 2003, 1753, 1618, 1755, and with those mentioned in the TRANSITORY PROVISIONS, which are indicated in this text. As far as earthquake-resistant aspects, this Code overrules all others.

The prescriptions of this Code comes along with the Commentary that complements and facilitates its application. Number of Sections with commentaries are underlined.

1.2 SCOPE

The prescriptions of this Code are oriented to the design of new reinforced concrete , steel or mixed steel-concrete buildings, of typified behavior, in which simplifications can be used based on previous experiences. The upgrading, reinforcement or repair of existing buildings, will follow what is established in Chapter 12 of this Code.

The requirements for the analysis and design of buildings with pre-cast bearing members are not established here, neither are the requirements for special structures such as, although not limited to: bridges, transmission towers, peers, hydraulic structures, nuclear power plants, electrical and mechanical installations, elevated tanks, etc. In this cases additional considerations or special studies that complement the basic lineaments of the present Code are required.

The national, state or municipal authorities, according to the case and determined by laws, will have to establish the rules for the evaluation, improvement or demolition of existing buildings, as well as the incorporation of the seismic variables in the development plans.

1.3 RESPONSABILITIES

In accordance with the laws of the Republic, the responsibility to ensure the correct application of this Code and its general lineaments lies on the urban planners, architecture and engineering professionals, as well as on builders and inspectors, in their corresponding fields of action. The owner will have to ensure an adequate inspection, adjust to the foreseen use of the project and maintenance of the building. Special precautions will have to be made to avoid the presence of non structural elements or components, such as, filling walls (continuous or discontinuous), parapets, flower stands, and others that may modify the assumed structural model used in the project, such as the short column effect.

Particularly, the professional responsible for the inspection will have to keep record of the project deviations that may occur during its execution.

CHAPTER 2

DEFINITIONS AND NOTATION

2.1 DEFINITIONS

The meaning of the terms of regular use in this Code are defined. Their meanings may differ from those established in other COVENIN-MINDUR venezuelan codes. The underlined words point out terms defined in this vocabulary.

accelerogram. Time record of the variation of the accelerations due to strong ground motions.

accelerograph, accelerometer. Instrument specifically designed to register accelerograms.

accidental eccentricity. Additional value to the static eccentricity that takes into account the effects due to irregularities in the distribution of the masses and rigidities, as well as the effects of the rotational excitation of the ground.

appendices. Architectonic parts, such as canopies, parapets and façade elements.

base level. Level of the building where it is assumed that the seismic actions are transmitted to the structure.

building. Is a structure that has diaphragms capable to compatibilize the horizontal displacements of the members that reach that level.

connection. Combination of joints to transfer forces between two or more members.

damping coefficient. It measures the damping of the structure as a percentage of the critical damping. The critical damping is the limit value above which the free movement of the structure is not vibratory.

design acceleration. Value of the ground acceleration for the earthquake resistant design of engineering works.

design forces. Forces that represent the seismic action over the building or its components; they are specified at yield level.

design level. Group of code prescriptions associated to a determined ductility factor, that is applied in the design of members of the seismic resistant system, typified in this Code.

design motions. Ground motions selected in such way that its probability of exceedence is adequate to the expected performance during the service life of the structure; they are characterized by their response spectra.

design spectrum. Spectrum that incorporates the response reduction factor corresponding the adopted seismic resistant system.

diagonalized frames. Vertical truss type systems or equivalent, alocated to resist the seismic actions whose members are submitted mainly to axial forces.

diaphragm. Part of the structure, generally horizontal, with enough rigidity on its plane, designed to transmit the forces to the vertical elements of the seismic resistant system.

drift. Total lateral displacements difference between two consecutive levels

ductility demand. Ratio between the maximum displacement reached by a system during its seismic response and the yield displacement.

ductility. Ability that the components of a structural system have, to make alternate incursions in the inelastic domain without a noticeable loss in its resistance.

dynamic amplification factor. Ratio between the dynamic eccentricity and the static eccentricity.

dynamic analysis. Analysis in which the seismic actions are characterized by means of a design spectrum or accelerograms.

dynamic eccentricity. Ratio between the torsional moment obtained from a dynamic analysis with three degrees of freedom per level, calculated with respect to the rigidity center, and the shear force at that level.

floor. Each one of the levels that conforms the building.

horizontal acceleration coefficient. Ratio between the maximal horizontal acceleration and the acceleration of gravity.

inertia radius. Is the square root of the ratio between the rotational inertia with respect to the shear center and the mass, for each building level.

interstory. Space between two consecutive levels.

overresistance. Real value of the strength capacity, including the structural and non-structural elements, which exceeds the calculated nominal resistance.

P-Δ effect. Effect due to the axial loads and the lateral displacements over the bending moments of the members.

permanent actions. Represents the gravitational loads due to the weight of all the structural and nonstructural components, such as structural walls, floors, roofs, partition walls, service units attached to the structure and any other fixed service load.

post-seismic analysis. Static analysis of the stability after an earthquake, taking into consideration the eventual changes of the soil resistance as a consequence of seismic shaking.

rehabilitation. See upgrading

reinforcement. Constructive actions to improve the seismic capacity of the building through the modification of its resistance and rigidity.

repair. Damage repair, but without increasing the seismic capacity of the building beyond its initial condition.

response reduction factor. Factor that divides the ordinates of the elastic response spectrum to obtain the design spectrum.

response spectrum. Represents the maximal response of oscilators with one degree of freedom and with a common damping coefficient, submitted to a history of given accelerations, expressed in terms of the period.

rigidity center of a level. Point on the level where the level is displaced without rotating with respect to the lower level when a horizontal shear force is applied.

seismic action. Accidental action due to the occurrence of quakes, which incorporates the translational and rotational effects with respect to the vertical axis.

seismic coefficient. Ratio between the horizontal shear design force that acts at the base level and the total weight above it.

seismic forces. External forces, capable of reproducing the extreme values of the displacements and the internal solicitations caused by the seismic excitation acting at the base of the building.

seismic hazard. Quantifies the probability of occurrence of future seismic events that can adversely affect the integrity of buildings and its occupants.

seismic resistant system. Part of the structural system that is considered to provide the building with the resistance, rigidity and ductility necessary to uphold the seismic actions.

seismic threat. See seismic hazard.

seismic zone. Geographical zone in which it is admitted that the maximum expected intensity of the seismic actions, in a given period of time, is similar in all its points.

service life. Period of time or duration in which it is assumed that a structure will be used for the purpose which it was designed for.

shear center. Point where the shear force acts in a level, considering that the horizontal forces in each level act in the respective centers of mass.

short column effect. Marked reduction of the free column length by the effect of lateral restrictions.

site studies. Evaluation of the seismic threat taking into consideration the local conditions of the site.

soft story. Configuration characterized by a marked difference in rigidity between adjacent levels.

static eccentricity. Distance between the shear force action line and the rigidity center.

structural walls. Walls specially designed to resist a combination of shear, moment and axial forces induced by the seismic ground motions and/or the gravitational actions.

torsional moment. Addition of the torsional pairs in each level above the considered level, including this, plus the normal torsional moment at that level, product of the level's shear force multiplied by its static eccentricity.

torsional pair. Moment vector normal to the considered level and referred to its rigidity center.

torsional radius. Is the square root of the ratio between the torsional rigidity with respect to the shear center and the lateral rigidity in the considered direction, for each building level.

upgrading. Constructive actions aimed to reduce the seismic vulnerability of a building, such as: modifications, rehabilitation, reinforcement, seismic isolation or use of energy disipation devices.

variable actions. Loads originated by the use and occupation of the building, excluding permanent, wind or quake loads or actions.

weak story. Configuration characterized by a marked resistance difference between adjacent levels.

yielding. Condition characterized by the plastification of at least the most stressed region of the seismic resistant system, such as the formation of the first plastic hinge in a main component of it.

2.2 NOTATION

The sub-index x or y refer to directions X or Y. The sub-index i, j, k are used to refer to any level; for the upper most level the letter N is used.

A = Contact area (Subsection 11.4.5.2)

Ad = Design spectrum ordinate expressed as a fraction of the acceleration of gravity (Article 7.2)

Ao = Horizontal acceleration coefficient. (Article 4.2).

B_i = Width of the plant in the normal direction to the analyzed direction (Article 9.5)

C = Seismic coefficient (Article 7.1).

Cp = Seismic coefficient of elements or structural parts; (Table 7.2)

CP = Effects due to permanent loads (Subsection 7.3.2.1).

CV = Effects due to variable loads (Subsection 7.3.2.1).

ED = Effect due to the pressure of the soil or another material under dynamic conditions (Article 11.4).

F_i = Lateral force (Section 9.3.3).

Fp = Forces due to the seismic action on elements or parts of structures (Section 7.3.2).

F_t = Lateral force concentrated in the last considered level (Section 9.3.3).

M_a = Acting moment (Subsection 11.5.4.2)

M_r = Resisting moment (Subsection 11.5.4.2)

M_t = Torsional moment (Article 9.5).

N_a = Normal force to the contact area (Subsection 11.4.5.2)

N = Number of levels of a building (Section 9.3.1).

N_1 = Number of modes to consider in the dynamic analysis with one degree of freedom per level (Section 9.4.4) and (Section 9.6.2).

N_3 = Number of modes to consider in the dynamic analysis with three degrees of freedom per level (Subsection 9.6.2.1).

P = Vertical force (Article 8.5).

$P\text{-}\Delta$ = Second order effect (Article 8.5).

Q = Forces for the verification of the foundations' bearing capacity (Section 11.4.4)

Q_{ult} = Ultimate load capacity (Subsection 11.4.6.3).

R = Response reduction factor (Section 6.4.2).

R_s = Allowable pressure under static loads (Section 11.3.4).

S = Effects due to seismic actions (Article 11.2).

S_e = Cohesive grounds' sensibility (Subsection 11.4.5.1).

S_u = Undrained shear resistance (Article 11.2).

S_{us} = Undrained residual resistance of cohesive soils (Subsection 11.4.5.1).

T = Fundamental period of the building in seconds.

T_a = Fundamental period of the building in seconds, based on empirical relationships (Subsection 9.3.2.2).

T_o = Smaller value of the period in the range of constant spectral values, in seconds (Article 7.2).

T^* = Maximum value of the period in the interval where the spectral values have a constant value, in seconds (Article 7.2).

$\overline{T}$ = Period associated to the mode which has the largest contribution in the base shear force (Section 9.4.6).

V = Shear force

V_o = Base Shear force (Section 7.1.1).

V_s = Shear wave velocity of propagation (Article 5.1).

V_{sp} = Average velocity of shear waves (Article 5.1).

W = Total weight of the building above the base level (Section 7.1.1).

W_p = Weight of elements or parts of structures (Section 7.3.2).

c = Adhesion between the floor and foundation (Subsection 11.4.5.2).

e = Static eccentricity, equal to the distance between the rigidity center and the shear action line in the analyzed direction (Article 9.5).

g = Acceleration of gravity equal to 9.81 m/seg^2

h = Height

q = Maximum transferred compression stresses by the foundation to the ground (Subsection 11.4.5.1).

q_{adm} = Soil's bearing capacity (Subsection 11.4.5.1).

q_{ult} = Ultimate strength bearing capacity of the soil (Section 11.4.5).

r = Inertial radius (Article 9.5).

r_t = Torsional radius (Article 9.5).

Δ = Total lateral displacement including the inelastic effects; sub-index e indicates the elastic part of it (Article 10.1).

θ = Stability coefficient (Article 8.5).

Φ = Resistance reduction factor

Φ_{im} = Modal coordinate of i level in mode m (Section 9.4.5).

α = Importance factor (Section 6.1.3).

β = Average magnification factor (Article 7.2).

δ = Drift (Article 10.1).

δ_{ei} Difference of the elastic lateral displacements between two consecutive levels, in its corresponding mass centers (Article 8.5).

μ = Shear modification factor (Section 9.3.1).

μ_f = Ground-foundation friction coefficient (Subsection 11.4.5.2).

ρ = Overturning reduction factor (Section 9.2.4).

φ = Correction factor of the horizontal acceleration coefficient (Article 5.1).

ϕ = Resistant reduction factor (Subsection 11.4.6.3)

τ = Dynamic amplification factor of the torsional moment (Article 9.5).

τ' = Design control factor in the most rigid zone of the plant, for the considered direction (Article 9.5).

ξ = Damping coefficient (Commentary C-7.2).

ε = Representative value of e/r (Article 9.5).

Ω = Representative value of r_t/r (Article 9.5).

CHAPTER 3

APPLICATION GUIDE AND BASIC LINEAMENTS

3.1 CLASIFICATION

In order to apply this Code, all buildings have to be assigned to one of the seismic zones established in Chapter 4 and properly classified according to Chapter 6. The dynamic response of the foundation soils, have to be classified according to the spectral forms typified in Chapter 5.

3.2 SEISMIC ACTION, CRITERIA AND ANALYSIS METHODS

The seismic action is characterized by means of design spectra specified in Chapter 7, which take into account: the seismic zoning, the geotechnical profiles, the damping coefficient and the ductility. The analysis criteria are provided in Chapter 8 and the analysis methods are established in Chapter 9 for the superstructure and in Chapter 11 for the infrastructure .

The maximum displacements shall not exceed the limits prescribed in Chapter 10.

3.3 DESIGN REQUIREMENTS

The quality of the materials to be used, the design and details of the resistant members and its unions, will have to satisfy the COVENIN Codes . In particular, the joints of the seismic resistant system, should have a resistance that exceeds those of the members.

Whenever (exceptionally), and subject to the approval of the ad-hoc authority, design procedures different to the ones established in the actual COVENIN Codes are used, equivalent levels of safety will have to be guaranteed.

3.3.1 TRANSITORY DISPOSITION

Until the COVENIN-MINDUR 1753 Code is not updated, the use of the last ACI 318 Code version is authorized, complemented with the dispositions indicated in this document.

3.4 OTHER BUILDINGS

In the analysis and design of buildings that cannot be classified in any of the Types described in this Code, special considerations will have to be followed, according to each case, that complement the basic lineaments of the present Code, previously approved by the ad-hoc authority

3.5 BASIC LINEAMENTS

In addition to the criteria established in Articles 3.1 to 3.3 and in Chapter 8, the present Code is ruled according to the following lineaments

a) The design solicitations presuppose that the seismic resistance system is capable of absorbing and dissipating energy under alternate type actions, in the inelastic range, without appreciable loss of its resistance;

b) The energy dissipation and absorption mechanisms should not compromise the stability of the building. The energy dissipation zones are distributed among the different elements that constitute the structure, predominantly in beams or lintels;

c) The response reduction factors R, are backed-up by abundant experimental and field information;

d) The design spectra are given at yield level, thus the load factor of the seismic actions is equal to 1.0;

e) The seismic action is considered as an accidental one and is not combined with other accidental actions of similar probability of occurrence. Even though wind loads may be greater than those of quakes, the detailing requirements and the limitations of the seismic design established here, should be kept;

f) The presence of the non-structural elements, are incorporated in what concerns its effect in the: rigidity, resistance and ductility of the seismic resistant system;

g) The design considers the action of the three translational components of the quake and the rotation about the vertical axis;

h) This Norm presupposes that the structural elements are joined between themselves, in a way that they allow the transmission of the seismic loads;

i) The mathematical model describes in an adequate way the expected structural response. Whenever it applies, in the calculation of the seismic resistant system displacements, the effects of the rotation of the nodes, the deformations due to shear and bending of the members, as well as its axial deformations, should be included. Whenever rigid arms are modeled its longitude will be limited to a fraction of it;

j) The final reliability of the building, depends on the fulfilment of this Code and the design ones, as well as the correct execution, inspection and maintenance.

CHAPTER 4

SEISMIC ZONING

4.1 ZONING MAP

In order to apply this Code, the country has been divided in eight zones. This are indicated in Figure 4.1 and in Table 4.2. The zonation of areas adjacent to dams of more than 80 meters high shall be ruled by special studies.

4.2 DESIGN MOTIONS

The parameters that characterize the design motions depend on the local geotechnical conditions defined in Chapter 5. The coefficient of the horizontal acceleration for each zone is given in Table 4.1. The vertical coefficient of acceleration, will be taken as 0.70 times the A_o values given in Table 4.1.

TABLE 4.1

VALUES OF A_o

SEISMIC ZONES	A_o
7	0.40
6	0.35
5	0.30
4	0.25
3	0.20
2	0.15
1	0.10
0	--

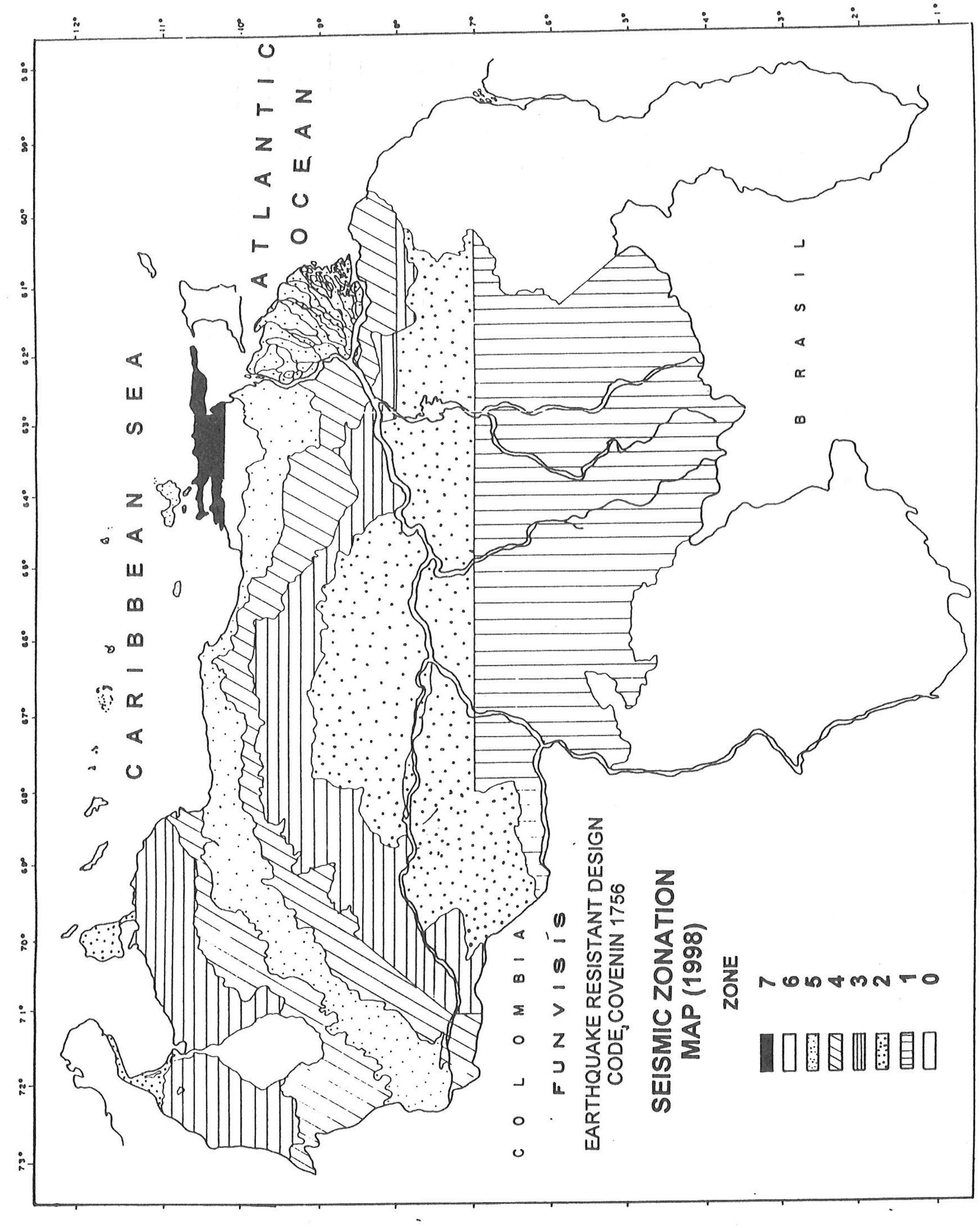
CARIBBEAN SEA
ATLANTIC OCEAN
BRASIL
COLOMBIA
FUNVISIS
EARTHQUAKE RESISTANT DESIGN CODE, COVENIN 1756
SEISMIC ZONATION MAP (1998)
ZONE
7
6
5
4
3
2
1
0

TABLE 4.2

SEISMIC ZONING OF VENEZUELA

STATE	
AMAZONAS	**Zone 1:** County of Atures **Zone 0:** Counties of: Autana, Manapiare, Atabapo, Alto Orinoco, Guainia, Río Negro.
ANZOATEGUI	**Zone 6:** Counties of: Guanta, Juan Antonio Sotillo, Turístico Diego Bautista Urbaneja and Area of the Simón Bolívar County to the North of parallel 10º N. **Zone 5:** Counties of: Píritu, Libertad, Fernando de Peñalver, San Juan de Capistrano Area of the Pedro María Freites County, to the North of the La Encrucijada-La Ceiba-El Tejero road and Area of the Simón Bolívar County to the South of paralel 10º N **Zone 4:** Counties of: San José de Guanipa, Simón Rodríguez, Aragua, Santa Ana, Anaco, Juan Manuel Cajigal, Francisco del Carmen Carvajal, Manuel Ezequiel Bruzual, and Area of the Pedro María Freites County, to the South of the La Encrucijada-La Ceiba-El Tejero road **Zone 3:** Counties of: Sir Arthur Mc Gregor, Francisco de Miranda, Independencia. **Zone 2:** County of José Gregorio Monagas.
APURE	**Zone 4:** Area of the Páez County, to the West of the 71°W meridian **Zone 3:** Area of the Páez County, excluding the zone to the West of the 71°W meridian **Zone 2:** Area of the Pedro Camejo County located to the North of parallel 7º N and the Counties of: Rómulo Gallegos, Muñoz, Achaguas, Biruaca, San Fernando. **Zone 1:** Counties: Area of the Pedro Camejo County located to the South of parallel 7º N.
ARAGUA	**Zone 5:** Counties of: Tovar, Santiago Mariño, Mario Briceño Iragorry, Libertador, Girardot, José Angel Lamas, Jose Rafael Revenga, Francisco Linares Alcántara (*). **Zone 4:** Santos Michelena, Bolívar, Sucre, Rivas, Zamora, San Sebastián, San Casimiro. **Zone 3:** Counties of: Camatagua, Urdaneta.

TABLE 4.2 (Cont.)

SEISMIC ZONING OF VENEZUELA

STATE	
BARINAS	**Zone 4:** Counties of: Alberto Arvelo Torrealba, County of Cruz Paredes, Bolívar. Areas to the Northwest of the Ezequiel Zamora, Antonio José de Sucre, Peraza, Barinas and Obispos Counties, limited by a line parallel to the Santa Bárbara-Boconoito road, about 10 km to the Southwest of it. **Zone 3:** Rest of the State, excluding the areas in Zone 4 and the Arismendi County. **Zone 2:** Arismendi County
BOLÍVAR	**Zone 3:** Area of the Piar County North of parallel 8º N and the Caroní County. **Zone 2:** County of Heres, Areas of the Cedeño, Sucre, Raúl Leoni, Sifontes, Roscio y El Callao Counties, located to the North of parallel 7º N, and Area of the Piar County North of parallel 7º N and South of parallel 8º N. **Zone 1:** Areas of the Cedeño, Sucre, Raúl Leoni, Sifontes, José Tadeo Monagas, Piar and El Callao Counties, located to the South of parallel 7º N, County of Gran Sabana and County of Padre Pedro Chien (*).
CARABOBO	**Zone 5:** Counties of: Guacara, San Diego, Naguanagua, Montalbán, Miranda, Los Guayos, Juan José Mora, Puerto Cabello, Bejuma, San Joaquín, Diego Ibarra, Lago de Valencia and Areas of the Valencia and Libertador Counties to the North of parallel 10º N. **Zone 4:** County of Carlos Arvelo and Areas of the Valencia and Libertador Counties to the South of parallel 10º N.
COJEDES	**Zone 4:** Counties of: Anzoategui, San Carlos, Lima Blanco, Falcón. **Zone 3:** Counties of: Girardot, Ricaurte, Rómulo Gallegos, Tinaco, Pao de San Juan Bautista.
DELTA AMACURO	**Zone 5:** Counties of: Pedernales, Tucupita and Areas of the County of Antonio Díaz located in the Delta to the North of the Orinoco River. **Zone 4:** County of Casacoima and Areas of the County of Antonio Díaz located to the South of the Orinoco River. **Zone 3:** Areas of the County of Antonio Díaz located South of parallel 8º N.
DTO. FEDERAL	**Zone 5:** The whole District

TABLE 4.2 (Cont.)

SEISMIC ZONING OF VENEZUELA

STATE	
FALCÓN	**Zone 4:** Counties of: Monseñor Iturriza, Silva **Zone 3:** Rest of the State. **Zone 2:** Counties of: Falcón, Carirubana, Los Taques.
GUARICO	**Zone 3:** Counties of: Ortíz, Juan Germán Roscio, Julián Mellado, Chaguaramas, José Tadeo Monagas, San José de Guaribe, José Félix Rivas, Pedro Zaraza and Area of the County of Leonardo Infante to the North of parallel 9º N. **Zone 2:** Counties of: Camaguan, San Gerónimo de Guayabal, Francisco de Miranda, Las Mercedes and Area of the County of Leonardo Infante to the South of parallel 9º N, El Socorro and Santa María de Ipire.
LARA	**Zone 5:** Counties of: Morán, Andrés Eloy Blanco, Jiménez, Iribarren, Palavecino, Simón Planas, Crespo. **Zone 4:** Counties of: Torres and Urdaneta.
MÉRIDA	**Zone 5:** Counties of: Tovar, Antonio Pinto Salinas, Guaraque, Sucre, Andrés Bello, Caracciolo Parra Olmedo, Justo Briceño, Miranda, Rangel, Libertador, Campo Elías, Arzobispo Chacón, Aricagua, Zea, Rivas Dávila, Julio Cesar Salas, Pueblo Llano, Cardenal Quintero, Santos Marquina and Padre Noguera. **Zone 4:** Counties of: Alberto Adriani, Obispo Ramos de Lora, Tulio Febres Codero and Julio César Salas.
MIRANDA	**Zone 5:** Counties of: Andrés Bello, Buroz, Brión, Zamora, Plaza, Sucre, Chacao, Guaicaipuro, El Hatillo, Baruta, Los Salias, Carrizal and Areas of the Páez and Pedro Gual Counties to the North of the Autopista de Oriente. **Zone 4:** Counties of: Urdaneta, Paz Castillo, Lander, Acevedo, Cristóbal Rojas, Simón Bolívar, Independencia and Areas of the Páez and Pedro Gual Counties to the South of the Autopista de Oriente.
MONAGAS	**Zone 6:** Counties of: Acosta, Piar, Caripe, Bolívar, Punceres **Zone 5:** Counties of: Cedeño, Ezequiel Zamora, Santa Bárbara and Area of the County of Maturín to the North of parallel 9º N. **Zone 4:** Counties of: Aguasay, Libertador, Uracoa, Sotillo and Area of the County of Maturín to the South of parallel 9º N.
NUEVA ESPARTA	**Zone 5:** The whole State

TABLE 4.2 (Cont.)

SEISMIC ZONING OF VENEZUELA

STATE		
PORTUGUESA	**Zone 4:**	Counties of: San Jenaro de Boconoito, Sucre, Guanare, Monseñor José Vicente de Unda, Ospino, Esteller, Araure, Páez, Agua Blanca, San Rafael de Onoto.
	Zone 3:	Counties of: Guanarito, Papelón, Santa Rosalía, Turén
SUCRE	**Zone 7:**	Counties and Areas located to the North of the parallel that goes along the North coast of the Santa Fe Gulf (approximately 10° 20' N)
	Zone 6:	Rest of the State.
TACHIRA	**Zone 5:**	Counties of: Simón Rodríguez, Antonio Rómulo Costa, Seboruco, José María Vargas, Michelena, Andrés Bello, Guasimos, Independencia, Lobatera, Pedro María Ureña, Libertad, Bolívar, Rafael Urdaneta, Junín, Torbes, San Cristóbal, Cadenas, Sucre, Francisco de Miranda, Córdoba, Fernández Feo, Libertador, Ayacucho, Jauregui, Uribante (*), Samuel Darío Maldonado (*).
	Zone 4:	Counties of: García de Hevia, Panamericano, Samuel Darío Maldonado.
TRUJILLO	**Zone 5:**	Counties of: Valera, Urdaneta, Boconó, Carache, Trujillo, Pampan, Candelaria, Pampanito, San Rafael de Carvajal, Juan Vicente Campo Elías.
	Zone 4:	Counties of: La Ceiba, Monte Carmelo, Bolívar, Sucre, Miranda, Andrés Bello, José Felipe Marquez Cañizales, Motatán, Rafael Rangel, Escuque.
YARACUY	**Zone 4:**	Counties of: Bolívar, Manuel Monge
	Zone 5:	Counties of: Veroes, San Felipe, Bruzual, Peña, Nirgua, Independencia, Cocorote, Sucre, Aristides Bastidas, La Trinidad, Urachiche, José Antonio Páez.
VARGAS	**Zone 5:**	The whole State.
ZULIA	**Zone 4:**	Counties of: Jesús María Semprún, Catatumbo, Colón, Francisco Javier Pulgar, Sucre.
	Zone 3:	Counties of: Mara, Jesús Enrique Lossada, Maracaibo, San Francisco, La Cañada de Urdaneta, Rosario de Perijá, Machiques de Perijá, Baralt, Valmore Rodríguez, Lagunillas, Cabimas, Santa Rita, Miranda, Simón Bolívar.
	Zone 2:	County of: Páez and Almirante Padilla
CARIBBEAN ISLANDS	**Zone 5:**	All the islands of the Caribbean region

(*) County with uncertain geographical limits.

CHAPTER 5

TYPIFIED SPECTRAL SHAPES FOR SOIL FOUNDATIONS

This Code considers four typified spectral shapes (S1 to S4) and a correction factor for the horizontal acceleration coefficient (φ), which depends on the characteristics of the foundation ground's geo-technical profile.

5.1 SELECTION OF THE SPECTRAL SHAPE AND φ FACTOR

The spectral shape and φ factor shall be selected in accordance to Table 5.1.

TABLE 5.1 SPECTRAL SHAPE AND φ CORRECTION FACTOR

Material	V_{sp} (m/s)	H (m)	Spectral Shape	φ
Sound/fractured rock	>700	--	S1	0.85
Soft or Moderately meteorized rock	>400	≤50	S1	0.90
		>50	S2	0.95
Very hard or very dense soils	>400	<30	S1	0.90
		30-50	S2	0.95
		>50	S3	1.00
Hard or dense soils	250-400	<15	S1	0.90
		15-50	S2	0.95
		50-70	S3 [(b)]	1.00
		>70	S4	1.00
Firm/mid-dense grounds	170-250	≤50	S2 [(c)]	1.00
		>50	S3 [(b)]	1.00
Soft to loose soils	<170	≤15	S2 [(c)]	1.00
		>15	S3 [(b)]	1.00
Soft soils with inter-spaced strata With other more rigid soils [(a)]	<170	< H_1	S2	1.00
		> H_1	S3	0.90

(a) The width of the strata should be greater than 0.1 H
(b) If Ao ≤ 0.15, use S4
(c) If Ao ≤ 0.15, use S3

where:

H = Depth in which material is found with Vs shear wave velocities greater than 500 m/s
H_1 = Depth from the surface to the top of the stratum (m) = 0.25 H
V_{sp} = Average velocity of the shear waves in the geo-technical profile (m/s).
φ = Correction factor of the horizontal acceleration coefficient.

The spectral shapes are given in Table 7.1 and Figure C-7.1.

5.2 SPECIAL CASES

In those cases in which the selection of the spectral shape in accordance to Table 5.1 comes out doubtful, the spectral shape leading to the most unfavorable seismic actions shall be used.

When, within the geo-technical profile, there are soils that have a degrading resistance or that suffer volumetric changes under the action of the seismic actions, special studies should be made in order to evaluate the dynamic response of the profile as well as establish the spectral shape and horizontal acceleration coefficient to be used in the design. The models used for the various analysis shall reflect the changes in the properties of such soils due to the effect of the cyclic load.

CHAPTER 6

CLASSIFICATION OF BUILDINGS ACCORDING TO USE, LEVEL OF DESIGN, TYPE AND STRUCTURAL REGULARITY

To the effects of the application of this Code, the buildings will be classified according to their use, level of design, type and structural regularity.

6.1 CLASSIFICATION ACCORDING TO USE

6.1.1 GROUPS

The building shall be classified in one of the following Groups:

GROUP A

Buildings where essential installations operate, essential in emergency conditions or which failure may give place to heavy human or economic losses, such as, although not limited to:

- Hospitals: Type IV, Type III and Type II.
- Important government or county buildings, monuments and temples of exceptional value.
- Buildings that contain objects of exceptional value, like certain museums and libraries.
- Firemen or police stations, or headquarters.
- Power stations, high voltage and telecommunications substations. Pumping plants.
- Deposits of toxic or explosive materials and centers that use radioactive materials.
- Control towers; hangars; air traffic centers.
- Educational buildings with capacity for 200 people or more.
- Buildings that may endanger any one belonging to this Group.

GROUP B1

Buildings of public or private use, densely occupied, permanently or part-time, such as:
- Buildings with an occupation capacity of more than 3000 people or roofed area of more than 20.000 m^2.
- Health centers not included in Group A.
- Educational centers with capacity for less than 200 people.
- Buildings classified in Groups B2 or C which can endanger those of these Group.

GROUP B2

Buildings of public or private use, of low occupation, which do not exceed the limits indicated in Group B1, such as:
- Houses.
- Apartment, office or hotel buildings.
- Banks, restaurants, cinemas and theaters.
- Warehouses and storage buildings.
- All buildings classified in Group C, whose failure can put in danger those of these Group

GROUP C

- Buildings that can not be classified in the previous groups, nor destined to be housed or for public use, whose failure can not cause damage to buildings of the three first Groups.

6.1.2 MIXED USES

Buildings that house areas belonging to more than one Group, will be classified in the most demanding Group.

6.1.3 IMPORTANCE FACTOR

According to the previous classification an importance factor α is established, as given in Table 6.1.

TABLE 6.1

IMPORTANCE FACTOR

GROUP	α
A	1.30
B1	1.15
B2	1.00

Buildings of Group C do not require seismic analysis.

6.2 CLASSIFICATION ACCORDING TO THE DESIGN LEVEL

In order to apply this Norm, the three levels of design that are specified in Section 6.2.1., are distinguished.

6.2.1 DESIGN LEVELS

DESIGN LEVEL 1

The design in seismic zones does not require the application of additional requirements to those established for gravitational actions.

DESIGN LEVEL 2

It requires the application of the additional requirements for this Design Level, established in the COVENIN-MINDUR Codes.

DESIGN LEVEL 3

It requires the application of all the additional requirements for the design in seismic zones established in the COVENIN-MINDUR Codes.

6.2.2 REQUIRED DESIGN LEVELS

One of the Design Levels (DL) indicated in Table 6.2 shall be used. In the case of structures with the irregularities indicated in Table 6.3, typified in Section 6.5, it is required to extend the compliance of Design Level DL3 along the whole length of members that are part of the irregularity pointed out.

TABLE 6.2

DESIGN LEVELS (DL)

GROUP	SEISMIC ZONE		
	1 and 2	3 and 4	5,6 and 7
A; B1	DL2 DL3	DL3	DL3
B2	DL1 (*) DL2 DL3	DL2 (*) DL3	DL2 (**) DL3

(*) Valid for buildings of up to 10 floors or 30 m tall.
(**) Valid for buildings of up to 2 floors or 8 m tall.

TABLE 6.3

AREAS AND/OR COMPONENTS IN WHICH THE COMPLIANCE OF THE DESIGN REQUIREMENTS DL3 SHOULD BE EXTENDED

TYPE OF IRREGULARITY		IRREGULARITY (SECTION 6.5.2)	AREAS OR COMPONENTS
VERTICAL	a.1.	Soft story.	All the components of the story and of the adjacent stories.
	a.2.	Weak story.	
	a.7.	Discontinuity in the plane of the system resistant to lateral loads.	The components where the discontinuity occurs and all the adjacent components.
	a.9	Short columns.	
IN PLAN	b.2.	High torsional risk.	The whole structure.
	b.4.	Flexible diaphragm.	All the components that are linked to the referred diaphragm.

6.3 CLASSIFICATION ACCORDING TO THE TYPE OF STRUCTURE

To meet the objectives of this Code, types of structural systems are established according to the components of the seismic resistant systems, described in Section 6.3.1. A structure can be classified in different types, in its two orthogonal directions of analysis.

All the structural types, with the exception of **Type IV**, require diaphragms with the necessary rigidity and resistance to distribute the seismic actions efficiently between the different members of the seismic resistant system. In Seismic Zones 3 to 7, both included, the systems of floors without beams are not allowed, nor floors where all the beams are flat.

6.3.1 TYPES OF STRUCTURAL SEISMIC RESISTANT SYSTEMS

TYPE I: Structures capable of resisting all of the seismic actions through deformations due essentially to the bending of its beams and columns, such as the structural systems formed by frames. The axis of the columns should be kept continuous all the way to its foundation.

TYPE II: Structures formed by combinations of **Types I and III**, both having the same Design Level. Its joint action should be capable of resisting the design seismic forces. The frames alone should be capable of resisting at least twenty five per cent (25%) of these forces.

TYPE III: Structures capable of resisting the design seismic actions through diagonalized frames or structural reinforced concrete walls or of a mixed steel-concrete section, that support all permanent and variable loads. **Type II** structures whose frames are not capable of resisting at least twenty five per cent (25%) of the total seismic forces by themselves, respecting in its design the Design Level adopted for the whole structure will be considered equally within this group. **Type IIIa** are distinguished as the systems setup by reinforced concrete walls linked with ductile lintels, as well as steel frames with eccentric diagonals coupled with ductile links.

TYPE IV: Structures that do not have diaphragms with the necessary stiffness and resistance to efficiently distribute the seismic forces between the different vertical members. Structures supported by one column.

6.3.2 COMBINATION OF STRUCTURAL SYSTEMS

In the event that in any analysis-direction more than one structural system is used, in that direction the R value will be the smaller of the corresponding values given in Table 6.4. When in the vertical combination of two systems, one of the components supports a weight equal to or less than ten per cent (10%) of the total weight of the building, it is not necessary to satisfy this requirement.

6.4 RESPONSE REDUCTION FACTOR

The maximum values of the R reduction factor, for the different types of structures and Design Levels, are given in Table 6.4, which should be applied in accordance to Section 6.2.2. When explicit requirements for a certain Design Level do not exist in the corresponding code, the R value corresponding the next more demanding design level will be adopted.

6.4.1 CASE OF IRREGULAR STRUCTURES

The R values will be reduced multiplying the values of Table 6.4 by 0.75, without them being less than 1.0. The Design Level DL3 will always be applied in the following cases: (i) where exceptionally irregularities givenin Table 6.3 are presented, and (ii) in the Type I systems of limited redundance, such as: buildings with less than three resistant lines in one of its directions, discontinued columns, columns hinged at the base and floor systems without beams.

6.5 CLASSIFICATION ACCORDING TO THE STRUCTURE REGULARITY

Every building will be classified as regular or irregular according to the following definitions:

6.5.1 REGULAR STRUCTURE BUILDING

Buildings that are not included in any of the items of Section 6.5.2 will be considered regular.

TABLE 6.4

R REDUCTION FACTORS

DESIGN LEVEL	REINFORCED CONCRETE STRUCTURES				
	STRUCTURE TYPE (SECTION 6.3.1)				
	I	II	III	IIIa	IV
DL3	6.0	5.0	4.5	5.0	2.0
DL2	4.0	3.5	3.0	3.5	1.5
DL1	2.0	1.75	1.5	2.0	1.25

DESIGN LEVEL	STEEL STRUCTURES				
	STRUCTURE TYPE (SECTION 6.3.1)				
	I	II	III	IIIa	IV
DL3	6.0 (1)	5.0	4.0	6.0 (2)	2.0
DL2	4.5	4.0	-	-	1.5
DL1	2.5	2.25	2.0	-	1.25

(1) In frames with trussed beams a value of 5.0 will be used limited to buildings of no more than 30 meters tall.

(2) Where the fasten beam-column connection is flexible type 2, use 5.0.

DESIGN LEVEL	STEEL-CONCRETE MIXED STRUCTURES				
	STRUTURE TYPE (SECTION 6.3.1)				
	I	II	III	IIIa	IV
DL3	6.0	5.0	4.0	6.0 (1)	2.0
DL2	4.0	4.0	-	-	1.5
DL1	2.25	2.50	2.25	-	1.0

(1) For structural walls reinforced with steel planks and edge members of steel-concrete mixed section, use 5.0.

6.5.2 IRREGULAR STRUCTURE BUILDING

The building that in one or both of its main directions has one of the following characteristics is considered irregular:

a) **Vertical Irregularity**

a.1.) **Soft Story**

The lateral rigidity of any story, is less than: 0.70 times that of the upper story, or 0.80 times the average of the rigidities of the three upper stories. In the calculation of the rigidities the contribution of the partitions shall be included; in the case that its contribution is larger for the lower floor than for the upper floors, this could be omitted.

a.2) **Weak Story**

The lateral resistance of any story, is less than: 0.70 times that of the upper story, or 0.80 times the average of the resistance of the three upper stories. In the calculation of the resistance the contribution of the partitions will be included; in the case that its contribution is higher for the lower floor than for the upper floors, this could be omitted.

a.3) **Irregular mass distribution of one of the adjacent levels**

When the load of any level exceeds 1.3 times the load of one of the adjacent levels. The comparison with the last roof level of the building is exempted. To verify this, the load of the appendixes will be added to the weight of the level that supports them.

a.4) **Systematic mass increase with elevation**

The distribution of loads of the building grows systematically with height. For this verification the load of the appendixes will be added to the weight of the level that supports them.

a.5) **Variations in geometry of the structural system**

The horizontal dimension of the structural system in any floor exceeds 1.30 times that of the adjacent. The case of the last level is excluded.

a.6) Excesive slenderness

The ratio between the height of the building and the lesser dimension of the structure at the base level exceeds 4. Equally when this situation is presented in a significant portion of the structure.

a.7) Discontinuity in the plane of the system resistant to lateral loads

According to some of the following cases:

i) The horizontal disalignment of the axis of a vertical member, wall or column, between two consecutive floors, surpasses 1/3 of the horizontal dimension of the lower member in the direction of the disalignment.

ii) The width of the column or wall in a story presents a reduction that exceeds twenty per cent (20%) of the width of the column or wall in the story immediately above in the same horizontal direction.

iii) Columns or walls that do not continue once they reach a lower level different from the base level.

a.8) Lack of connection between vertical members.

Some of the vertical members, columns or walls, is not connected to the diaphragm of some level.

a.9) Short column effect

Marked reduction in the free length of columns, due to the effect of lateral restrictions such as walls, or other non-structural elements.

b) Plan Irregularities

b.1) Large eccentricity

At some level the eccentricity between the shear action line in any direction, and the rigidity center surpasses twenty per cent (20%) of the plan inertial radius.

b.2) High torsional risk

If any of the following situations is present at any level:

i) The torsional radius r_t in any direction is lower than fifty per cent (50%) of the inertial radius r.

ii) The eccentricity between the shear action line and the rigidity center of a given level surpasses thirty per cent (30%) of the value of the torsional radius r_t in any direction.

b.3) Non-orthogonal system

When an important portion of the seismic resistant system planes are not parallel to the main axis of such system.

b.4) Flexible diaphragm

i) When the rigidity in its plane is less than that of an equivalent reinforced concrete slab of 8 cm in width.

ii) When a significant number of levels have entrances with the shorter length exceeding forty per cent (40%) of the dimension of the lesser rectangle that inscribes the floor, measured in parallel to the direction of the entrance; or when the area of such entrances surpasses thirty per cent (30%) of the area of the aformentioned circumscribed rectangle.

iii) When the floors present a total area of internal openings that surpass twenty per cent (20%) of the raw areas of the floors.

iv) When prominent openings exist adjacent to the important quake resistant planes or, in general, when there is a lack of adequate connections with them.

v) When in any floor the length/width ratio of the smaller rectangle that inscribes such floor is more than 5.

CHAPTER 7

SEISMIC COEFFICIENT AND DESIGN SPECTRA

7.1.- SEISMIC COEFFICIENT FOR BUILDINGS

The seismic coefficient defined as Vo/W shall not be less than (α Ao) / R, where:

α = Importance factor (Table 6.1).

Ao = Horizontal acceleration coefficient for each zone (Table 4.1)

R = Reduction factor (Table 6.4).

Vo = Shear force at base level, obtained using the analysis procedures of Chapter 9.

W = Total weight of the building above the base level. For the determination of the total weight W, the percentages of the variable actions established in the COVENIN-MINDUR 2002 Code will have to be added to the permanent actions, as it is now indicated:

a) Liquid storage tanks: one hundred per cent (100%) of the service load, with the full tank.

b) Warehouses and storage places in general, where the load is of the permanent type such as libraries, files: one hundred per cent (100%) of the service load.

c) Public parking lots: in no case the adopted value will be less than fifty per cent (50%) of the variable service load prescribed in the respective codes, considering the parking lot to be full.

d) Buildings where there may be a concentration of public, larger than 200 persons such as: schools, theaters, comercial, institutional and industrial: fifty per cent (50%) of the service load.

e) Building stories, not included in (d) and parking lots different from (c): twenty five per cent (25%) of the variable service load.

f) Non-accessible roofs and terraces: zero per cent (0%) of the variable load.

7.2.- DESIGN SPECTRA

The ordinates A_d of the design spectra, are defined in function of its period T in the following way:

$$T < T^{+} \qquad Ad = \frac{\alpha\varphi Ao\left[1+\frac{T}{T^{+}}(\beta-1)\right]}{1+\left(\frac{T}{T^{+}}\right)^{c}(R-1)} \qquad (7.1)$$

$$T^{+} \leq T \leq T^{*} \qquad Ad = \frac{\alpha\varphi\beta\, Ao}{R} \qquad (7.2)$$

$$T > T^{*} \qquad Ad = \frac{\alpha\varphi\beta Ao}{R}\left(\frac{T^{*}}{T}\right)^{0.8} \qquad (7.3)$$

where:

A_d = Design spectral ordinate, expressed as a fraction of the acceleration of gravity.

α = Importance factor (Table 6.1).

A_o = Horizontal acceleration coefficient (Table 4.1).

φ = Horizontal acceleration coefficient correction factor (Table 5.1).

β = Average magnification factor (Table 7.1).

T_o = Value of the period from where the spectral ordinates have a constant value (sec) (Table7.1).

T* = Maximum value of the period in the interval where the normalized spectral values have a constant value (sec) (Table 7.1).

T^{+} = Characteristic variation period of ductile response (Table 7.2).

c = $\sqrt[4]{R/\beta}$

R = Response reduction factor (Article6.4)

TABLE 7.1
VALUES OF β, T_o y T^*

SPECTRAL SHAPE	β	T_o (sec)	T^* (sec)
S1	2.4	0.1	0.4
S2	2.6	0.2	0.8
S3	2.8	0.3	1.4
S4	3.0	0.4	2.0

TABLE 7.2
VALUES OF T^+ (1)

CASE	T^+ (sec)
$R < 5$	0.1 (R-1)
$R \geq 5$	0.4

(1) If $T^+ \geq T^*$, then $T^+ = T^*$
If $T^+ < T_o$, then $T^+ = T_o$

7.3.- SEISMIC FORCES IN COMPONENTS, APPENDIXES AND INSTALLATIONS.

7.3.1.- CRITERIA OF ANALYSIS AND DESIGN

The elements that do not form integral part of the building's structure, the smaller structures linked to it, its connections with the main structure, as well as the flexible elements that may oscillate vertically referred to in Section 7.3.2.1, will have to be designed to resist the seismic actions that result from the application of one of the following criteria:

a) The actions that result from applying the methods given in Chapter 9, assuming that they are part of the structure.

b) Those that are deduced from applying the actions prescribed in Section 7.3.2. In this case, for the seismic analysis of the main structure, the weight of the parts will be added to the weight of the corresponding level.

The actions over the architectural components, appendixes, mechanical and electrical systems that are considered vital or essential, will have to be evaluated taking into consideration the dynamic response of the system.

7.3.2.- SEISMIC COEFFICIENTS

The elements and parts of structures referred to in Section 7.3.1 will have to be designed to resist the horizontal seismic forces F_p in the most unfavorable direction, calculated according to the formula:

$$F_p = (F i/ Wi)\ C_p W_p \qquad (7.4)$$

where:

(Fi / Wi) = Ratio between the lateral force at level i where the element or the structural part and the weight of that same level is located. This ratio will not be less than $\alpha\ \varphi$ Ao.
Ao = Horizontal acceleration coefficient (Table 4.1).
α = Importance factor (Table 6.1).
C_p = Seismic coefficient for elements or parts of structures(Table 7.3).
W_p = Weight of the considered element.
φ = Correction factor of the of the horizontal acceleration coefficient (Table 5.1).
W_i = Weight of level i of the building.

TABLE 7.3

VALUES OF C_p

COMPONENTS, APPENDIXES AND INSTALLATIONS	C_p
Cantilevered substructures; engine room; tanks with its contents.	6/R
Walls: - Framed - Not framed - Cantilever	 1.0 1.5 4.0
Glass panels	2.0
Mechanical or electrical equipment	2.0
Fire hazard systems and/or electrical energy restitution	3.0
Connections of pre-cast elements; glass panels; other fragile elements	4.0
Vertical parapets; ornaments; publicity displays; chimneys	4.0

7.3.2.1.- HORIZONTAL CANTILEVERS

In the design of horizontal cantilevers of buildings, where the vertical component has not been incorporated in the analysis, it will be considered: (1) the combination of gravitational load indicated in the Code multiplied by $(1 + 0.67\ \alpha\ \beta\ \varphi\ Ao)$; (2) the net effect of a vertical load upwards equal to 0.2 (CP+CV).

CHAPTER 8

GENERAL REQUIREMENTS, ANALYSIS CRITERIA AND SAFETY VERIFICATION

8.1.- GENERAL

The seismic resistant system must be conceived in such way that the premature failure of a few elements does not threaten the stability of the building.

8.2.- ANALYSIS DIRECTIONS

The structures shall be analyzed in two orthogonal horizontal directions, applied in the most unfavorable ones, of the seismic resistant system. The effects of the earthquake and those of the gravitational actions, shall be combined according to Article 8.6.

8.3.- ANALYSIS REQUIREMENTS

The analysis of the effects of the seismic actions have to satisfy the following requirements:

8.3.1.- HYPOTHESIS FOR THE ANALYSIS

The effects of the seismic actions will be analyzed assuming linear elastic behavior according to the principles of the Theory of Structures.

The masses will be considered located in the corresponding centers of masses, incorporating the degrees of freedom and the inertial properties that are significant in the seismic response.

8.3.2.- DEFORMATION COMPATIBILITY

It will be verified that the deformations in the structural elements are compatible among them, without exceeding its resistance capacity.

8.3.3.- RIGIDITY OF THE DIAPHRAGMS

In the methods of analysis given in this Code it is assumed that the floors, roofs and its connections act as non-deformable diaphragms in its plane, and are designed to transmit the forces to the vertical elements of the seismic resistant system. If the diaphragms do not have the necessary rigidity, its flexibility will have to be considered in the analysis and design.

This diaphragms will have to be capable of transmitting the lateral forces F in its plane, which are obtained applying the methods given in Chapter 9, but in no case less than 0.15 times the weight of the floor or the roof.

Pre-cast floors or roofs shall be accepted as diaphragms, as long as the effectiveness of the joint between the different members can be proven.

8.4.- SUPERPOSITION OF TRANSLATIONAL AND TORSIONAL EFFECTS

The methods of analysis indicated in Chapter 9 superpose the translational and torsional effects due to the action of quakes, as well as accidental torsion, due to the rotational excitation of the ground and the uncertainties in the distribution of masses and rigidities.

8.5.- P-Δ EFFECTS

The P-Δ effects will be taken into account when at any level the stability coefficient θ_i, given in formula (8.1) exceeds 0.08

$$\theta_i = \frac{\delta_{ei} \sum_{j=i}^{N} W_j}{V_i \, (h_i - h_{i-1})} \qquad (8.1)$$

where:

δ_{ei} = Difference of the elastic lateral displacements between two consecutive levels, in its corresponding mass centers.

W_j = Weight of level j of the building (Article 7.1).

V_i = Design shear at level i.

hi = Height of level i

The structure will have to be re-dimentioned when at some level, the value θi exceeds θmax given by the equation (8.2)

$$\theta_{max} = \frac{0.5}{R} \leq 0.25 \qquad (8.2)$$

8.6.- COMBINATION OF ACTIONS

The structures and its elements will have to be designed according to one of the two combination criteria that are established as follows:

a) When needed, the complete quadratic combination, or the square root of the sum of the squares of the maxima corresponding to each direction of the quake. Its value will have to be inverted in sign in order to combine it with the effects of the loads.

b) The incorporation of one hundred per cent (100%) of the seismic effect in one main direction with thirty per cent (30%) of the seismic effect of the seismic actions in the orthogonal direction and vice-versa, with all the possible signs.

This combinations include the effects of the vertical acceleration. For the cantilevered horizontal elements Subsection 7.3.2.1 shall be applied.

The design of the foundations and retaining walls will be ruled according to that established in Chapter 11 of this Code.

8.7.- DEVICES TO REDUCE THE SEISMIC RESPONSE

The use of seismic isolation systems designed to reduce the seismic response, properly justified, is authorized.

CHAPTER 9

METHODS OF ANALYSIS

9.1.- CLASSIFICATION OF THE METHODS OF ANALYSIS

Each building shall be analyzed taking into consideration the translational and torsional effects, by one of the methods described below, which have been organized in ascending order of rigour.

9.1.1.- STATIC ANALYSIS

The translational effects are determined with the Equivalent Static Method (Article 9.3). The torsional effects are determined with the Equivalent Static Torsion Method (Article 9.5).

9.1.2.- IN PLANE DYNAMIC ANALYSIS

The translational effects are determined according to the Modal Superposition Method with one Degree of Freedom per Level (Article 9.4). The torsional effects are determined with the Equivalent Static Torsion Method (Article 9.5).

9.1.3.- SPATIAL DYNAMIC ANALYSIS

The translational and torsional effects are determined according to the Modal Superposition Method with Three Degrees of Freedom per Level (Article 9.6).

9.1.4.- SPATIAL DYNAMIC ANALYSIS WITH FLEXIBLE DIAPHRAGM

The translational and torsional effects are determined according to that indicated in Article 9.7 in which the flexibility of the diaphragm is included.

9.1.5.- OTHER METHODS OF ANALYSIS

In Article 9.8 an alternate Method to those described previously is presented, recommendable for the case of structures not typified in this Code.

In Article 9.9 an inelastic static analysis procedure is presented that can be used as an option along with the methods of analysis described previously.

9.2.- SELECTION OF THE METHOD

In Tables 9.1 and 9.2 the methods of analysis that at the least should be used are established, respectively for regular and irregular buildings, according to the classification of Article 6.5.

The prescribed methods can be substituted by other more refined according to the order given in Section 9.1.

TABLE 9.1

SELECTION OF THE METHOD OF ANALYSIS FOR REGULAR BUILDINGS

HEIGHT OF THE BUILDING	MINIMUM REQUIREMENT
Not exceeding 10 floors nor 30 meters	STATIC ANALYSIS(Section 9.1.1)
Exceeds 10 floors or 30 meters	IN PLANE DYNAMIC ANALYSIS (Section 9.1.2)

TABLE 9.2

SELECTION OF THE METHOD OF ANALYSIS FOR IRREGULAR BUILDINGS

TYPE OF IRREGULARITY		MINIMUM REQUIREMENT
VERTICAL	a.1; a.2; a.4; a.7; a.8	SPACIAL DYNAMIC ANALYSIS(Section 9.1.3)
	a.3; a.5; a.6	IN PLANE DYNAMIC ANALYSIS(Section 9.1.2)
ON PLAN	b.1; b.2; b.3	SPACIAL DYNAMIC ANALYSIS (Section 9.1.3)
	b.4	SPACIAL DYNAMIC ANALYSIS WITH FLEXIBLE DIAGRAM(Section 9.1.4)

9.3.- EQUIVALENT STATIC METHOD

9.3.1.- BASE SHEAR FORCE

The base shear force V_0, in each analysis direction, is determined according to the expression:

$$V_0 = \mu \, A_d \, W \qquad (9.1)$$

where:

A_d = Ordinate of the design spectrum defined in Article 7.2 for the period T given in Section 9.3.2.

W = Total weight of the building above the base level (Article 7.1).

μ = The larger of the following values:

$$\mu = 1.4\left[\frac{N+9}{2N+12}\right] \qquad (9.2)$$

$$\mu = 0.80 + \frac{1}{20}\left[\frac{T}{T^*} - 1\right] \qquad (9.3)$$

where:

N = Number of levels.

T = Fundamental period.

T* = Period given in Table 7.1.

The value $\frac{V_0}{W}$ should be greater than or equal to the minimum seismic coefficient established in Article 7.1

9.3.2.- FUNDAMENTAL PERIOD

9.3.2.1.- In each direction of the analysis the fundamental period T will be calculated according to what is established in the following formula:

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{N} W_i (\delta_{ei})^2}{g \sum_{i=1}^{N} Q_i \, \delta_{ei}}} \qquad (9.4)$$

Where:

Q_i = Lateral force applied in the center of masses of level i of the building and given by:

$$Q_i = W \frac{W_i h_i}{\sum_{j=1}^{N} W_j h_j} \qquad (9.5)$$

W = Total weight of the building.
W_i = Weight of level i.
h_i = Height of the level measured from the base.
δ_{ei} = Lateral elastic displacement of level i, for the action of the Q_i lateral loads.
N = Number of levels of the building.
g = Acceleration of gravity.

The T value of the fundamental period calculated according to formula 9.4 will not exceed 1.4 T_a, where T_a is given in Section 9.3.2.2.

9.3.2.2.- As an alternative to the method described in Section 9.3.2.1, the fundamental period T may be taken equal to T_a, obtained from the following expressions:

a) **For Type I buildings**

$$T_a = C_t \, h_n^{0.75} \qquad (9.6)$$

where:

C_t = 0.07 for reinforced concrete or steel-concrete mixed buildings.

C_t = 0.08 for steel buildings.

h_n = Height of the building measured from the last level, to the first level with totally or partially restricted displacements.

b) For Type II, III and IV buildings

$$T_a = 0.05\ h_n^{0.75} \qquad (9.7)$$

9.3.3.- VERTICAL DISTRIBUTION OF THE DESIGN FORCES DUE TO TRANSLATIONAL EFECTS

The lateral design forces in each level and for each direction of the analysis will be obtained by distributing vertically the base shear force V_o, determined with formula (9.1), according to the following expression:

$$V_o = F_t + \sum_{i=1}^{N} F_i \qquad (9.8)$$

where:

F_t = Lateral force concentrated in level N calculated according to the following expression:

$$F_t = \left(0.06\ \frac{T}{T^*} - 0.02\right) V_o \qquad (9.9)$$

And limited between the following boundaries:

$$0.04\ V_o \leq F_t \leq 0.10\ V_o \qquad (9.10)$$

F_i = Lateral force corresponding to level i, calculated according to the following formula:

$$F_i = (V_o - F_t)\ \frac{W_i\ h_i}{\sum_{j=1}^{N} W_j\ h_j} \qquad (9.11)$$

W_j = Weight of level j of the building.

h_j = Average height from the base to level j of the building.

Forces F_i and F_t will be applied in the mass centers of the respective levels.

When structures such as water tanks, machinery rooms, advertisement placards and similar ones are placed on the last level N, the criteria of Article 7.3 will be applied.

9.4.- MODAL SUPERPOSITION METHOD WITH ONE DEGREE OF FREEDOM PER LEVEL

9.4.1.- MATHEMATICAL MODEL

For the application of this method, the building has to be modeled as a system of masses concentrated in each level, each one having one degree of freedom corresponding to the lateral displacement in the considered direction.

9.4.2.- MODES

The modal forms and its corresponding vibration periods in the analyzed direction are calculated using the elastic rigidities and the masses of the systems.

9.4.3.- ANALYSIS

The participation factor γ_j of each vibration mode is given by:

$$\gamma_j = \frac{\sum_{K=1}^{N} M_k\ \Phi_{kj}}{\sum_{K=1}^{N} M_k\ \Phi_{kj}^2} \qquad (9.12)$$

The maximum displacement u_{kj} and the lateral force F_{kj} at level "k" of the mode "j", are given by:

$$u_{kj} = \Phi_{kj} \cdot \gamma_j \cdot A_{dj} \cdot g \cdot \left[\frac{T_j}{2\pi}\right]^2 \qquad (9.13)$$

$$Fkj = M_k \,.\, \Phi_{kj} \,.\, \gamma_j \,.\, A_{dj} \,.\, g \qquad (9.14)$$

The shear force V_{oj} at the base of the building, in the mode "j", is given by:

$$V_{oj} = \beta_j \,.\, M \,.\, A_{dj} \,.\, g \qquad (9.15)$$

$$\beta_j = \frac{1}{M} \frac{\left[\sum_{K=1}^{N} M_k \Phi_{kj}\right]^2}{\sum_{K=1}^{N} M_k \Phi_{kj}^2} \qquad (9.16)$$

Where:

- Φ_{kj} = Modal coordinate of level "k" in the mode "j".
- M_k = Mass of level k.
- N = Total number of floors.
- A_{dj} = Ordinate of the design spectrum for the period Tj (Article 7.2).
- T_j = Vibration period of mode "j".
- g = Acceleration of gravity.
- M = Total mass of the building = W/g (Article 7.1).
- β_j = Fraction of the total mass of the building, or participating mass, associated with the response in mode "j".

9.4.4.- NUMBER OF VIBRATION MODES

In each direction, the analysis should at least incorporate the number of modes N_l that are now indicated below:

a) for buildings with less than 20 levels:

$$N_l = \frac{1}{2}\left(\frac{T1}{T^*} - 1.5\right) + 3 \geq 3 \qquad (9.17)$$

b) for buildings with 20 levels or more:

$$N_l = \frac{2}{3}\left(\frac{T1}{T^*} - 1.5\right) + 4 \geq 4 \qquad (9.18)$$

where:

T_l = period of the fundamental mode.

The values N_l should be rounded up to the immediately superior integer number. For structures with less than 3 floors, the number of modes to be incorporated is equal to the number of floors.

9.4.5.- MODAL COMBINATION

The base shear force and the shear force in each level will be determined by the combination of the respective modal values. The combination will take place taking the square root of the sum of the squares of each modal value or by the complete quadratic combination. The concentrated forces at each level, will be obtained out of the shear forces; they will be applied on the centers of mass.

9.4.6.- BASE SHEAR CONTROL AND DESIGN VALUES

The base shear V_o will have to be compared with the one calculated according to Section 9.3.1 with a period $T = 1.6\, T_a$, which will be noted here by $\overline{V_o}$ .When V_o is less than $\overline{V_o}$ the design values will have to be multiplied by $\overline{V_o} / V_o$. The V_o / W design ratio shall not be less than the minimum seismic quotient given in Article 7.1

The P-Δ effects will be considered according to Article 8.5 in order to obtain the eventual increments of shear forces, displacements and drifts.

Finally the torsional effects will be considered according to Article 9.5 and its effect will be added to the results of the previous analysis.

The design values will be multiplied by factor C which incorporates de uncertainties of the determination of the building's periods. Factor C is the larger value between the ratios Ad^- / Ad^o y Ad^+ / Ad^o, being:

- Ad^o = ordinate of the design spectrum for period $\overline{T}$.
- Ad^- = ordinate of the design spectrum for period 0.9 $\overline{T}$.
- Ad^+ = ordinate of the design spectrum for period 1.1 $\overline{T}$.
- $\overline{T}$ = period associated to the mode that has the greatest contribution in the base shear force.

9.5.- METHOD OF THE EQUIVALENT STATIC TORSION

In each level and in each direction the effects of the indicated torsional moments are incorporated, added to the shear forces applied in the rigidity centers. For each resistant element the most unfavorable solicitations will be selected derived from the combinations of shear force and the different indicated torsional moments.

The torsional moments will be obtained through the following formulas:

$$M_{ti} = V_i\,(\tau e_i + 0.1\,B_i) \qquad (9.19)$$

$$M_{ti} = V_i\,(\tau' e_i - 0.1\,B_i) \qquad (9.20)$$

where:

V_i = Design shear force in level i for the analyzed direction, calculated according to Article 9.3 or Article 9.4.

e_i = Static eccentricity at level i, between the center of rigidity and the line of action of the shear in the analyzed direction. It will always be taken as positive in formulae (9.19) and (9.20).

B_i = Width of the floor in the normal direction to the analysed direction.

τ = Torsional dynamic amplification factor for the considered direction.

τ' = Design control factor of the most rigid zone of the level, for the considered direction.

The modification factors of eccentricity, for each direction, can be calculated according to the following expressions:

$\tau = 1 + [6.25 - 20\,\varepsilon\,\Omega]\Omega^4$	For $0.5 \le \Omega \le 1$
$\tau = 1.5 + [5.75 - 20\varepsilon(2-\Omega)](2-\Omega)^4$	For $1 \le \Omega \le 2$
$\tau = 1.5$	For $2 \le \Omega$
$\tau' = 6(\Omega - 1) - 0.6$	With $-1 \le \tau' \le 1$

where:

ε = Representative value of the ratio e / r but not larger than 0.2

Ω = Representative value of the ratio r_t / r but not less than 0.5

e = Representative value of the eccentricities between the center of rigidity and the shear action line of the building levels in the analyzed direction.

r = Representative value of the inertial radius of the building levels.

r_t = Torsional radius in the considered direction, representative of the group of levels of the building.

Alternatively the values of τ and τ' can be determined through a dynamic analysis, with 3 degrees of freedom, of the system of one level with the representative values previously indicated.

When representative values of e, r or r_t can not be established, because they reach too different values between the different levels, the Method of Article 9.6 will have to be applied. It will equally be done that way when the limitations of ε and Ω are exceeded.

9.6.- MODAL SUPERPOSITION METHOD WITH THREE DEGREES OF FREEDOM PER LEVEL

9.6.1.- GENERALITIES

This method takes into account the coupling of the translational and torsional vibrations of the building and considers three degrees of freedom per level.

9.6.2.- DESIGN VALUES

9.6.2.1.- DYNAMIC RESPONSE

The minimum number of vibration modes (N_3) to be used in the dynamic analysis, shall be the larger of the two following values:

i) $N_3 = 3N_1$, where N_1 is given by the formulas (9.17) and (9.18) of Section 9.4.4.

ii) N_3 = Number of modes that warranties that the sum of the participating masses of the first N modes exceeds ninety per cent (90%) of the total mass of the building, for each one of the analyzed directions.

The maximum of any dynamic response value of interest due to the action of a seismic component X (R_x) ó Y (R_y), is obtained combining the modal values according to the criteria of the complete quadratic combination, that takes into account the coupling between modes of close frequency.

In each direction, the base shear V_o obtained from the modal combination will have to be compared to the one calculated according to Section 9.3.1 with a period T = 1.6 T_a, which is noted here by $\overline{V_o}$. When V_o is less than $\overline{V_o}$, the design values will have to be multiplied by $\overline{V_o}/V_o$. The V_o/W design ratio will not be less than the minimum seismic coefficient given in Article 7.1. The P-Δ effects are incorporated in a similar way as is established in Section 9.4.6.

9.6.2.2.- ADDITIONAL TORSION

The effects of the rotational component of the ground and that of the uncertainties in the location of mass and rigidity centers, are included in the design adding to the results of the dynamic analysis, the most unfavorable solicitations that result from statically applying the following torsional moments:

For X direction:

$$Mt_{kx} = \pm V_{kx}\left(0.10B_{ky}\right) \qquad (9.21)$$

For Y direction:

$$Mt_{ky} = \pm V_{ky}\left(0.10B_{kx}\right) \qquad (9.22)$$

where:

- V_{kx} = Shear force at level k of the building, in the X direction due to the X seismic component.
- V_{ky} = Shear force at level k of the building, in the Y direction due to the Y seismic component
- B_{kx} = Larger horizontal dimension of the building in the X direction, at level k.
- B_{ky} = Larger horizontal dimension of the building in the Y direction, at level k
- Mt_{kx} = Additional torsional moment to be applied at level k, for the case of quake in the X direction.
- Mt_{ky} = Additional torsional moment to be applied at level k, for the case of quake in the Y direction

The torsional moment at any level will not be less than any of the upper levels.

For the X seismic component, the static application of the level torques obtained from Mt_{kx}, lead to a generic solicitation that is noted by R_{tx}. For the Y seismic component, the static application of the level torques obtained from Mt_{ky}, lead to a generic solicitation that is noted by R_{ty}.

9.6.2.3.- COMBINATION OF THE DYNAMIC RESPONSE AND THE ADDITIONAL TORSION

To the absolute values of the dynamic responses $|R_x|$ *and* $|R_y|$ obtained from applying Subsection 9.6.2.1, for the X and Y seismic components, respectively, the absolute value obtained from the solicitations resulting from the application of additional torsion is added, $|R_{tx}|$ *and* $|R_{ty}|$ obtained from applying Subsection 9.6.2.2, to determine the complete seismic solicitations in each direction R_x^* and R_y^*, in each member or resistant level.

For X direction:

$$R_x^* = |R_x| + |R_{tx}| \qquad (9.23)$$

For Y direction:

$$R_y^* = |R_y| + |R_{ty}| \qquad (9.24)$$

9.7.- DYNAMIC ANALYSIS METHOD WITH FLEXIBLE DIAPHRAGM

9.7.1.- FIELD OF APPLICATION

In this Section an alternative analysis is presented for the case of buildings that have the plan irregularities defined as b.4 in Section 6.5.2, or where the mechanical characteristics of the floor system do not warranty an equivalent behavior as an infinitely rigid diaphragm.

9.7.2.- MATHEMATICAL MODEL

The floor system will be modeled through techniques of finite elements or similar. The type and number of elements to be used will be the one to represent adequately the flexibility of the floor taking into account its geometrical characteristics, connectivity and rigidity. The degrees of freedom of each element must be defined in the directions associated to the displacements in its own plane.

The mass of each floor will be distributed among the diverse elements that form it, simulating the real distribution of its mass. The distribution of mass shall correspond to the total mass of the floor and its rotational inertia.

9.7.3.- ANALYSIS

9.7.3.1.- GENERAL

The building will be analyzed through methods of dynamic analysis under the action of the two horizontal components of the earthquake given by the design spectrum specified in Article 7.2.

9.7.3.2.- NUMBER OF MODES

The number of vibration modes to be used in the analysis is the one that warranties that the sum of the participating masses of the modes in each one of the horizontal directions of the seismic action, exceeds ninety per cent (90%) of the total mass of the building.

9.7.3.3.- MODAL COMBINATION

The combination of the maximal response values in each mode, will be done according to the criteria of the complete quadratic combination just as it is indicated in the Subsection 9.6.2.1, for each direction of the seismic action.

9.7.4.- ADDITIONAL TORSION

The effects of the accidental eccentricities and that of the rotational component of the ground are included in the design considering the following four additional cases of dynamic analysis. In each case, the distribution of mass in each level will be modified, in a gradual manner, such that the mass center is displaced to a distance dx in the X direction and a distance dy in the Y direction, given by:

1) + dx ; + dy
2) + dx ; - dy
3) - dx ; + dy
4) - dx ; - dy

where:

dx = 0.03 Bx
dy = 0.03 By

Bx = The largest horizontal dimension in each level in the X direction.

By = The largest horizontal dimension in each level in the Y direction.

In each of these cases, the centers of mass for all the levels will be displaced in the same magnitude and in the same direction and sense.

9.7.5.- COMBINATION OF THE DYNAMIC RESPONSE AND THE ADDITIONAL TORSION

The complete seismic solicitations are the most unfavorable of those obtained when comparing the results of the four analysis described in Section 9.7.4 with the results of the dynamic analysis of the building without changing the position of the centers of mass.

9.7.6.- CONTROL OF MINIMUM SHEAR

The base shear V_{ox} and V_{oy} in each one of the main directions X, Y of the building, will have to be compared to those calculated according to Section 9.3.1 with a T period of T=1.6 Ta , which are noted here by $\bar{V}_{ox}$ and $\bar{V}_{oy}$, respectively. The complete seismic solicitations and the displacements for each direction of the seismic action will have to be multiplied by the factors $\left(\bar{V}_{ox} / \bar{v}_{ox}\right)$ and $\left(\bar{V}_{oy} / \bar{V}_{oy}\right)$ respectively, which shall not be less than 1.0.

The design ratio Vo/W will not be less than the minimum seismic coefficient given in Article 7.1 for both directions X and Y of the building.

9.7.7.- P-Δ EFFECT

With the floor solicitations and the displacements obtained in Section 9.7.6, the compliance of what is established in Section 8.5 should be verified .

9.8.- DYNAMIC ANALYSIS METHOD WITH ACCELEROGRAMS

9.8.1.- GENERAL

The method is of general application. In particular it is required for structures not typified among the Structural Types defined in Article 6.3. In these structures it is recommended to do an inelastic analysis that supplies realistic values for the demands of ductility of the structure and its components. It may be used in substitution of the methods of analysis based on an elastic model of the building, described in Articles 9.2 to 9.7.

9.8.2.- STRUCTURAL MODEL

The structure will be modeled considering an inelastic behavior representative of its mechanical characteristics. The restitution diagram adopted will have to be backed-up by experimental information.

9.8.3.- INELASTIC ANALYSIS

The structure will be analyzed through procedures of direct integration (step by step) for accelerograms representative of the seismic action expected at the site. If the analysis includes the simultaneous action of the two horizontal components of the quake, the pair of accelerograms to be used will have to have an adequate correlation coefficient.

For the analysis at least four accelerograms or pairs of accelerograms shall be used. The probable dynamic response will be obtained by averaging the obtained responses for all the accelerograms of the group.

The P-Δ effect shall be included in the analysis.

9.8.4.- ACCELEROGRAMS

The seismic movements to be used in the analysis may be registered accelerograms or simulated ones through known procedures.

The average elastic spectrum of the accelerograms of the group will have to conservatively approach the design spectrum given in Article 7.2 for the value R=1.0, in the range of the own periods of the structure.

9.8.5.- RESULTS EVALUATION

It will have to be verified that the structure and its members comply with the general requirements established in Chapter 3, as well as the allowable drifts and displacements given in Chapter 10.

9.9.- INELASTIC ESTATIC ANALYSIS PROCEDURE

This procedure constitutes an adequate option to be used in conjunction with the methods of analysis given in Article 9.3, in order to obtain information about the failure mechanisms, the local and global ductility demands, and the identification of critical zones. The distribution of lateral static loads to be applied will be obtained with the application of the method of Article 9.3, applied in a monotonic and increasing way, until reaching the failure or ultimate state of the structure.

The structure shall be modeled considering an inelastic behavior representative of its mechanical characteristics.

CHAPTER 10

CONTROL OF DISPLACEMENTS

10.1.- TOTAL LATERAL DISPLACEMENT

The total lateral displacement Δ_i at level i will be calculated as:

$$\Delta_i = 0.8\, R\, \Delta_{ei} \qquad (10.1)$$

where:

- R = Reduction factor given in Article 6.4.
- Δ_{ei} = Lateral displacement of level i calculated for the design forces, assuming that the structure behaves elastically, including translational and torsional effects at that level, as well as P-Δ effects.

The drift δ_i is the difference of the total lateral displacements between two consecutive levels:

$$\delta_i = \Delta_i - \Delta_{i-1} \qquad (10.2)$$

10.2.- LIMIT VALUES

The limits given in Table 10.1 shall be checked at each resistance line and at the farmost points of the rigidity center. The ratio that follows, will not exceed at any level the values given in Table 10.1:

$$\frac{\delta i}{(hi - h_{i-1})} \qquad (10.3)$$

where:

$(h_i - h_{i-1})$ = Separation of two consecutive levels.

TABLE 10.1

LIMIT VALUES OF: $\frac{\delta i}{(hi - h_{i-1})}$			
TYPE AND DISPOSITION OF NON-STRUCTURAL ELEMENTS	BUILDINGS		
	GROUP A	GROUP B1	GROUP B2
Susceptible of suffering damages because of deformations of the structure.	0.012	0.015	0.018
Not susceptible of suffering damages because of deformations of the structure.	0.016	0.020	0.024

10.3.- MINIMUM SEPARATIONS

10.3.1.- BOUNDARIES

Any building will have to be separated from its boundary a distance larger than:

$$\left(\frac{R+1}{2}\right) \Delta_{eN} \qquad (10.4)$$

Where:

Δ_{eN} = Maximum elastic lateral displacement of the last level in the considered direction, but not less than 3.5 cm in the first 6 meters plus four per thousand (4 ‰) of the height that exceeds the latter.

10.3.2.- ADJACENT BUILDINGS

To determine the separation between adjacent buildings the values obtained from the criteria given in Section 10.3.1 will be used. The minimum separation between adjacent buildings will be equal to the square root of the sum of the squares of the values obtained for each building..

10.3.3.- BUILDINGS IN CONTACT

Two adjacent buildings may be in contact provided that all the slabs are at the same level and it is proven that its interaction does not bring about unfavorable effects.

CHAPTER 11

FOUNDATIONS, RETAINING WALLS AND SLOPES

11.1.- VALIDITY AND SCOPE

This Chapter contains the minimum requirements for the seismic resistant design of the infrastructure of buildings, constituted by the foundations and its respective tiebraces. Additionally, the prescriptions for the design of retaining walls are included, and the criteria to evaluate the stability of slopes such as natural hillsides, filled embankments and areas whose surface is moderately or softly tilted.

11.2.- GEOTECNICAL PARAMETERS AND METHODS OF ANALYSIS

11.2.1.- GEOTECNICAL PARAMETERS

In case that cohesive soils are present at the site, whose resistance degrades due to seismic action, the degraded resistance due to the effect of the cyclic load has to be determined and those properties applied for the evaluation of the static stability and the deformations immediately after the earthquake. This will be called postseismic analysis.

When discontinuities exist in the structure of the soil or of the rock, such as planes of stratification, cracking, planes, foliations or of any other nature, the value of the resistance to be used for the total mass shall consider the presence of such discontinuities. Under this conditions, it is also necessary to verify the stability for those failure mechanisms controlled by the resistance along such discontinuities.

11.2.2.- METHODS OF ANALYSIS

For the analysis of foundations, retaining walls and slopes, pseudostatic methods can be used based on allowable displacements, for the determination of equilibrium of forces and/or moments, as well as in the comparison of stresses. Alternatively, methods of analysis can be used coupling stress-strain relations with accelerograms. The prescriptions for the pseudostatic methods are given in Articles 11.4, 11.5 and 11.6, while those that correspond to coupling stress-strain relations with accelerograms are given in Article 11.7.

11.3.- REQUIREMENTS FOR THE STRUCTURAL DESIGN

The structural components of the foundations and the retaining walls shall be designed respecting the corresponding design level and following the prescriptions of the COVENIN-MINDUR 1753 Code. The structural design of the piles will be complemented with what is specified in Section 11.4.6 of the present Code.

11.4.- FOUNDATIONS

11.4.1.- VERIFICATION OF THE FOUNDATION SYSTEM

The design of the foundation system shall ensure that the structural resistance of each component is capable of supporting the seismic forces and moments transmitted by the superstructure, that the ground be able to support the transferred actions through the foundations and that the joint ground-foundation stiffness is enough not to suffer excessive displacements that impair the functionability of the foundation or the superstructure. It is understood that the seismic resistant requirements expressed in this Section will have to be satisfied, in addition to those necessary to support other loads that the foundation could be subjected to during the duration of its useful life.

When the use of systems of mixed foundations, and/or of very unequal rigidities turns to be necessary, its behavior under seismic action will have to be verified, using an adequate model for the foundation system used.

Where the foundation conditions are not homogeneous because of horizontal or vertical variability of the geotechnical profile, the support capacity and the allowable differential settlements between the components of the foundation system shall be verified.

11.4.2.- TIE BEAMS

The foundations will be tied among them, in two directions, preferably orthogonal, with structural members capable of withstanding axially the larger load in the columns that are connected by the tie beams multiplied by a coefficient equal to $(\alpha\ \varphi\ A_0)/3$, but not less than ten per cent (10%) of such load.

In case that the tie beams form part of the load resisting slab at its level, they will be designed considering all the actions, fulfilling also the previous requirements.

11.4.3.- PEDESTALS

The pedestals shall be designed for the solicitations resulting from the analysis. The minimum reinforcement of each pedestal will follow the COVENIN-MINDUR 1753 Code.

11.4.4.- SUPERPOSITION EFFECTS

The load cases to consider for the analysis of shallow and pile foundations are defined in Table 11.1:

TABLE 11.1 SUPERPOSITION OF EFFECTS.

ANALYSIS CASE	Q
With seismic solicitations	1.1 CP + CV ± S 0.9 CV ± S
Postseismic	1.1CP + CV

where: Q = Solicitation for the verification of the load bearing capacity of the foundations.

CP = Effect due to permanent loads.

CV = Effect due to variable loads.

S = Effect due to the seismic actions, including the effect of the vertical component.

11.4.5.- SHALLOW FOUNDATIONS

For the safety verification of a foundation under seismic actions, it will be allowed that the maximum stresses transmitted to the ground can be larger than those admissible under static loads.

It is necessary to verify the compatibility of the expected differential settlements as a consequence of the seismic action, with those allowed in the static case, particularly, when the foundation base is on non-cohesive soils.

Under the most unfavorable conditions associated to the seismic actions, partial uplift is accepted provided it does not exceed 25% of the total supporting area. With the exception of buildings of only one level corresponding groups B and C, the use of shallow foundations is not allowed when the analysis confirm the existence of cohesive soils with Sensitivity (Se) greater than 2 (See Subsection 11.4.5.1) or non-cohesive liquefiable soils. For these exceptional cases it must be verified that the shallow foundation is capable of transmitting the loads successfully and keep the settlements within the levels that do not compromise the performance of the building in spite of the potential shear loss resistance of the soil.

For any other building, the use of shallow foundations will only be allowed in potentially liquefiable soils if they are properly treated to prevent the loss of shear resistance and, also, prove that the differential settlements do not compromise the performance of the structure.

11.4.5.1.- VERIFICATION OF THE BEARING CAPACITY

The maximum compression stresses transferred to the ground (q) due to load combinations given in Table 11.1, should comply with the following:

a) When the ultimate bearing capacity is used:

$$q \leq (q_{ult}/Se) * 0.6 \qquad (11.1a)$$

b) When the allowable bearing capacity is used:

$$q \leq 2(q_{adm}/Se) * 0.6 \qquad (11.1b)$$

where: q = Maximum compression stresses imposed by the foundation to the ground for the load cases given in Section 11.4.4.

q_{ult}= Ultimate ground bearing capacity using static load capacity factors. For non-cohesive liquefiable soils, calculate q_{ult} using the undrained residual resistance S_{us} from the soil as if it was cohesive ground with internal angle of friction equal to zero.

q_{adm}= Allowable bearing capacity for static loads. For non-cohesive liquefiable soils, calculate q_{adm} using the soil residual resistance S_{us} as if it were cohesive ground with internal angle of friction equal to zero.

Se= Sensitivity of the cohesive soil. Use Se=1 when the seismic actions for cohesive or non-cohesive grounds are included. Also use Se=1 when non-cohesive liquefiable soils are analyzed.

11.4.5.2.- VERIFICATION OF SLIDE STABILITY

It shall be verified that in the effective contact area between the foundation and the ground, the shear force, V, induced by the solicitations calculated according to Section 11.4.4, does not exceed the slide resisting force given by the formula 11.2:

$$V \leq (\mu_f N_a + cA)\, 0.8 \qquad (11.2)$$

where: μ_f = Ground-foundation friction coefficient.

N_a= Force normal to the contact area, acting simultaneously with V.

c = Adhesion between the ground and the foundation.

A= Contact area of the foundation.

11.4.6.- PILE FOUNDATIONS

In general, the requirements of this Section are directed to reinforced, pre-stressed or post-stressed concrete piles, and also steel or similar. Wooden piles will be considered acceptable, as long as a seismic resistant behavior is ensured according to what is established in this Code.

For pile design Section 11.4.4 will have to be satisfied with relation to the analysis that considers the seismic actions and the postseismic case.

11.4.6.1.- PILE-CAPS

For piles, isolated or in group, pile-caps interconnected through tie beams shall be used. The dimensioning and detailing of this elements should ensure that the pile develops its resistance capacity in the connection. Confinement reinforcement required on the upper part of piles, will be extended within the longitude of the pile-cap (See Section 11.4.7).

11.4.6.2.- CONSIDERATIONS ABOUT CONSTRUCTION METHODS

The construction method shall be selected taking into consideration the ground characteristics, in order to avoid damages or discontinuities in the pile during the process of installation.

The influence of the construction method shall be considered in the load bearing capacity of the pile and in the rigidity of the ground-pile group. Consideration shall also be given to residual stresses in the pile, associated with the construction method, if these exist.

11.4.6.3.- AXIAL LOAD CAPACITY

For the determination of the axial load capacity of the pile, regardless if it is by compression or by tension, the following expression will have to be verified:

$$Q \leq Q_{ult}\, \phi \qquad (11.3)$$

where: Q = Maximum compression or tension load obtained from the load cases established in Section 11.4.4.

Q_{ult} = Ultimate bearing capacity to compression or tension of the ground-pile system.

ϕ = Reduction factor according to what is established in Subsection 11.4.6.4.

In piles constituted by sections, the maximum tension force shall not exceed seventy five per cent (75%) of the connections' resistance.

11.4.6.4.- REDUCTION FACTORS

The resistance reduction factors, to tension as well as to compression, are given in Table 11.2. In it, there is a distinction for cases in which load tests have been made to verify the real capacity.

TABLE 11.2

MAXIMUM RESISTANCE REDUCTION FACTORS , ϕ, FOR AXIAL LOAD CAPACITY OF PILES

		Case of analysis	
Type of load	Load test	With seismic solicitations	Post-seismic
Compression	Yes	0.9	0.75
	No	0.7	0.75
Tension	Yes	0.9	0.75
	No	0.6	0.75

11.4.6.5.- GROUP EFFECT

In groups of piles which separation center to center is smaller than 8 times the diameter of a pile, the reduction in the total load capacity of the group shall be evaluated, refered to the calculated sum of the capacities of the individual piles. The variation in the rigidity of the ground in lateral and axial directions should also be considered.

11.4.7.- DESIGN REQUIREMENTS FOR FOUNDATIONS WITH PILES

The design of the pile-caps and that of the connections between them and the pile should ensure that the pile develops its maximum resistance capacity. On top of that, in the cases where it is required to ensure a ductile zone in the upper part of the pile, the design of such zone will be made with the same considerations that if it were a column. In this cases the pile-cap and the connection should be designed to ensure that ductile behavior is reached.

The design of the piles shall be performed based on the state of strains imposed by the seismic actions, considering the interaction between the ground and the piles, under axial and lateral forces.

11.5.- RETAINING WALLS

11.5.1.- CLASSIFICATION

In order to verify its stability, retaining walls will be classified into the following types:

a) Gravity
b) Cantilever
c) Anchored
d) Reinforced soil

11.5.2.- PSEUDOSTATIC ANALYSIS OF THE RETAINING WALLS

When methods based in the equilibrium of forces are used, the dynamic force will have to be calculated considering the behavior between the wall and the retained material. Additionally, if the material behind the wall is saturated during the service conditions, the hydrodynamic effect shall be included in the analysis.

When allowable displacement methods are used, there should be representative estimates of the ground maximum velocities.

11.5.3.- SUPERPOSITION OF EFFECTS

The load cases to be considered with the methods of analysis that use the equilibrium of forces are defined in Table 11.3.

TABLE 11.3
SUPERPOSITION OF EFFECTS

CASE OF ANALYSIS	Q
With seismic solicitations	1.1 CP + 1.0 CV + ED ± S 0.9 CP + ED ± S
Postseismic	1.1CP + CV

Where: Q= Total effect for the capacity verification.
CP= Effect due to permanent loads.
CV= Effect due to variable loads.
ED= Dynamic lateral force of the soil wedge movilized behind the wall.
S= Effect due to the seismic actions different to the soil thrust, but considering the inertia forces of the wall, calculated with a seismic coefficient equal to $0.75\ \varphi\ A_o$.

11.5.4.- STABILITY VERIFICATION

The stability of retaining walls will follow what is established in Table 11.4.

TABLE 11.4

CRITERIA FOR THE SEISMIC VERIFICATION OF RETAINING WALLS

TYPE OF WALL	STABILITY, GLOBAL BEARING CAPACITY AND SLIDING	STABILITY TO SLIDING	ANCHORING ELEMENTS	INTERNAL STABILITY
Gravity	Yes	Yes	NA	Na
Cantilever	Yes	Yes	NA	Na
Anchorage	Yes	Yes	Yes	Na
Reinforced Soil	Yes	Yes	NA	Yes

Na: Not apply

11.5.4.1.- REQUIREMENTS FOR THE VERIFICATION OF THE GLOBAL STABILITY, THE BEARING CAPACITY AND SLIDING

The verification of the global stability shall be done according to what is prescribed in Article 11.6.

The verification of the bearing capacity of the foundation ground under the wall and the sliding shall be done for the combinations given in Table 11.3, according to what has been established in Subsections 11.4.5.1 and 11.4.5.2. Equally, in case that the retaining wall is founded on piles, the requirements established in Section 11.4.6 will have to be satisfied.

11.5.4.2.- REQUIREMENTS FOR THE VERIFICATION OF THE STABILITY TO OVERTURNING

For the verification of the stability to overturning the combinations of Table 11.3 will be used, according to the following expression:

$$\Sigma M_a \leq 0.7 \Sigma M_r \qquad (11.4)$$

where: ΣM_a = Sum of acting moments stemming from the load cases established in Section 11.5.3

ΣM_r = Sum of resistant moments.

11.6.- SLOPE STABILITY

The slope stability shall be verified obligatorily in the following cases:

a) When the regional and local geological conditions indicate potential instability of the zone.

b) When the area is affected by modifications of its original topography, including fillings, specially where there are zones with high drainage lines and fill-up bodies not confined on edges of hillsides.

c) Places with unfavorable geotechnical conditions like those with high pore pressures or grounds with degrading resistance during the seismic action.

d) When the failure surface might be controlled by geological discontinuities, in which case fault potential surfaces along such discontinuities should be considered.

11.6.1.- PSEUDOSTATIC ANALYSIS OF SLOPED GROUNDS

For the case of the pseudostatic methods of inertial balance, the maximum horizontal inertia force will be calculated with a seismic coefficient not less than $0.5\varphi A_0$, acting in the most unfavorable direction. Likewise, the undegraded shear resistance will be used.

When a reduction of the ground shear resistance occurs, the post-seismic stability of the sloped ground will be evaluated using the degraded resistance.

For the case of methods based on allowable displacements, representative values of the maximum ground velocities are needed.

11.6.2.- SAFETY FACTORS

For all cases of pseudostatic analysis of inertial balance; i) with seismic actions and ii) postseismic, the minimum safety factor to failure will have to be greater than or equal to 1.2.

11.7.- METHODS OF ANALYSIS COUPLING STRESS-STRAIN WITH ACCELEROGRAMS

These methods can be used in the foundation analysis, retaining walls and sloped grounds as long as they adequately incorporate the non-linear soil behavior and, in the foundations and walls case, the interaction between them and the ground.

For the analysis, at least four (4) representative accelerograms of the expected seismic actions at the site shall be used. Such accelerograms can be registered or simulated by means of well known procedures. The average elastic spectrum of the selected accelerograms will have to conservatively approach the design spectrum given in Article 7.2 for the value R=1.

The dynamic response for design purposes shall be obtained from the analysis of the responses obtained for all cases with the selected accelerograms.

CHAPTER 12

EXISTING BUILDINGS

12.1.- SCOPE

In this Chapter the lineaments for the evaluation, upgrading, rehabilitation, reinforcement or reparation of existing buildings are established. They should be applied in conjunction with the rest of the prescriptions of this Code, except for the modifications indicated hereon.

12.2.- APPLICATION RANGE

The prescriptions of this Chapter are considered to be of enforced application in situations that may affect the seismic resistance behavior of existing buildings such as, although not restricted to:

a) Buildings whose structural system does not fulfil the requirements of Table 6.2 or that do not have the required strength or stiffness
b) Buildings that are damaged due to the occurrence of a quake.
c) Changes of usage, or enlargement of a building.
d) Substantial modifications of the resisting structure, total or partial elimination of diaphragms, suppression or addition of partition walls, or other situations where the expected response of the building to intense quakes is modified.
e) Evident deterioration in the resisting structure globally and/or lack of maintenance.
f) Non compliance with the prescriptions established in Chapter 11.
g) Buildings that have exceeded or that are close to reaching the limit of its service life.

12.3.- SEISMIC RESISTANCE CLASSIFICATION

12.3.1.- DESIGN LEVEL AND RESPONSE FACTOR, R

A design level (DL) will be assigned to each existing building based on the compliance of the code requirements of seismic resistance incidence in the components of the seismic resistant system, established in the actual COVENIN Codes. Its analysis will be made with the corresponding R value. When the minimum requirements of seismic resistance incidence, included in the present Codes and listed in Table 12.1 are not met, the design level shall be assigned as DLO. If a structure classifies as DLO the value R = 1.0 will be used.

TABLE 12.1
CODE REQUIREMENTS OF SEISMIC RESISTANCE INCIDENCE

	REINFORCED CONCRETE STRUCTURES
a)	The development lengths for anchorage and splice of the corrugated reinforcing bars. For smooth bars, the COVENIN Code 1753 required lengths will be doubled.
b)	The minimum and maximum amounts of reinforcement, for shear, bending and Compression with bending.
c)	The minimum diameters and maximum separations of the transverse reinforcement to resist shear and to confine compressed bars.
d)	The limitations of the tension and cold bending testing for bars or reinforcement nets, established in the COVENIN Codes.
	STEEL STRUCTURES
a)	The maximum width/thickness relationships of the wings of the profiles.
b)	The maximum relationships of slenderness in compressed members.
c)	The thickness limitations and length of welding, and bolts and rivets spacing.
d)	The stiffeners of the columns in the connections to bending.
e)	The lateral stiffening of beams and trusses.

12.3.2.- TYPE OF STRUCTURE

The building in consideration shall be assigned one of the Structural Types provided in Section 6.3.1 of this Code. In the case of frame type structures, with lack of beams in a main direction, Type IV will be adopted unless the partial collaboration of the slab can be incorporated, in which case Type I can be taken but with Design Level DL1 or DL0. In the case of structures whose seismic resistant system is formed by slabs and walls in only one direction, for its classification the same considerations will be applied than that of the direction lacking walls.

12.4.- SEISMIC ACTION FOR DESIGN AND/OR REVISION

In general, the value α φ Ao established in this Code shall be taken as horizontal acceleration coefficient. This value may be reduced if site studies are made; in no case the final values shall be less than 0.8 α φ Ao.

12.5.- EVALUATION

In the evaluation of an existing building the data corresponding to its project and construction should be used, as well as the resistance of the materials, the quality of the execution, the state of its maintenance and the analysis of its expected performance, taking into account the present uncertainties. The evaluation of the materials will have to be assumed by a Laboratory or specialized Institute.

The results of the evaluation of the building should justify the decision making in the following aspects:

a) The seismic resistant classification in the terms established in Article 12.3.

b) The selection of the base level and foundation type.

c) The structural model in accordance with the lineaments established in Chapter 8 of this Code.

d) The selection of the method of analysis among those established in Chapter 9.

e) The eventual unfavorable incidence that the location of the existing partitions may have in the response. Particular attention shall be paid to the following aspects:

   i) Discontinuity of the partitions density between the different levels, specially those that lead to irregularities of Types a1) ; a2) ; and a9), described in Section 6.5.2.

   ii) Concentration of partitions in a given zone of the plan that may generate torsional effects.

12.6.- ANALYSIS AND VERIFICATION

The analysis and verification of the building, as well as the upgrading project, will be done following the procedures established in the first 11 Chapters of this Code, with the modifications established in the Sections preceeding this Article.

In case that reinforcement structures with different ductility than the existing structure are added special care shall be taken not to exceed its displacements capacity, considering the compatibility of deformations and the P-Δ effects.

CHAPTER 13

SEISMIC INSTRUMENTATION

13.1.- GENERAL

Buildings that are constructed nation wide according to the present Code, located in zones 5, 6 and 7, will be instrumented according to the following lineaments:

13.1.1.- Every building with more than six levels in height or a constructed area of at least 6000 m^2, has to be instrumented with two accelerographs.

13.1.2.- Every building with more than 10 levels in height regardless of the constructed area, has to be instrumented with three accelerographs.

13.1.3.- At least one instrument should be placed on free field, in the different geotechnical profiles existent in the area, when dealing with:

a.- Developments with more than 10 buildings of 6 or less levels.

b.- Developments with more than 40 one-family homes.

c.- Buildings of special importance.

13.2.- TYPE OF INSTRUMENT

FUNVISIS is the governmental entity that runs the National Accelerographs' Network and thus is in charge of approving the types of instruments that are placed in the buildings that require them according to the requirements of the present Chapter. In the seismic instrumentation of buildings, digital accelerographs should be used to have a record of strong motions. FUNVISIS will be co-owner of the information generated regardless of who is the owner of the instrument.

13.3.- LOCATION OF THE INSTRUMENTS

Whenever two instruments are required, these will be located at the base and at the top of the building. In the case that three instruments are required, the third one will be located at mid height. This accelerographs will have to be interconnected among them to warrantee simultaneous records.

The instruments should be placed away from areas of circulation yet at easy access for its maintenance. In the building project, required spaces for the location of the devices will have to be thought of in advance.

13.4.- COSTS

The costs of aquisition, installation, area of installation and maintenance, will be paid by the owner of the building. The maintenance should be done with the frequency required by the instrument suplier; nontheless, it is recommended a maintenance frequency that in the average is of no less than six months.