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NEW DESIGN CONSIDERATIONS FOR SEISMIC ISOLATED BUILDINGS IN COLOMBIA

Carlos M. Piscal Arévalo

**DIRECTOR
Francisco López Almansa**

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**Universitat Politècnica de Catalunya
Escola Tècnica Superior d'Enginyers de Camins, Canals i Ports de Barcelona
Departament d'Enginyeria Civil i Ambiental**

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Carlos Mario Piscal Arévalo

To God and every bless that he gives me, especially my daughters Isabella and Julieta, and my wife Paola.

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Resumen

La mayoría de la población colombiana se encuentra ubicada en zonas de amenaza sísmica alta e intermedia (NSR-10, 2010) por lo cual es de suma importancia la protección de las edificaciones, con una especial atención en las que albergan un considerable número de personas y aquellas que son consideradas como indispensables para la atención de la comunidad después de un evento sísmico. Dentro de las más recientes y exitosas medidas de protección de estructuras a nivel mundial, se encuentra el denominado *aislamiento de base*; a pesar de sus notorias ventajas, en Latinoamérica y más específicamente en Colombia, su uso es aún limitado. Lo anterior puede deberse, entre otras, a las siguientes razones: escasa divulgación en la zona de esta técnica y de las ventajas de su uso, falta del conocimiento necesario (por parte de los profesionales de la construcción) para llevar a cabo los procesos requeridos de diseño y construcción, y a la ausencia de una normativa nacional específica; esta última circunstancia es relevante, dado que la normativa internacional que se aplica en su lugar (ASCE 7 en el caso colombiano) no recoge las particulares locales y puede generar sobrecostos importantes.

El objetivo final de esta investigación es consolidar el aislamiento sísmico en Colombia y en otros países próximos; para alcanzar este propósito, en esta investigación se llevan a cabo las siguientes actividades: (i) se estudian y comparan los requisitos estipulados en diferentes normativas de aislamiento sísmico desarrolladas a nivel mundial (Japón, China, Rusia, Italia, USA, Chile, México), (ii) en cada uno de estos códigos, se contrastan los resultados de fuerzas y desplazamientos obtenidos mediante métodos aproximados de análisis (Fuerza horizontal equivalente) y los resultados obtenidos mediante métodos más sofisticados (análisis dinámico cronológico), (iii) se proponen, para Colombia, factores modificadores del espectro de diseño en función del amortiguamiento, y (iv) se formulan nuevas consideraciones para el diseño de estructuras aisladas en Colombia (embrión de una normativa propia).

Summary

An important percentage of Colombian population is located in medium-to-high seismicity zones (NSR-10, 2010). Therefore, it is important to protect the buildings, principally the highly crowded ones, and those that are considered as indispensables for community attention after an earthquake event. One most successful techniques for structural protection is *base (seismic) isolation*; despite their obvious advantages, in Latin America (and, more specifically, in Colombia), its use is still only limited, whit a low number of isolated buildings. This situation can be due to several reasons: insufficient awareness of this technology, poor knowledge of the involved professionals (i.e. engineers) in design and construction of isolated structures, and lack of local design codes; this last circumstance, forces employing foreign regulations (ASCE 7 in the Colombian case) that do not account for the local characteristics and frequently lead to relevant cost increases. The objective of this Thesis is to promote base isolation in Colombia and other close countries; with this aim, the following tasks are performed: (i) to study and compare the requirements of different major seismic isolation regulations (Japan, China, Russia, Italy, USA, Chile, México), (ii) inside each code, to contrast the results (in terms of forces and displacements) obtained from approximated analysis methods (lateral equivalent method) and more sophisticated procedures (time-history analysis), (iii) to propose (for Colombia) damping modification factors for design spectra, and (iv) to formulate new considerations for design of isolated structures in Colombia (to be converted into a national design code).

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List of symbols

α : Coefficient in the Chilean regulations
 a_g : Acceleration at the bedrock in the Italian (European) regulations
 A : Maximum ground acceleration in the Chilean regulations
 A_a : Horizontal effective peak acceleration in the Colombian regulations
 A_v : Horizontal effective peak velocity in the Colombian regulations
 A_p : Lead core area
 A_r : Rubber area
 b, d : Shortest and longest plan dimensions of the building
 B : Damping modification factor in the American regulations
 B_a : Damping modification factor used for S_a spectrum
 B_d : Damping modification factor used for S_d spectrum
 B_D : Damping modification factor in the Chilean regulations
 B_0 : Coefficient in the Chilean regulations
 c : Damping coefficient
 C_c : Coefficient depending on the soil type in the Italian (European) regulations
 C_D : Coefficient in the Chilean regulations depending on soil and seismic zone
 C_U : Coefficient of use (Italian code)
 D : Design displacement of the isolators
 D_y : Yield displacement
 D, L : Dead and live loads
 D_T : Total (including torsion effects) design displacement of the isolators
 D_{TM} : Total (including torsion effects) maximum displacement of the isolators
 E_D : Energy dissipated
 e : Eccentricity (between the center of mass of the superstructure and the center of rigidity of the isolation system, plus the accidental eccentricity)
 f_c' : Characteristic value of the concrete compressive strength
 f_y : Reinforcement steel yield point
 f_{yp} : Shear stress at yield of lead
 F : Force
 F_a, F_v : Site coefficients
 F_y : Yield force
 F_{sub} : Design force for the superstructure
 F_{sup} : Design force for the substructure
 F_{Δ} : Force for obtaining the drift limit (Δ_{lim})
 F_0 : Spectral amplification factor in the Italian (European) regulations
 g : Gravity acceleration
 G_r : Shear modulus of rubber
 G, Q, E : Permanent (dead), variable (live) and seismic forces according to the European regulations
 $G_s(T)$: Soil amplification factor in the Japanese regulations
 h : Height above the isolation interface
 h_v, h_d : Viscous and hysteretic damping factors (Japanese code)
 H : Building height
 I : Importance factor
 k : Stiffness. Exponent in the vertical distribution of lateral forces (USA regulations)
 k_p : Plastic Stiffness
 $K_{e\ max}$: Maximum equivalent (secant) stiffness of the isolation layer with respect to the variability of its mechanic parameters (US)

$K_{esi,min}$: Minimum equivalent (secant) stiffness of the isolation layer with respect to the variability of its mechanic parameters (Italy)
 K_e : Effective stiffness of the isolation layer
 K_M : Equivalent stiffness of the isolation layer corresponding to the maximum displacement
 K_{oil} : Stiffness of the viscous damper representing the oil compressibility
 M_L : Local magnitude
 M_w : moment magnitude
 M, m : Mass of the superstructure
 P_r : Ratio between the effective translational and torsional periods
 q : Behavior factor in the European regulations; is equivalent to the response modification factor in the American codes
 Q : Characteristic force
 Q_E : Maximum considered earthquake effect (American regulations)
 r_x, r_y : Torsional radii in x and y directions (Table 13)
 R : Response modification factor in the American regulations
 S : Soil coefficient in the Italian (European) regulations
 S_a spectrum: Design spectrum pseudo accelerations
 S_d spectrum: Design spectrum pseudo displacements
 S_A : Pseudo-acceleration
 S_v : Pseudo-velocity
 S_s, S_1 : Design accelerations (MCE) for short periods and for 1 s, respectively (USA regulations)
 S_{DS}, S_{D1} : Design accelerations for short periods and for 1 s, respectively (USA regulations)
 S_{MS}, S_{M1} : Maximum accelerations for short periods and for 1 s, respectively (USA regulations)
 S_S : Stratigraphic amplification in the Italian (European) regulations
 S_T : Topographic amplification in the Italian (European) regulations
 S_0 : Spectral acceleration in bedrock in the Japanese regulations
 T : Period, fundamental period
 T_a, T_b, T_c and T_d : Corner periods in the Chilean regulations
 T_C, T_D : Corner periods of the design spectrum in the European regulations
 T_C, T_L : Corner periods of the design spectrum in the Colombian regulations
 T_C^* : period depending on the location in the Italian (European) regulations
 T_D : Soil period in the Chilean regulations
 T_D, T_M : Fundamental periods of the isolated building for the design and maximum displacements in the American regulations
 T_{fb} : Fundamental period of the building under fixed base conditions
 T_g : Soil characteristic period in the Chinese regulations (equation (8))
 T_r : Reference return period
 T_R : Return period
 T_0, T_S and T_L : Corner periods of the design spectrum in the USA regulations
 u : Relative displacement
 \dot{u} : Relative velocity
 \ddot{u} : Absolute acceleration
 \ddot{u}_g : ground acceleration
 v : Velocity
 V_N : Nominal structural life (Italian code)
 V_R : Reference period (Italian code)
 W, W_s : Seismic weights with and without the base level weight
 ω_i : Angular frequency
 $\bar{\omega}_i$: Damped angular frequency
 x : Displacement
 x, y : Horizontal coordinates or distances

Z : Zone factor in the Japanese regulations
 α : Exponent of the velocity in the viscous damper constitutive law
 α_p : Apparent shear modulus of lead
 $\alpha_A A, \alpha_V V, \alpha_D D$: Acceleration, velocity and displacement parameters in the Chilean regulations
 α_{max} : Factor related to the seismic intensity in the Chinese regulations
 β : Damping factor in the Chilean regulations
 β_{eff} : Effective damping
 β_i : Design spectrum in the Russian regulations
 β_M : Effective damping for the maximum earthquake
 Δ_{lim} : Drift limit
 γ, η_1, η_2 : Damping modification coefficients in the Chinese regulations
 η : Damping modification factor in the European regulations
 λ : Factor that modifies the major mechanical parameters of the rubber bearings
 ξ : Damping factor (damping ratio)
 ψ_E : Combination coefficient in the European regulations
 μ_s : friction coefficient

List of abbreviations

ASCE: American Society of Civil Engineers
BSL: Building Standar Law of Japan
DBE: Design Basis Earthquake
EERI: Earthquake Engineering Research Institute
EN: Euro-Norm
FEMA: Federal Emergency Management Agency
GB: National Standar of the People’s Republic of China
HDRB: High Damping Rubber Bearing
LDRB: Low Damping Rubber Bearing
LRB: Lead Rubber Bearing
MCE: Maximum Considered Earthquake
MPE: Maximum Probable Earthquake (Russian regulations)
MDOF: Multiple degree of freedom
MRF: Moment resisting frame
NRB: Natural Rubber Bearing
NSR: Reglamento Colombiano de Construcciones Sismo Resistentes
NCh: Norma chilena de aislamiento sismico
PGA: Peak ground acceleration
SLC: Collapse Prevention (“*Stato limite di prevenzione del collasso*”)
SDOF: Single degree of freedom
SLD: Damage (“*Stato limite di danno*”)
SLO: Operability (“*Stato limite di operatività*”)
SLV: Life Safety (“*Stato limite di salvaguardia della vita*”)
SMP: Maximum Possible Earthquake (“*Sismo Máximo Posible*”)
SRB: Sliding Rubber Bearing
RC: Reinforced Concrete
RK: Risk Categories
SDOF: Single Degree of Freedom

1. INTRODUCCION

1.1 Background and motivation

Colombia is a country with medium and high seismicity regions. Thus, along history, a number of strong earthquakes have shaken the country, with serious consequences in terms of fatalities and important destruction. Seismic isolation is a recently proposed technique for seismic protection of buildings and bridges. This technology has repeatedly proved worldwide its efficiency; therefore, might be used to reduce the seismic vulnerability in Colombia. However, in Latin America and more specifically in Colombia the use of this technique is still limited; more precisely, in Colombia there are approximately 30 isolated buildings (Mason, 2015), most of them being hospitals. Such scarcity can be due to several reasons: rather poor preparation of the involved professionals (mainly civil engineers and architects), insufficient promotion, some degree of distrust, and high design and construction costs. Regarding this last issue, can be partly attributed to the lack of a local design code, forcing to use foreign regulations (basically the USA ones), which might be over-conservative and do not contemplate the local particularities of Colombia (mainly, in terms of seismicity). Being aware of this situation, the Colombian Society of Seismic Engineering (AIS) is promoting a new design code for base isolated buildings in Colombia.

Given the situation described in the previous paragraph, the author of this document contacted in **September 2015** Prof. Francisco López Almansa, who accepted to supervise the research. Its final objective (far beyond the Doctoral Thesis of Mr. Piscal A.) is to promote the use of base isolation in Colombia, both for buildings and bridges; it affects all the potential causes of the current lack of development of this technology in Colombia. As discussed later, this Thesis focusses on design of base isolated buildings.

1.2 Objectives

1.2.1 Main objective

The main goal of this study is **TO PROPOSE NEW DESIGN CONSIDERATIONS FOR SEISMIC ISOLATED BUILDINGS IN COLOMBIA** to contribute to the future Colombian seismic isolation code and to promote the use of this technique in the country.

1.2.2 Specific objectives

- To select a number of major seismic isolation codes and to analyze and compare their main prescriptions.
- To select a prototype building representing a typical hospital facility.
- To develop numerical models of the prototype building equipped with seismic isolation systems.
- To generate artificial seismic accelerograms (in order to carry out nonlinear dynamic analyses) that are fitted to different design codes spectra.
- To design the isolation system of the prototype building according to the selected isolation codes and to compare the obtained results.
- To identify relevant aspects of seismic isolation regulations applicable to the future Colombian code.
- To obtain new damping modification factors for Colombia.
- To propose new design consideration for seismic isolated buildings in Colombia (i.e. the draft of the new Colombian regulation).
- To issue overall conclusions.
- To identify further research needs.

1.3 Methodology

This section describes in more detail the investigation carried out to achieve each of the above specific objectives.

Analysis of relevant seismic isolation codes. The most relevant regulations for base isolation are selected. This selection is based on the implementation of this technique in the corresponding country; moreover, the USA regulations are also considered, given their strong

influence in Latin America. As well, the Chilean and Mexican codes are included, as being the only countries in Latin America with specific regulations. The following regulations are analyzed: USA, Japan, China, Italy, Chile, Mexico and Russia. These regulations are thoroughly analyzed and compared; this includes, the analysis and design methodologies, the design spectra, the damping modification factor, the return periods for designing the superstructure and the isolation layer (seismic hazard levels), the importance factor, the variation of parameters of isolator units, among others.

Selection of a prototype building. A hospital building is chosen because its relevance in case of severe earthquakes and because most of the isolated buildings in Latin America have such use. Accordingly, the selected prototype building has the following characteristics (typical of hospital buildings): moderate height, horizontal architecture model (aiming to facilitate access and circulation), large span-length (for better use flexibility), redundant and spacious vertical connections (stairs, elevators, ramps), and wide horizontal connections (e.g. corridors) inside each story. Given that the Italian regulations allow considering different importance factors, housing use is also contemplated.

Developing numerical models of the prototype building with seismic isolation systems. Numerical models of the superstructure, the substructure and the isolation system are developed in Etabs. Given that no or little damage is expected in the superstructure and the substructure, linear behavior is assumed there; conversely, the nonlinear behavior of the isolation layer is incorporated into the model. Apart from that, a specific software for designing the rubber bearings (elastomeric isolators) is developed; this software is programmed in Java.

Generation of artificial seismic accelerograms fitted to different code spectra (to perform nonlinear time-history analyses). A suite of artificial accelerograms matching the corresponding design spectra are generated. As required by many design codes, seven accelerograms are generated for each spectrum; thus, 896 accelerograms are created.

To make nonlinear dynamic analysis and evaluate important results of the isolation systems principally, artificial seismic inputs fitted to each one of the selected codes spectra are created, the characteristics to generate these inputs are carefully defined. To try to avoid bias in the results, seven artificial accelerograms are obtained for each spectrum, for a total of 896

accelerograms, it considers two acceleration magnitude, two directions of application and four design parameters with different spectral characteristics, are necessary for the study.

Comparison among the results of designing the prototype building with the analyzed seismic isolation codes. The isolation layer of the prototype building is analyzed and designed with each of the considered codes; such operations are performed by using linear static and nonlinear dynamic analyses. In the dynamic analyses, the aforementioned artificial seismic accelerograms are employed. The comparison is established in terms of a number of design quantities: superstructure and substructure forces and isolation system displacements.

Identification of the relevant aspects of the analyzed seismic isolation codes that can be applicable to the future Colombian code. The prescriptions of the examined seismic isolation codes that are applicable to Colombia (keeping in mind the Colombian code (NSR-10)) are identified and discussed.

Proposal of a new damping modification to be used in Colombia. One of the most important parameters to estimate forces and displacements in the static linear method is the damping modification factor. Such factor depends on the characteristics of the local earthquakes, the soil type, the structural period, etc. In Colombia, the damping factor contained in the former USA code (ASCE 7-10, 2010) is currently used; however, such factor is fitted to the US characteristics, being clearly different than those in Colombia (and other Latin American countries). Therefore, the need of developing a specific damping modification factor for Colombia is obvious. Given the scarcity of strong Colombian (historical) records, the proposed factor is developed after a suit of artificial accelerograms that have been generated to match the 5% design spectra for each of the seismic zones the country is divided in. The employed methodology is based on performing linear dynamic analyses on under and overdamped SDOF systems by using the aforementioned artificial accelerograms. Although previous studies have highlighted the differences among factors generated after natural and artificial inputs, it has been observed that such discrepancies are mainly due to the longest significant (Trifunac) duration of the artificial accelerograms; therefore, the artificial inputs are generated as their duration fits those of the available local strong motion records. The obtained damping modification factor is compared with the results for some available Colombian records. The sensitivity of the calculated factors to the soil type, period and seismic zone is investigated, and matching expressions are provided. Such expressions are compared with the prescriptions of

the major worldwide design codes and with other studies. The suitability of the proposed formulation is further verified with nonlinear time history analysis of an example on an isolated hospital building; the results are satisfactory.

Proposal of new design consideration for seismic isolated buildings in Colombia. New design considerations and criteria are proposed; it includes expressions of the relevant design quantities, the proposed damping modification factor, importance factors, drift limits, etc. Noticeably, the requirements for essential and normal importance buildings are different.

Conclusions. Both overall and particular conclusions are issued.

Further research. Taking profit of the results of this study, new research needs are identified and discussed.

1.4 Organization of this document

This document is organized in six chapters and an appendix, where the first chapter is this introduction. The second chapter is the state of the art, which contains a review of the seismicity of Colombia and the microzonation of Cali and Bogota; this chapter also contains a review of seismic isolation concepts and its application both in Colombia and worldwide. The third chapter describes the survey on the aforementioned seismic isolation codes. The fourth chapter presents the study on the proposed damping modification factor. Chapter 5 describes new considerations for the future design code of seismic isolated buildings in Colombia. Finally, Chapter 6 presents the overall conclusions of the research, and the future investigations. A list of the consulted bibliography is included after chapter 6. Appendix A lists the publications generated during this research.

2. STATE OF THE ART

2.1 Seismic information of Colombia

2.1.1 Seismicity of the country

Colombia is located in the denominated Circum-Pacific ring, a zone with high seismic activity. In the country three lithospheric plates, Nazca, Caribbean, and South American, converge (Figure 2-1), and their movements produce different types of geologic faults (Hudson, 2010).

The predominant faults in Colombia have north-south direction. There are a considerable number of these faults seismically activate, like show Figure 2-2.

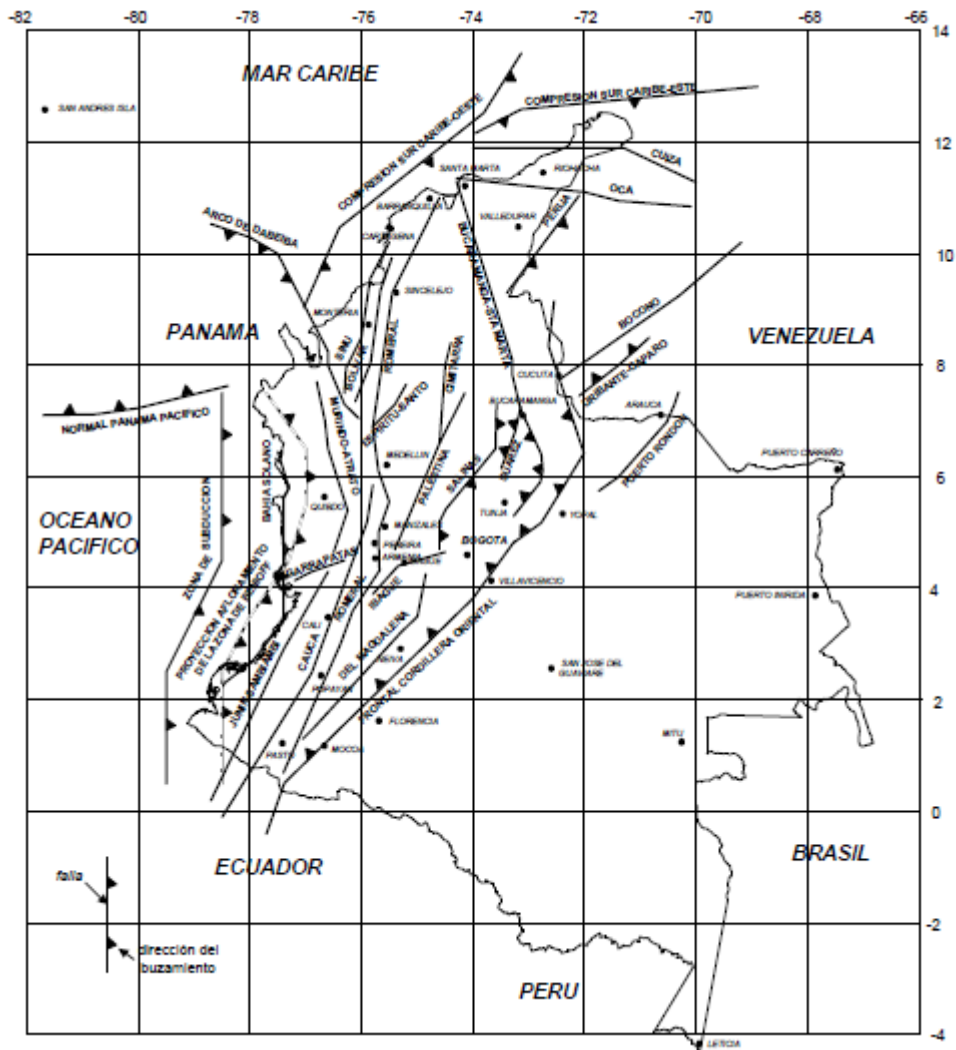


Figure 2-2. Colombian seismic faults (NSR-98, 1998)

Colombia has been exposed to a considerable number of earthquakes coming of the different faults. The Figure 2-3, shows epicentral location of these with magnitude $M_s \geq 3$ recorded between 1541 and 2009.

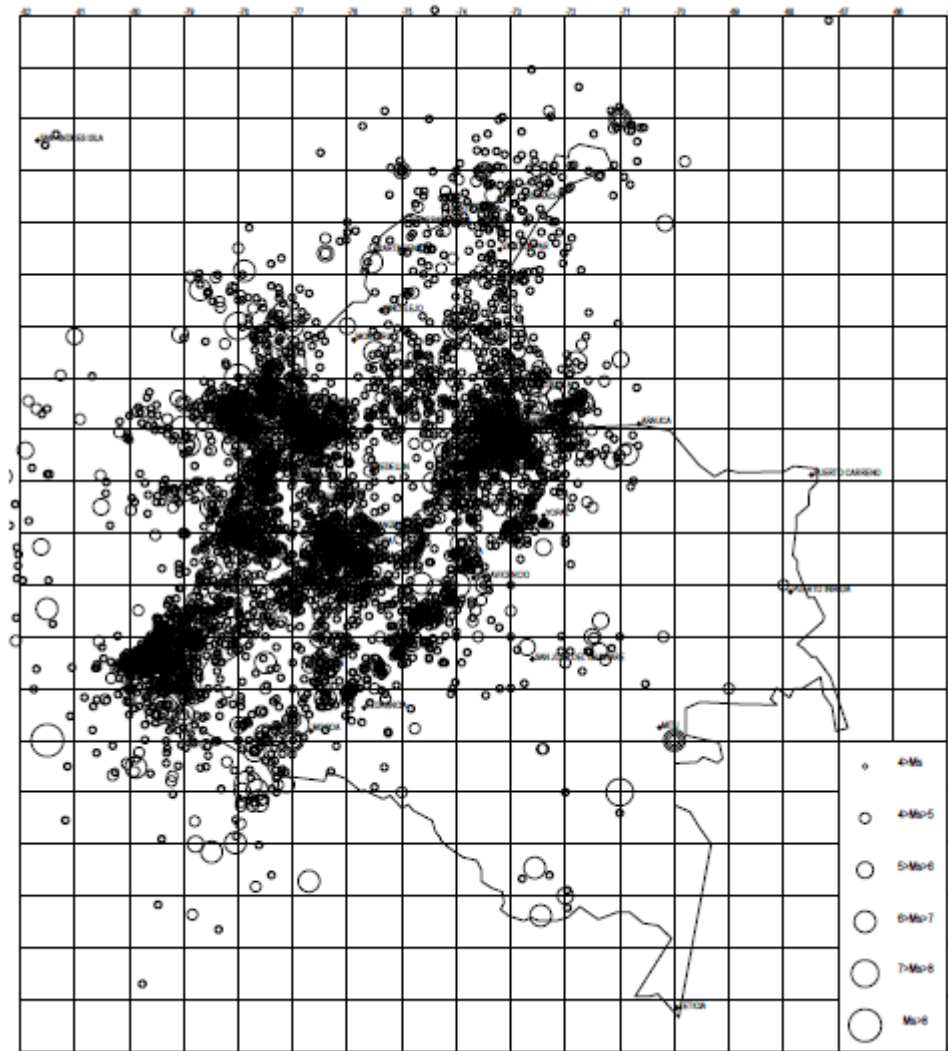
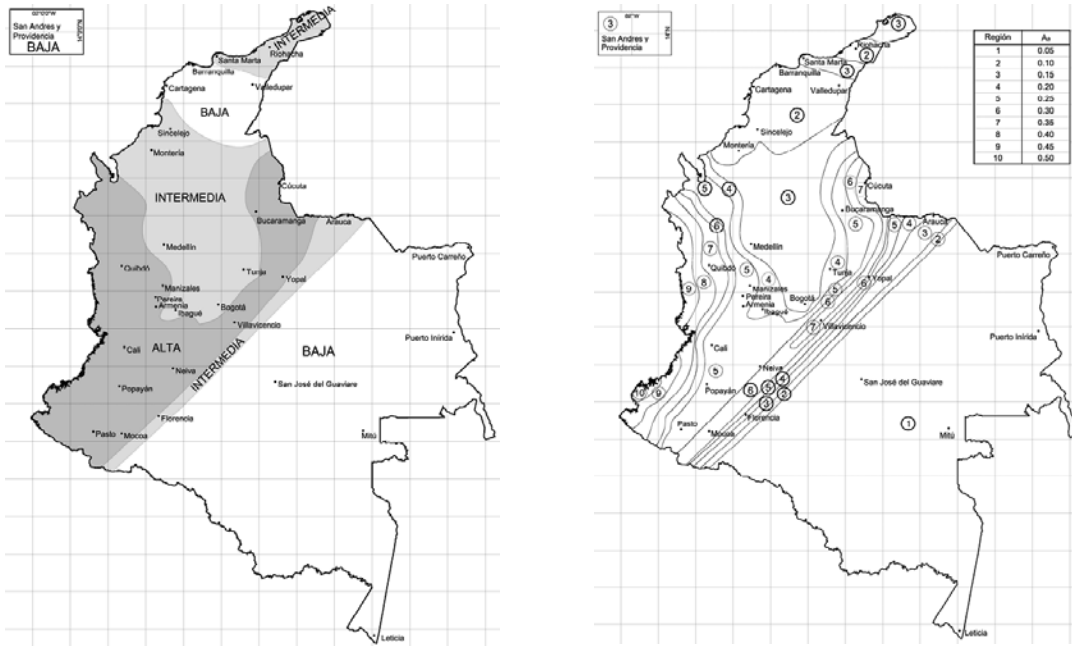


Figure 2-3. Earthquakes with magnitude $M_s \geq 3$ (NSR-98, 1998)

Former figure is coherent with the definition of seismic hazard showed in the Figure 2-4 and stipulated in Colombian Regulations of Earthquake Resistant Building NSR-10. In this figure the country is divided in three hazard levels: low, intermediate and high. 39.7% of Colombian population live in high seismicity zone, 47.3% live in intermediate and 13% live in low (NSR-10, 2010). Each seismic zone is subdivided in several sub-regions in terms of the horizontal pseudo acceleration coefficient both for low (A_a) and intermediate (A_v) periods whose values range between 0.05 to 0.50 (g).



(a) Seismic zones

(b) A_a factor

Figure 2-4. Colombian Seismic hazard (NSR-10, 2010)

It is important to note that cities such Bogota and Cali currently has seismic microzonation studies that govern its definition of seismic hazard.

2.1.2 Seismic microzonation of Bogotá

Adopted through order 523, 2010 (Decreto 523, 2010). Divide the city in 16 seismic zones, like shows Figure 2-5. Parameters required to build design spectra for each zone are defined in Table 2-1. A_a and A_v are 0.15 and 0.20 respectively for all zones.

Table 2-1. Parameters for design spectra, Bogotá microzonation

Zone	F_a	F_v	T_C (s)	T_L (s)
Cerros	1.35	1.30	0.62	3.0
Piedemonte A	1.65	2.00	0.78	3.0
Piedemonte B	1.95	1.70	0.56	3.0
Piedemonte C	1.80	1.70	0.60	3.0
Lacustre-50	1.40	2.90	1.33	4.0
Lacustre-100	1.30	3.20	1.58	4.0
Lacustre-200	1.20	3.50	1.87	4.0
Lacustre-300	1.05	2.90	1.77	5.0
Lacustre-500	0.95	2.70	1.82	5.0
Lacustre aluvial 200	1.10	2.80	1.63	4.00
Lacustre aluvial 300	1.00	2.50	1.60	5.00
Aluvial 50	1.35	1.80	0.85	3.50
Aluvial 100	1.20	2.10	1.12	3.50
Aluvial 200	1.05	2.10	1.28	3.50
Aluvial 300	0.95	2.10	1.41	3.50
Deposito ladera	1.65	1.70	0.66	3.00

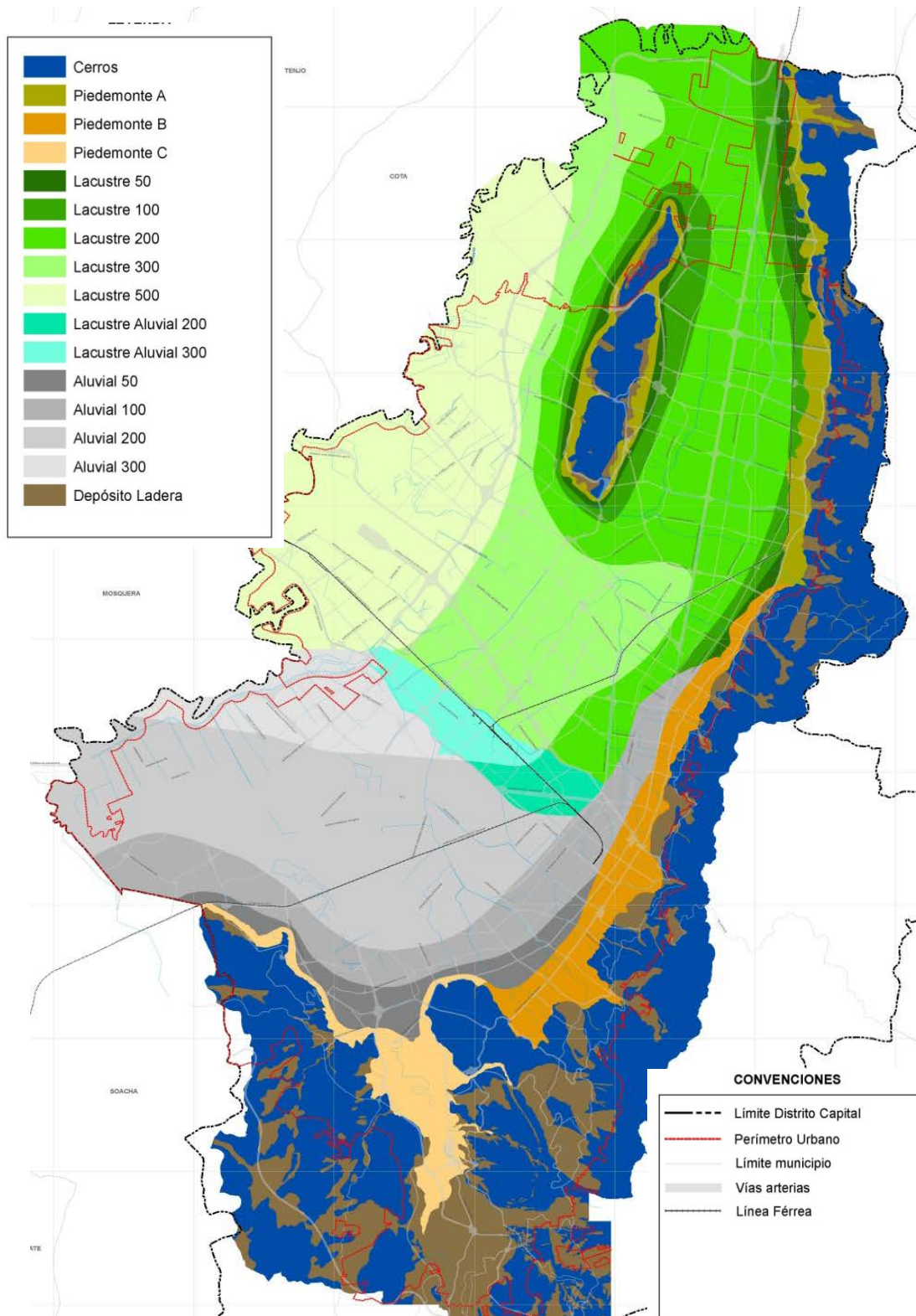


Figure 2-5. Seismic microzonation of Bogotá (Decreto 523, 2010)

2.1.3 Seismic microzonation of Cali

Adopted through order 411, 2014 (Decreto 411, 2014). Divide the city in 10 seismic zones, like shows Figure 2-6 . Parameters required to build design spectra for each zone are defined in Table 2-2. A_a and A_v are 0.25 and 0.25 respectively for all zones.

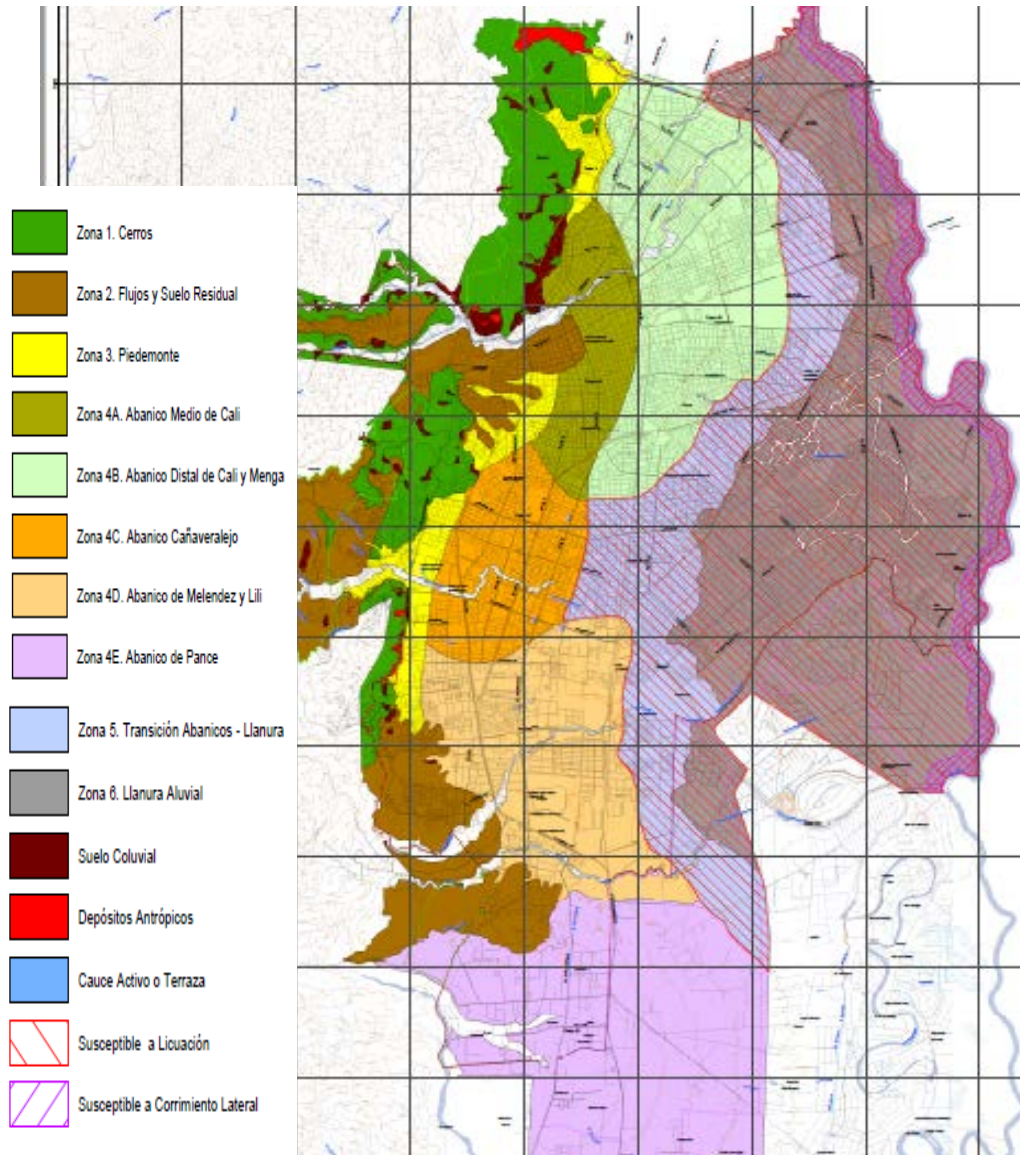


Figure 2-6. Seismic microzonation of Cali (Decreto 411, 2014)

Table 2-2. Parameters for design spectra, Cali microzonation

Zone	T_C (s)	F_a	T_L (s)	F_v	
1.Cerros	0.55	0.86	3.00	0.99	
2.Flujos y suelo residual	0.45	1.20	3.00	1.13	
3.Piedemonte	1.05	1.36	2.00	2.98	
4a. Abanico medio de Cali	0.75	1.20	2.00	1.88	
4b. Abanico Distal de Cali y Menga	T_c	0.70	1.04	2.50	1.52
	T_L	1.60	0.80	2.50	2.67
4c. Abanico de Cañaveralejo	T_c	0.45	1.60	2.00	1.50
	T_L	1.50	1.04	2.10	3.25
4d. Abanico de Melendez y Lili	1.20	0.99	2.00	2.48	
4e. Abanico de pance	0.95	0.91	3.00	1.61	
5. Transición abanicos llanura	T_c	0.60	1.12	2.50	1.40
	T_L	1.35	0.83	2.50	2.34
6. Llanura Aluvial	1.15	1.09	2.50	2.61	

2.2 Seismic isolation

2.2.1 Introduction

It is important remember that the damage produced by earthquakes in conventional structures (without isolation) is due to the direct connection between these and the soil, because the soil transport all the energy of the earthquake and transmitted it directly to the structures, where it is manifested fundamentally by means of movement, acceleration and deformation of both the structural and nonstructural elements; the dissipation of the energy is carry out by means of damage.

Since many years ago, different researchers have tried to find multiple ways or mechanisms to decouple the direct connection between the soil and the structure to avoid the total transmission of the energy between them, obviously total release of this joint is not real, therefore different

options have been considered, for instance rollers, balls, cables, rocking columns, sand, etc (Naeim and Kelly, 1999).

Nowadays, devices with high horizontal flexibility and high vertical rigidity (commonly termed as isolators) have been developed to such purpose, these devices are incorporated between the soil and the structure generating the technique called “*seismic base isolation*”. Because the seismic base isolation system has devices highly flexible in horizontal direction, the energy is concentrated on them through deformation and only a little part of the total energy is transmitted to the structure, in other words, during seismic shaking, the main body of the building remains basically motionless, while the isolators are significantly strained. Of this way, seismic isolation controls the damage and protect structures.

Figure 2-7, illustrates some constructive differences between a structure with and without seismic isolation. The Figure 2-7 a) shows a conventional building with five stories and a basement, the foundation is composed by individual footings, the structure is directly joint to the soil. The Figure 2-7 b) shows the same building but with base isolators to uncouple the principal structure of the soil, and the seismic gap required to permit the free displacement of the isolators. The group of isolators constitute the isolation interface, the parts of the building above and below the isolation interface are termed as superstructure and substructure, respectively. Additional dampers can be included too in the system. It is important to note that a basement story or crawl space are necessary for maintenance operations.

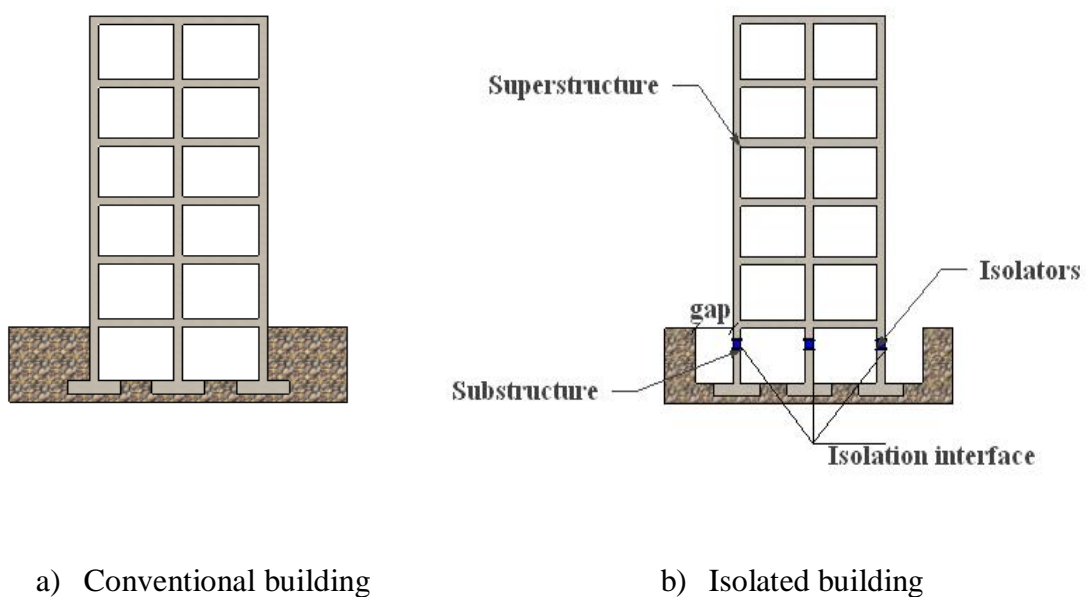


Figure 2-7. Differences between a structure both with and without seismic isolation

To support wind loads a sacrificial wind restraint system might be provided; obviously, it is required that the wind forces are lower than the seismic ones. All the connections (gas, water, electricity, internet, etc.) need to be flexible to accommodate the seismic displacement, with equipment like elevators, particular constructive considerations must be have into account.

The advantages of seismic isolation come of two principal variables:

Flexibility

The flexibility of isolation system makes that fundamental period of the structure in both directions increases notably (2 -3 s) and its value moves away from the predominant low period (< 1 s) of the seismic excitations, in this way there is a considerable reduction in the design spectral acceleration, but with an increase of displacement. The Figure 2-8 shows this concept graphically, for typical periods of isolated structures, the accelerations are small and displacements are high, for typical periods of conventional structures (low/mid height) the behavior is opposite to the former case.

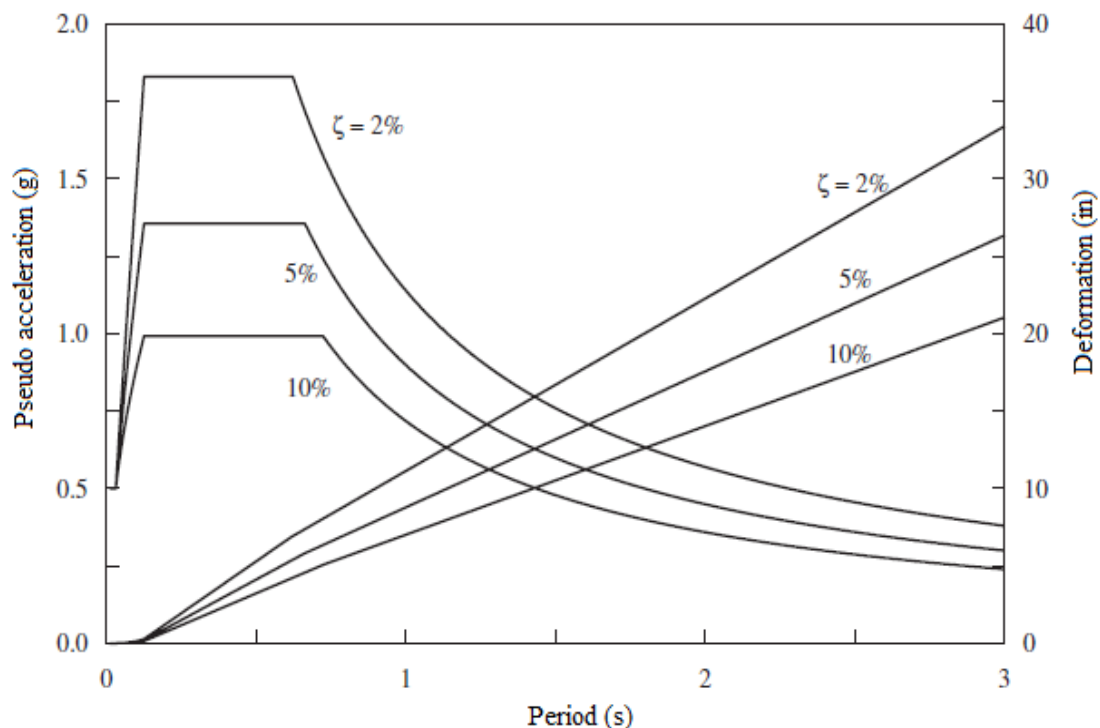


Figure 2-8. Influence of flexibility and damping in pseudo acceleration and deformation (Chopra, 2001)

The ideal behavior of a structure during earthquake excitations would be with low accelerations to avoid nonstructural damage, and low displacements to avoid structural damage, this behavior is impossible to achieve with conventional structures, because when spectral accelerations are low spectral displacements are high, and vice versa. In isolated structures while spectral accelerations are low, spectral displacement are high, but these displacements correspond to isolation system and not to the structure which has low displacements.

Damping

The devices used in seismic isolation (isolators and dampers), generally provide higher additional damping than in conventional structures, where traditionally 5% damping is considered. The effect of additional damping is a reduction in spectral ordinates regard normal spectra for 5% of damping. This effect can be observed in Figure 2-8, generally in isolated structures, damping values are between 20 and 30 %, it is depending on the isolator type. In conventional structures achieve higher values of damping is very difficult, because the damping in these structures is associated among others to inelastic deformations and therefore damage.

Regard dynamic behavior of conventional vs isolated buildings, the high flexibility and high damping of the last ones, introduce in the total system three new orthogonal local modes (two translational and one torsional) whose deformation concentrates mainly in the isolators, and have a high modal participation. The high modes correspond to the superstructure, which keeps mainly unstrained (i.e. rigid-body motion) and its modal participation is very low, therefore both the displacements and the forces in the superstructure are low, which correspond to the philosophy of base isolation technique, it means reduce demand and does not increase resistance in the structure.

This technic has a minor advantage regard its use in the three principal cases described follow:

Building height. In high-rise buildings, the weight is high and the fundamental period is long; also, the wind forces can be higher than seismic ones.

Soil stiffness. Base isolation is not so adequate for soft soil, since it filters out the short period waves while amplifies the long period components; therefore, given the similarity between long periods and those of the isolation system, in the superstructure the response could prove enlarged, instead of reduced

Input pulses. Velocity pulses (due to near-fault effects) can generate large permanent displacements (fling), thus requiring big seismic gap.

2.2.2 Worldwide application of seismic isolation

Apart from similar techniques of ancient cultures, seismic isolation of buildings started been used in 1960. The first use of a rubber isolator was in the Pestazzoli School 1969 in Skopje, Yugoslavia, where rubber blocks without any kind of reinforcement (steel plates) were used, for this reason the weight of the building caused them to bulge sideways. The first seismic isolated building in the world to incorporate lead rubber bearings (LRBs) was the William Clayton building in Wellington, New Zealand (1978), this type of isolator is widely used in this country, because this is the place where this device was invented. The first building in the world with high damping rubber bearings (HRBs) was the Foothill Community Law and Justice Center (FCLJC) in California (1985), USA. In the rehabilitation of buildings subject, in 1986 the City and Country Building was the first existing structure retrofit with this technology, and the United States Court of Appeals Building in California, was the first building isolated retrofit with friction pendulum bearing. Finally, in California, 1991, was constructed the USC University hospital, the first seismic isolated Hospital in the world (Morgan and Mahin, 2011; Naeim and Kelly, 1999; Taylor and Igusa, 2004).

From then on, seismic isolation has been deeply investigated, and many applications have been reported. Table 2-3, displays the number of buildings with base isolation in the countries where this technology is most spread, these figures are only approximated and were reported between 2013 and 2016 (Martelli et al., 2014; Mason, 2015)

Table 2-3. Number of buildings with base isolation

Type of building	Country						
	Japan	China	Russia	Italy	New Zealand	USA	Chile
Essential facilities	660	330				75	16
Other uses	2340	1170	600	400	100	163	35
Houses	5000	3500	-			12	28
Total	8000	4970	600	400	100	250	79

Other countries have less buildings with seismic isolation: Thailand 50, Canada 50, Armenia 45, Turkey 40, Mexico 25, Colombia 20, Peru 10, Ecuador 7 (Martelli et al., 2014; Mason, 2015).

Specifically, in Colombia some of the buildings with seismic isolation are described below:

“Amiga de Comfandi” Hospital: It was the first building with seismic isolation in Colombia, it was finished in 2011 and it is located in Cali, a city with a high seismic hazard. It has an area of 70.000 m², the devices used were 154 LRBs with 750mm of diameter, 58 LRBs with 58mm of diameter and 220 Sliders.

“Imbanaco” Hospital: This hospital too placed in Cali, was finished in 2012. It has an area of 72.000 m², 7 stories and 3 basements (30 m deep). It was constructed with LRBs of 1155mm (17), 1054.1mm (13), 952.5 (67) mm of diameter and 23 Sliders.

“Fundadores” Building: This academic building, finished in 2014, pertains to Autonomía University of Manizales. It has 6 stories, and it was constructed with 29 isolators and 26 sliders.

“Rogelio Salmons” Cultural Center: Placed in Manizales, with an area of 7.000 m². The first part of the project was finished in the year 2016.

Other projects with seismic isolation are: Terrasole building and Los Rosales Hospital in Pereira.

2.2.3 Performance of existing isolated buildings under some recent earthquakes

A great number of isolated buildings have performed satisfactorily under strong earthquakes, for instance on February 27, 2010 in Chile, a magnitude Mw 8.8 earthquake occurred at the coast of Maule. This natural event caused approximately 800.000 victims (estimated injured, lost housing, died, and missing) 370.000 buildings damaged or destroyed, 1.714 educational buildings non- operative, 18 Hospitals uninhabitable and 31 with moderate damage. In the case of isolated buildings during the earthquake, all the isolators were activated, the behavior of both structural and nonstructural elements were excellent. Buildings such as Militar Hospital and San Carlos de Apodquino Hospital remained working normally. According to the government,

the cost of rebuilding public facilities could be of about 1200 million US\$ (Almazán, 2012; Revista BIT, 2010)

In 2011 an earthquake with a magnitude Mw 9.0 occurred in Tohoku, Japan. The combined impacts of the earthquake and tsunami caused at least 155.000 victims (estimated injured, lost housing, died, and missing) and 332.000 buildings damaged or destroyed. It is probably not an exaggeration to say that in this earthquake there were the major number of buildings with protective systems exposed to a ground shaking compared with all previous global earthquakes combined and the total buildings with control systems exposed to them. Moreover, the magnitude and duration of this earthquake (near to five minutes), allowed to observe the performance of full-scale structures subjected to long duration, low frequency and high amplitude movement. The Figure 2-9 shows response spectra recorded at station K-NET MYG013 in Sendai, it is evident the high ordinal spectra and the predominant period near to 0.8 seconds (EERI, 2012)

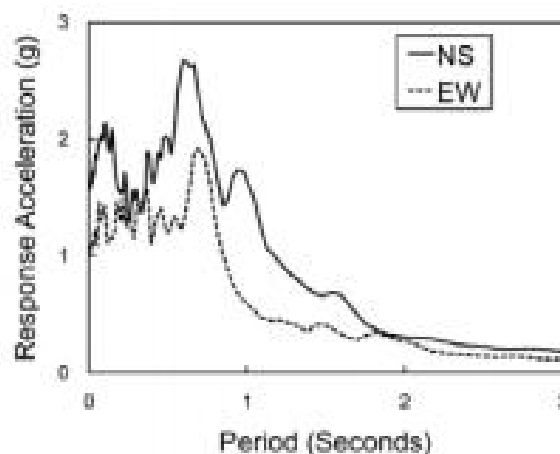


Figure 2-9. Response Spectra recorded at station K-NET MYG013 in Sendai. (EERI, 2012)

The results of some isolated instrumented buildings were reported. The Table 2-4 and the Table 2-5, show some important figures of isolated buildings exposed to Tohoku earthquake. All of them remained fully functional after the earthquake and minimized the dynamic amplification of ground motions over the height of the building. The minor nonstructural damage of Yozemi Tower refers to some damage in some joint covering the seismic gaps at the base of building without affect the functionally (EERI, 2012).

Table 2-4. Description of the buildings and isolation systems (EERI, 2012).

Location	Name	Stories	Isolation system
Senday	MT office	18	26 Rubber bearings + 10 sliders
Tokyo	Yozemi Tower	26	25 Rubber bearings + 24 sliders + 12 passive oil dampers and 12 semi active controlled dampers
Tokyo	J2	20	16 Rubber bearings + 14 yielding metal dampers+ 2 oil dampers + devices to limit uplift effect
Tokyo	Main building Shimizu corporation	6	6 Lead rubber bearings

Table 2-5. Description of the behavior of buildings after the earthquake (EERI, 2012).

Maximum displacement of isolation system (cm)	Peak ground acceleration (cm/s ²)	Peak floor acceleration (cm/s ²)	Structural damage	Nonstructural damage
23	311	207	None	Minor
10	----	----	None	None
12.5 (radial)	69 (NE-SW)	116.2 (NE-SW)	None	None
----	132 (Y)	64 (Y)	None	None

In April 20, 2013, an earthquake with magnitude of $M_w=6.6$ struck Lushan (China). There were more than 250.000 victims (died and injured). About 400.000 buildings were damaged or destroyed, included numerous essential and public buildings. The structures with isolation system, showed once again, an excellent behavior. Particularly two documented cases are very interesting. In the first one, two school located one next to the other. The school with conventional foundation had a PGA value of 0.2 g and it value was amplified, at the roof, to 0.72 g, while in the school with isolated structure the aforementioned PGA was reduced to 0.12 g at the roof. The other case corresponds to two blocks of Lushan's Hospital, the first block, with conventional foundation, suffered damage in partitions, roof and equipment contained, which made it unusable after the earthquake. Conversely, the seismically isolated block was the

only Lushan's Hospital that remained fully operational and without damage. The Figure 2-10 a), shows damage suffered by conventional Hospital, and the Figure 2-10 b) shows like the isolated Hospital remained operability after the earthquake (Martelli et al., 2014; Zhou et al., 2013).



Conventional block



Isolated block

Figure 2-10. Behavior of Lushan's Hospital during earthquake

2.2.4 Theoretical basis of seismic isolation

The follow dynamic equations are derived in function of the follow assumptions:

- Can be used the equivalent damping and equivalent stiffness. Therefore, the isolation system is represented with linear behavior
- The superstructure has a linear behavior

2.2.4.1 Single degree of freedom

A structural model of simple degree of freedom (SDOF) (Figure 2-11) can be used to represent the motion of a seismically isolated building, assuming that the movement of the structural system mainly occurs at the isolation interface, while the superstructure is extremely rigid (rigid body movement).

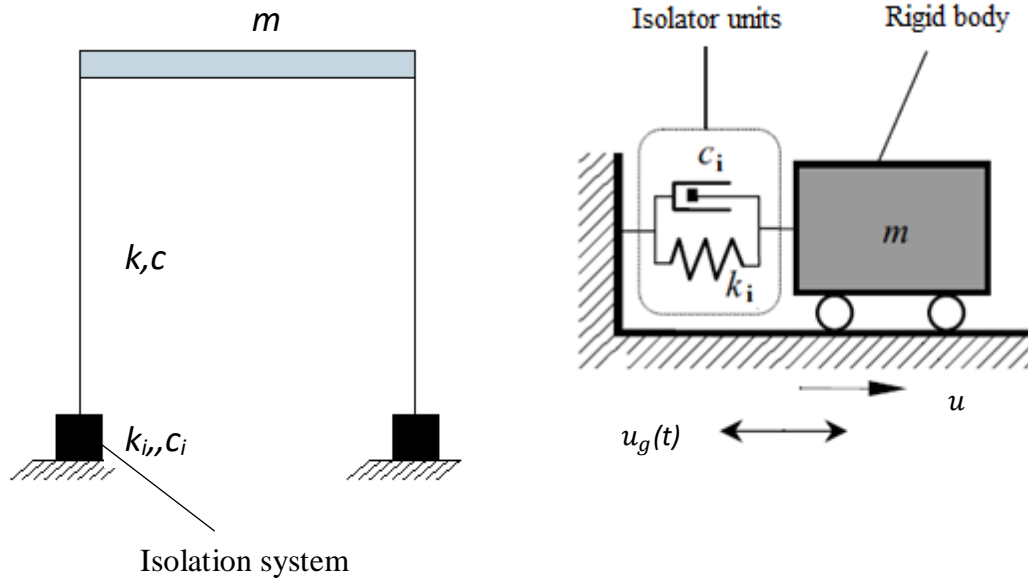


Figure 2-11. Sketch of SDOF seismically isolated structure (Cheng et al., 2008)

The equation of dynamic equilibrium, for each instant of time, of a linear SDOF structure subjected to earthquake excitation is described in the equation (2-1).

$$m \ddot{u}(t) + c_i \dot{u}(t) + k_i u(t) = -m \ddot{u}_g(t) \quad (2-1)$$

In this equation, i subscripts indicates that correspond to isolation system, m represents the mass of the superstructure, k_i is the lateral stiffness of the isolation system, and c_i is the damping coefficient. The relative displacements for both isolation systems and ground, are expressed as $u(t)$ and $u_g(t)$ respectively.

Introducing the expressions for angular frequency, $\omega_i = \sqrt{k_i/m}$ and critical damping coefficient, $\xi_i = c_i/2 \omega_i m$, equation (2-1) can be rewritten as:

$$\ddot{u}(t) + 2 \xi_i \omega_i \dot{u}(t) + \omega_i^2 u(t) = -\ddot{u}_g(t) \quad (2-2)$$

Making a numerical integration of equation (2-2), considering the total seismic input, divided in a series of impulses applied in t time and with a duration of dt , the solution showed in equation (2-3) is obtained.

$$u(t) = e^{-\xi_i \omega_i t} \left[u_o \cos \bar{\omega}_i t + \frac{\dot{u}_o + u_o \xi_i \omega_i}{\bar{\omega}_i} \sin \bar{\omega}_i t \right] - \frac{1}{\bar{\omega}_i} \int_0^t \ddot{u}_g(\zeta) e^{-\xi_i \omega_i (t-\zeta)} \sin \bar{\omega}_i (t-\zeta) d\zeta \quad (2-3)$$

$\bar{\omega}_i = \omega_i \sqrt{1 - \xi_i^2}$, is the damped angular frequency. Previous integrals are the particular solution of the system described in (2-2) and they are named convolution integrals or Duhamel integrals. If there are not initial movement and velocity of the studied structure at the beginning of the earthquake, the expression (2-3) can be simplified as show in equation (2-4)

$$u(t) = -\frac{1}{\bar{\omega}_i} \int_0^t \ddot{u}_g(\zeta) e^{-\xi_i \omega_i (t-\zeta)} \sin \bar{\omega}_i (t-\zeta) d\zeta \quad (2-4)$$

When the value of damping ratio does not exceed 20% of critical damping, the term $\bar{\omega}_i$ is replaced by ω_i . Therefore equation (2-4) can be rewritten as:

$$u(t) = -\frac{1}{\omega_i} \int_0^t \ddot{u}_g(\zeta) e^{-\xi_i \omega_i (t-\zeta)} \sin \omega_i (t-\zeta) d\zeta \quad (2-5)$$

The maximum absolute value of the integral is called pseudo-velocity S_V , therefore of equation (2-5), the spectral displacement S_D has the follow relation with the pseudo-velocity:

$$S_V = \omega_i S_D \quad (2-6)$$

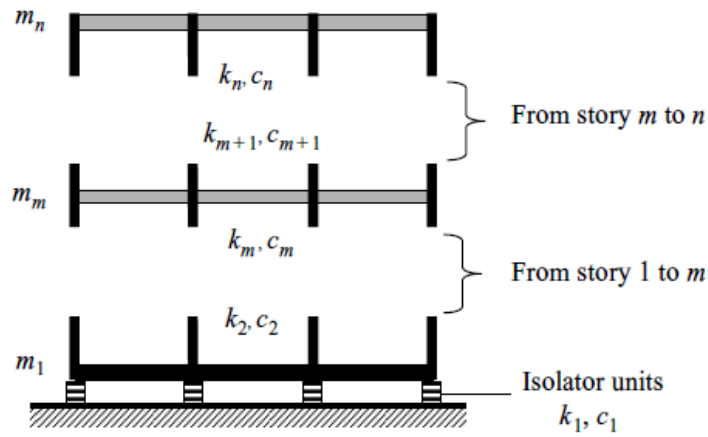
And the relation between the pseudo-acceleration S_A , and the spectral displacement, can be written like in equation (2-7).

$$S_A = \omega_i^2 S_D \quad (2-7)$$

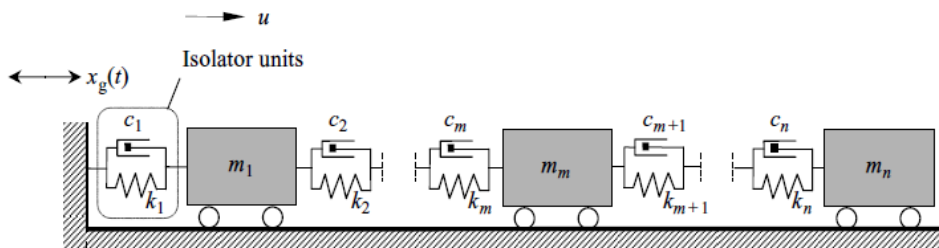
The term \ddot{u}_g is a problem's data, thus S_A , S_V , S_D are dependent of both damping ratio and angular frequency of isolation system. The equation (2-7) is a basic expression for develop an analysis of equivalent lateral force to design of seismically isolated structures.

2.2.4.2 Multiple degree of freedom

Figure 2-12, shows a seismically isolated structure with multiple floors, and its equivalent spring model.



a) Multiple floors model



b) Spring model

Figure 2-12. Sketch of MDOF seismically isolated structure (Cheng et al., 2008)

The equations of dynamic equilibrium for each instant of time, of the level n (roof), m , and 1 (above isolation system) are described in equations (2-8),(2-9),(2-10)

$$m_n \ddot{u}_n + c_n (\dot{u}_n - \dot{u}_{n-1}) + k_n (u_n - u_{n-1}) = -m_n \ddot{u}_g \quad (2-8)$$

$$m_m \ddot{u}_m + c_m (\dot{u}_m - \dot{u}_{m-1}) - c_{m+1} (\dot{u}_{m+1} - \dot{u}_m) + k_m (u_m - u_{m-1}) - k_{m+1} (u_{m+1} - u_m) = -m_m \ddot{u}_g \quad (2-9)$$

$$m_1 \ddot{u}_1 + c_1 (\dot{u}_1) - c_2 (\dot{u}_2 - \dot{u}_1) + k_1 (u_1) - k_2 (u_2 - u_1) = -m_1 \ddot{u}_g \quad (2-10)$$

It is important to note that in intermediate floors (m), the dynamic equation have the influence of both higher and lower floors. In the case of the story above the isolation system and of the roof, only the story above or below respectively affects the dynamic equation.

Previous equations, can be expressed in matrix notation as:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = -[M]\{\gamma\} \{\ddot{u}_g\} \quad (2-11)$$

The vector $\{\gamma\}$ contains unitary elements and indicated that degree of freedom represented by one of the horizontal equation of the simultaneous equation system is collinear with ground acceleration. The solution of the equation (2-11) can be expressed in terms of generalized response vector $\{u'\}$ and the modal matrix $[\phi]$.

$$\{u\} = [\phi] \{u'\} \quad (2-12)$$

Expression (2-11) can be rewritten in terms of equation (2-12) and its successive derivatives

$$[M][\phi] \{\ddot{u}'\} + [C][\phi] \{\dot{u}'\} + [K][\phi]\{u'\} = -[M]\{\gamma\} \{\ddot{u}_g\} \quad (2-13)$$

Multiplying and dividing both sides of the equation (2-13) by $[\phi]^T$ and $[\phi]^T [M][\phi]$ respectively, it is obtained.

$$\{\ddot{u}'\} + \frac{[\phi]^T [C][\phi]}{[\phi]^T [M][\phi]} \{\dot{u}'\} + \frac{[\phi]^T [K][\phi]}{[\phi]^T [M][\phi]} \{u'\} = -\frac{[\phi]^T [M]\{\gamma\}}{[\phi]^T [M][\phi]} \{\ddot{u}_g\} \quad (2-14)$$

Defining damping ratio at each mode, $\xi_m = c_m / 2 \omega_m m_m$, $m = 1, \dots, n$, The expression

$\frac{[\phi]^T [C][\phi]}{[\phi]^T [M][\phi]} = 2 \xi_m \omega_m$, which is an diagonal matrix $n \times n$. Theoretically, it cannot be decoupled into a diagonal matrix, since damping value of isolation system is higher than for

superstructure, however, the coupling damping effects (off diagonal components) are proved to be small and negligible for most structures (Kelly, 1993). If damping effects are not neglected, the complex modal analysis must be used to find the solutions. A diagonal matrix is also obtained with $\frac{[\phi]^T [K] [\phi]}{[\phi]^T [M] [\phi]} = [\omega_m^2]$.

The expression $\frac{[\phi]^T [M] \{\gamma\}}{[\phi]^T [M] [\phi]}$ is denoted like Γ_m and it is called mode participation factor. Because the orthogonal conditions considered, equation (2-14) can be uncoupled and is expressed for each m mode of vibration and each value of ground acceleration:

$$\ddot{u}'_m + 2 \xi_m \omega_m \dot{u}'_m + \omega_m^2 u'_m = - \Gamma_m \ddot{u}_g \quad (2-15)$$

Applying Duhamel's integral, the solution of equation (2-15) for each story, is obtained. It is important to note that the follow solution does not have into account the term $\sqrt{1 - \xi_m^2}$

$$\ddot{u}'_m (t) = -\frac{1}{\omega} \Gamma_m \int_0^t \ddot{u}_g(\zeta) e^{-\xi_m \omega_m (t-\zeta)} \sin \omega_m (t - \zeta) d\zeta \quad m = 1, \dots, n \quad (2-16)$$

The relative displacement vector $\{u\}$, the velocity vector $\{\dot{u}\}$, and the acceleration vector $\{\ddot{u}\}$ are estimated as follows:

$$\{u\} = [\phi] \{u'\} \quad (2-17)$$

$$\{\dot{u}\} = [\phi] \{\dot{u}'\} \quad (2-18)$$

$$\{\ddot{u}\} = [\phi] \{\ddot{u}'\} \quad (2-19)$$

When damping ratio values of isolation system are higher than 20% of critical value, the modal displacement superposition method applied here is not applicable (Cheng et al., 2008).

2.2.5 Isolators

Isolators are devices with high flexibility in horizontal directions to increment both structural period and structural damping and high rigidity in vertical direction to support gravitational loads. There are principally two groups of isolators.

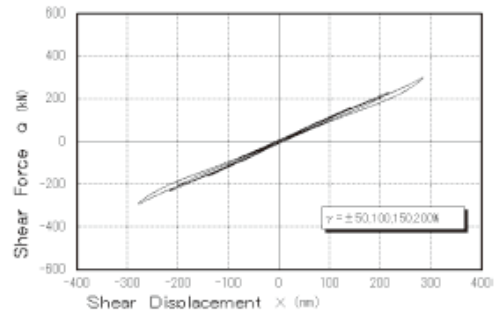
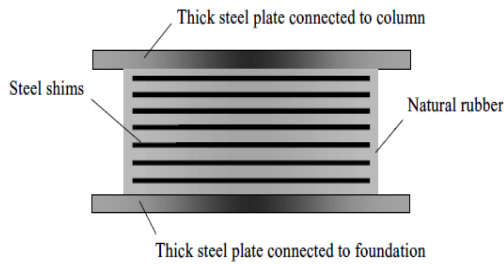
2.2.5.1 Elastomeric isolators

These isolators are typically composed of rubber and steel plates, some types of isolators combine characteristics of elastomeric and sliding bearings. The steel plates increment the vertical stiffness of the isolator and keeps the rubber layers from laterally bulging (Cheng et al., 2008). The thickness of steel and rubber depend on characteristics of both damping and stiffness required to the project, generally, thickness of rubber layers range between 4 and 10mm and thickness of steel plates range between 3 and 5mm. The bearing's stiffness in tension is many times less than in compression.

Elastomeric bearing has been widely used in a wide variety of structures. It is may be due, among other factors, to its excellent behavior during earthquakes and its lower cost. The main elastomeric isolators are called: natural rubber bearings (NRB), lead rubber bearings (LRB), high damping rubber bearings (HDRB) and sliding rubber bearings (SRB)

Natural rubber bearings (NRB)

NRBs have high horizontal flexibility, low level of damping (2-3% of critic damping) and a good restoring force. Because these devices have a low damping value, generally they are used with external dampers or other types of isolators (Bridgestone Corporation, 2013). The Figure 2-13 a) shows a sketch of NRB isolator and the Figure 2-13 b) shows the hysteretic behavior of a real test of this device, it is observed that the relation between force and displacement is almost linear.



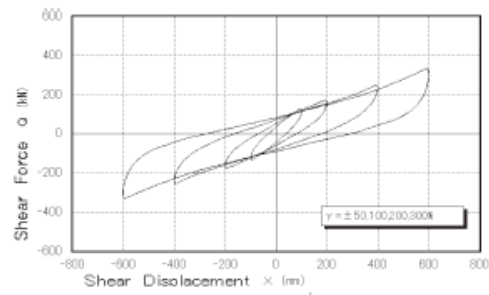
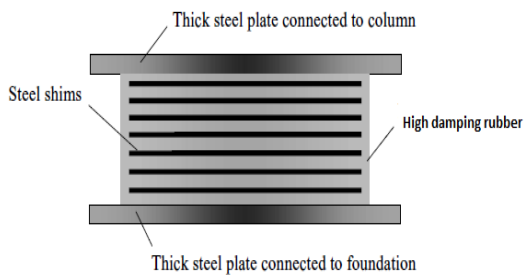
a) NRB isolator (Cheng et al., 2008)

b) Hysteretic behavior (Bridgestone Corporation, 2013)

Figure 2-13. NRB isolator

High damping rubber bearing (HDRB)

HDRBs are similar to NRBs, but the rubber compounds used in this device are modified, adding carbon black or other types of filler. This change results in higher damping values (10-20% of critic damping to 100% of shear deformation). This isolator has high horizontal flexibility and a good restoring force; therefore, they are used generally of independent way without dampers and additional isolators. The Figure 2-14 a) shows a sketch of HDRB isolator and the Figure 2-14 b) shows the hysteretic behavior of a real test of this device, it is observed that the relation between force and displacement is elastoplastic.



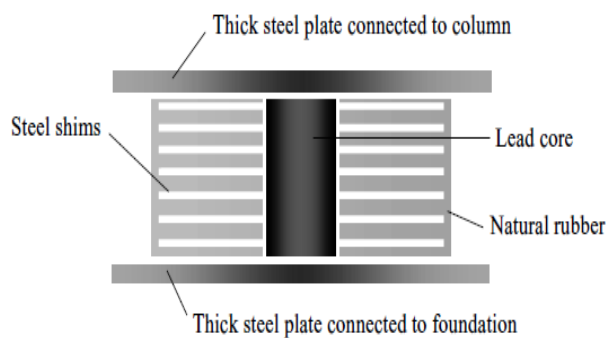
a) NRB isolator (Cheng et al., 2008)

b) Hysteretic behavior (Bridgestone Corporation, 2013)

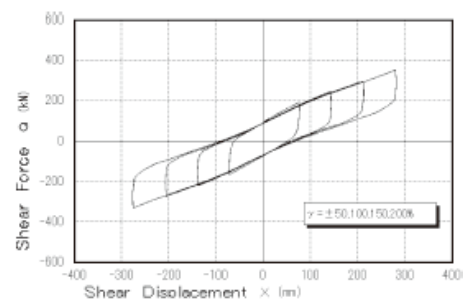
Figure 2-14. HDRB isolator

Lead rubber bearings (LRB)

Other way to increment damping to NRBs isolators, is including on central area of them a lead core. This lead core is pressured mounted in a hole with dimensions slightly smaller than core, because both a total fusion and an adequate behavior like structural unit are necessary. The behavior of this isolator depends on the lateral force applied, it means, when lateral forces are small the displacement of the steel plates is restrained by lead core, therefore the isolator shows high stiffness; when lateral forces are high the displacement of the steel plates makes that lead core yields, thus hysteretic damping is developed (15% to 35% of critical damping) (Cheng et al., 2008b) with the energy that is absorbed by the lead and the stiffness is reduced. The Figure 2-15 a) shows a sketch of LRB isolator and the Figure 2-15 b) shows the hysteretic behavior of a real test of this device, it is observed that the relation between force and displacement is elastoplastic.



a) LRB isolator (Cheng et al., 2008)



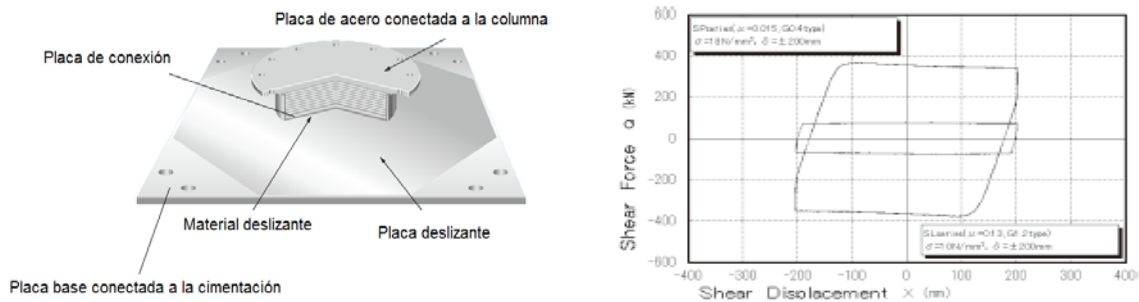
b) Hysteretic behavior (Bridgestone Corporation, 2013)

Figure 2-15. LRB isolator

Slider (SRB)

This device is a combination of natural isolator, slider material and a slider plate. The slider material is put on slider plate which is fixed to base plate. Small displacements are absorbed by rubber, while larger displacements cause the rubber bearing slides on the plate (Bridgestone Corporation, 2013). This isolator has high damping values, but it has not tension capacity neither restoring force. Generally, this device is used combined with other isolators (Trevor E Kelly, 2001). The Figure 2-16 a) shows a sketch of SRB isolator and the Figure 2-16 b) shows

the hysteretic behavior of a real test of this device, it is observed that the relation between force and displacement is almost elastoplastic perfect.



a) SRB isolator (Cheng et al., 2008)

b) Hysteretic behavior (Bridgestone Corporation, 2013)

Figure 2-16. SRB isolator

2.2.5.2 Friction pendulum bearings

With friction pendulum bearings (FPB) the lateral force is resisted by means of friction coefficient and the vertical load applied on the bearing. FPB generally has a spherical or concave sliding surface to provide to the system of return capacity to its original position after earthquake movement. This capacity is due to that a component of applied vertical load along the tangential direction helps the bearing move back to the center of the device. The spherical sliding surface is coated by a material with low friction value.

Characteristic values of damping for this device are between 10% and 40% regard to critic damping (Earthquake protection systems, 2018). Currently there are simple and triple friction pendulum systems, a sketch of them is showed in the Figure 2-17. Using a device with more concavities has the advantage that it is possible to obtain the same displacement of the system with smaller devices.

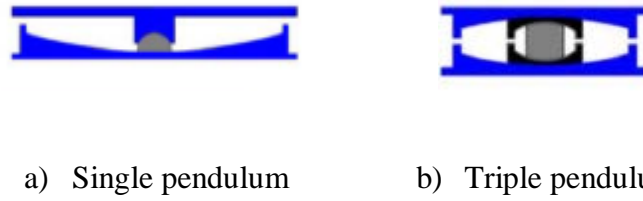


Figure 2-17. Friction pendulum bearings (Earthquake protection systems, 2018)

The properties of this type of isolators are defined in function of: curvature radius, friction coefficient and vertical weight on bearing. The hysteretic behavior of FPB is a combination of restoring forces (F_r) plus friction forces (F_f), such as view in Figure 2-18.

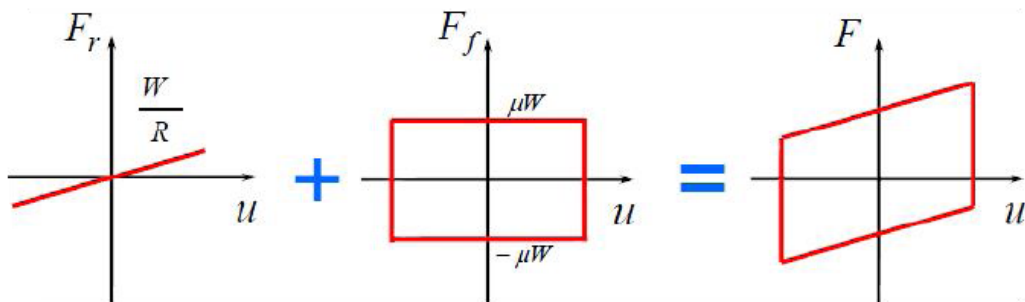


Figure 2-18 . Hysteretic behavior of FBP isolator

Results of a real test of a FPB isolator are showed in Figure 2-19.

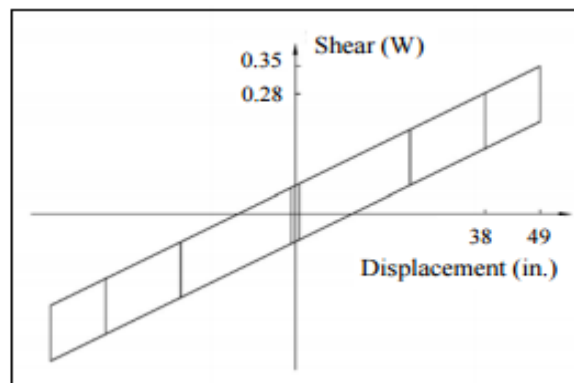


Figure 2-19. Hysteretic behavior, real test on FBP isolator

2.2.5.3 Dampers

Behavior of viscous dampers is represented through Maxwell's model; whose constitutive law is $F = k_{oil} x + c v^\alpha$ (Silvestri et al., 2010). In this expression, F is the force developed by the damper, k_{oil} is the stiffness that represent the compressibility of the fluid, x is the displacement of the damper, c is the damping coefficient, v is the velocity and α is the exponent which is equal to one for lineal behavior and smaller than one for nonlinear behavior

2.2.6 Mechanical properties of isolators

In the chapter 2.2.4 the linear theoretical formulation has a series of approximations, which cannot be true for some cases like: high irregularity for the superstructure, soft soils foundations that can be generate serious impact on superstructure and seismic isolation system, proximity of major active faults, etc. Theoretically, the nonlinear behavior of isolated structures is concentrated in isolators due to its flexibility and the superstructure can be reasonably assumed to exhibit linear response due to higher stiffness. This consideration is used for design, and provide required accuracy (Cheng et al., 2008)

To represent the nonlinear behavior both of elastomeric and friction bearings, the bilinear model is used. This model (Figure 2-20) is widely applied both in research and professional practice. Bilinear model is totally defined through three parameters: horizontal elastic stiffness (k_e), plastic stiffness (k_p), and characteristic force (Q), parameters which depend on isolator type. Characteristic force is used to define the stability of hysteretic behavior when the isolators are exposed to many loading cycles. Additional parameters like effective stiffness (k_{eff}), yield force (F_y), yield displacement (D_y) and Energy dissipated (E_D) are showed too in Figure 2-20.

Where: A_p = lead core area, A_r = rubber area, D = displacement of isolator system, f_{yp} = shear stress at yield of lead, G_r = shear modulus of rubber, H = total rubber thickness, *N.A* = Not applicable, R = curvature radius, W = vertical load, α_p = apparent shear modulus of lead, β_{eff} = effective damping, μ_s = friction coefficient, λ factor range between 0.05 and 1. Both effective stiffness and effective damping are expressed for a defined displacement (D)

It is important to note that the mechanical properties of the isolators vary due to heating, rate of loading, scragging, aging, environmental conditions, and manufacturing irregularities. Seismic codes generally include this effect through parameters that modifies the nominal properties of isolators to obtain minimum and maximum values of them.

3. SURVEY ON MAJOR WORLDWIDE REGULATIONS ON BASE ISOLATION OF BUILDINGS WITH RUBBER BEARINGS

Base isolation is an efficient solution for seismic protection of buildings. However, referring figures in Table 2-3 to number of people in seismic zones and to level of development, worldwide isolation implementation is highly uneven, despite the high seismicity of all the considered countries. This might be partly due to differences in levels of exigency of design codes (Feng et al., 2006; Higashino and Okamoto, 2006; Yenidogan and Erdik, 2016). Nations like Japan, China, Russia Italy and USA, are the leaders in the application of seismic isolation. Conversely, in other countries included South American countries this technique is significantly less common.

A crucial step in the promotion of any earthquake-resistant construction technique is the development of design codes that, although being inspired in the major international regulations, account for the local peculiarities, mainly in terms of seismicity. With the aim of assisting the code developers, this chapter analyzes code requirements for base isolation of Japan, China, Russia, Italy, USA, Chile and México, aiming to compare their level of exigency. This study focusses on rubber bearings, given their economy, satisfactory performance, robustness and low maintenance requirements (Cheng et al., 2008a; Pan et al., 2005). General and particular comparisons are performed, this last on an example of a sanitary building with seismic isolation. General assessment is carried out in terms of analysis and design procedures, return period of the design input, importance factor, response reduction factor due to damping, design spectra, and design displacements and forces. For the particular evaluation, the aforementioned sanitary building and the isolation layer are designed, according to USA code (ASCE 7-10, 2010), for a high seismicity zone (Los Angeles) and a medium one (New Mexico). After these designs, the major demanding design parameters according to the other analyzed regulations are determined and compared; static equivalent and nonlinear time-history analyses (using artificial accelerograms fitted to design spectra) are performed. These parameters are: forces on superstructure and substructure, isolation layer displacement, and superstructure forces for drift verification. Given that Italian code (NTC, 2008) allows considering several importance factors, situation for housing use is also analyzed.

Results show serious discrepancies among compared codes in relevant aspects such as performance of isolation buildings, seismic hazard level, and Analysis and design procedures.

3.1 Comparison among base isolation regulations

3.1.1 General considerations

This section presents a comparison among seismic isolation regulation for Japan (BSL, 2009), China (GB 50011, 2010) , Russia (SP 14.13330, 2014), Italy (NTC, 2008) , USA (ASCE 7-10, 2010; ASCE 7-16, 2016) ,Chile (NCh 2745, 2013a) and México (Manual de diseño de obras civiles, 2008). In the US, both current (ASCE 7-16, 2016) and previous (ASCE 7-10, 2010) regulations are analyzed. Next subsection describes analysis and design methodologies, and following subsections discuss each analyzed item.

For better understanding, it should kept on mind that ordinarily design starts by selecting desired values of first mode period and damping of the seismically isolated building. Typically, period ranges between 2 and 3 s and damping ranges between 20 and 30%.

3.1.2 Performance of isolated structures

In all codes, the seismic isolation wants a level of performance higher than in conventional structures. The performance of the superstructure must be near to elastic (immediate occupation performance), while in conventional building the structure will experiment damage and its performance will be inelastic (life security performance). In the Mexican code a total seismic isolation case (linear elastic structure) and a partial seismic isolation case (low inelastic performance) is permitted.

3.1.3 Analysis and design procedures

Analysis and design methodologies for base isolated buildings are basically the same than for ordinary (fixed base) ones: Static linear analysis (single mode), Modal spectral analysis (multi mode), and Nonlinear time-history analysis. In some codes is possible the use of nonlinear static analysis (Ej, Mexico)

Static linear analysis. This approach can be considered provided that some conditions are fulfilled. Building height is limited to 20 m (Chile and USA-ASCE 7-10, Mexico), 40 m (Japan) and 60 m (China). There is no height limitation in the new American code (ASCE 7-16, 2016). Japanese and Chinese codes state that isolators should be located in the base of the building. Regular super structural configuration and damping values smaller than 30% are too some requirements of some codes. This formulation uses a 5% damping design spectrum; influence of higher damping is represented by a reduction coefficient. Commonly, this method is used for preliminary design. Noticeably, using this analysis, only US and Chilean codes permit tension in isolators.

Modal spectral analysis. Requirements are less strict. Design spectrum corresponds to damping 2% for short periods (higher modes, linear behavior is sought) and to significantly higher damping ratios for long periods (mode involving only deformation in isolation layer).

Nonlinear time-history analysis. There are no requirements. All codes oblige to consider a certain number of pairs of accelerograms (acting simultaneously); this number is 3 in Chilean, Chinese and former US codes, 6 in Japan, 3 to 7 in Italy and Mexico, and 7 in new US code. Russian code does not contain any prescription; apparently, 7 accelerograms are used in practice (Mkrtychev et al., 2015). Except in Japan and China, nonlinear behavior is concentrated in isolator units; conversely, Japanese and Chinese regulations allow considering nonlinear behavior of superstructure. Nonlinear time-history analyses are widely used in Japan and China (Gao et al., 2013b, 2013a), although the proposed strategies are more simplified than in the other codes. In Chilean and US regulations, base shear from static linear analysis can be only slightly reduced using nonlinear time-history analysis.

3.1.4 Seismic hazard level

Table 3-1. Return period of the design input (years)

Country	Superstructure	Isolation system	Seismic gap
Japan	500	500	2500
China	1600 – 2500	1600 – 2500	
Russia	1000 – 5000	1000 – 5000	
Italy	475 – 950	975 – 1950	
USA (ASCE 7-10)	475	2475	
USA (FEMA P1050)	2475	2475	
Chile	475	950	

Hazard is expressed in terms of return period; Table 3-1 presents a summary of requirements. For substructure, all codes indicate that return period should be the same than in superstructure, although with smaller R . Most codes recommend $R = 1$; Chilean code allows up to 1.5. This parameter is variable in Mexico in function of location and type of structure.

3.1.5 Importance factor

Japanese code does not contain any prescription; usually 1.25 / 1.5 is considered in public buildings and in essential facilities, respectively (Pietra et al., 2014). Chinese code does not include any factor. Russian codes states importance factors 1 / 1.5 / 2 for structures of normal / high and exceptional importance, respectively. Italian code proposes equal coefficients than for fixed-base buildings. In USA and Chilean regulations do not deal with this issue ($I=1$). In Mexico there is no special consideration for this factor in the case of seismic isolation.

3.1.6 Reduction factor due to damping

Since base isolation permits important damping increases, this issue is relevant. Expressions for each country follow.

Japan
$$F_h = \frac{1.5}{1 + 10 (h_v + 0.8 h_d)} \geq 0.4 \quad (3-1)$$

In equation (3-1) , h_v and h_d are viscous and hysteretic damping factors, respectively; for 5% damping, $h_v + 0.8 h_d = 0.05$.

China
$$\gamma = 0.9 + \frac{0.05 - \xi}{0.3 + 6 \xi} \quad \eta_1 = 0.02 + \frac{0.05 - \xi}{4 + 32 \xi} \geq 0.0 \quad (3-2)$$

$$\eta_2 = 1 + \frac{0.05 - \xi}{0.08 + 1.6 \xi} \geq 0.55$$

In these expressions ξ is damping factor; use of γ , η_1 and η_2 is described in equation

Italy
$$\eta = \left(\frac{10}{5 + 100 \xi} \right)^{1/2} \geq 0.55 \quad (3-3)$$

USA
$$\frac{1}{B} = 0.25(1 - \ln \xi) \quad (3-4)$$

Chile
$$\frac{1}{D} = B_0 - (B_0 - 1) \exp(-a T_D |\beta - 0.05|) \quad B_0 = \frac{2(1 + \beta)}{1 + 14.68 \beta^{0.865}} \quad (3-5)$$

México
$$\beta = \left(\frac{0.05}{\xi} \right)^\lambda = \begin{cases} 0.45 & \text{Si } T_e < T_c \\ 0.45 \frac{T_c}{T_e} & \text{Si } T_e \geq T_c \end{cases} \quad (3-6)$$

In equation (3-5), T_D is soil period, β is damping factor; values of coefficient a are listed in Table 3-2. Alternatively, to equation (3-5) , equation (3-4) can be used, this being a more conservative approach. The Russian code does not contain any expression. In equation (3-6) T_e = structural period and T_c = period of begin of displacements constant zone in the spectra.

Table 3-2. Coefficient a in the Chilean code

β	Soil I	Soil II	Soil III
0.10	396.9	293.1	224.5
0.15	180.7	124.6	98
0.20	117.9	76.1	57.1
0.25	94.0	54.3	39.6

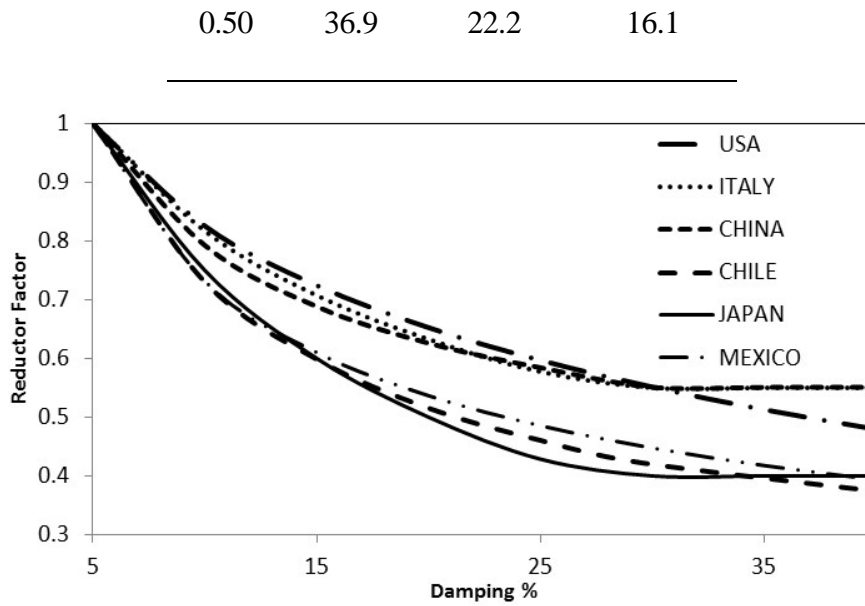


Figure 3-1. Reduction factor due to damping

Figure 3-1 displays the response reduction factor due to damping for each country; for China, η_2 is plotted, For Chile $\lambda=0.45$ is used. Figure 3-1 shows that factors for Japan and Chile and Mexico are significantly smaller than the other ones.

3.1.7 Design spectra

Japan. Spectral acceleration S_a is given by (3-7). Z is zone factor (ranging between 0.7 and 1), G_s (T) is soil amplification factor (Figure 3-2), and S_0 is spectral acceleration in bedrock (Table 3-3).

$$S_a = Z G_s(T) S_0(T) \quad (3-7)$$

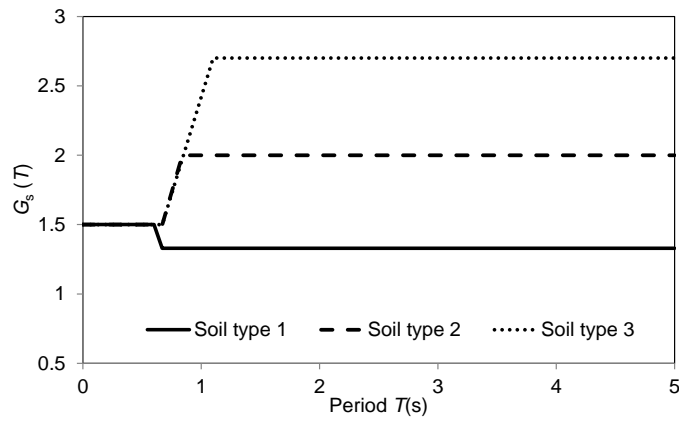


Figure 3-2. G_s Factor (Japan)

Table 3-3. Spectral acceleration in bedrock (S_0) according to Japanese code (m/s^2)

Period range	Level 1	Level 2
$T < 0.16 s$	$0.64 + 6 T$	$3.2 + 30 T$
$0.16 s \leq T < 0.64 s$	1.6	8.0
$0.64 s \leq T$	$1.024 / T$	$5.12 / T$

China. Design spectrum S_a obeys to equation (3-8); η_1 , η_2 and γ depend on damping factor (equation (3-8)), T_g is soil characteristic period and α_{max} is a factor related to the seismic intensity (Table 3-4).

$$\begin{array}{llll}
 0.45 \alpha_{max} & T = 0 & \eta_2 \alpha_{max} & 0.1 \leq T \leq T_g \\
 \left(\frac{T_g}{T}\right)^\gamma \eta_2 \alpha_{max} & T_g \leq T < 5 T_g & (\eta_2 0.2^\gamma - \eta_1 (T - 5 T_g)) \alpha_{max} & 5 T_g \leq T \leq 6
 \end{array} \quad (3-8)$$

Table 3-4. Parameter α_{max} of Chinese code

Hazard level	Intensity			
	6	7	8	9
Frequent Earthquake	0.04	0.08–0.12	0.16–0.24	0.32
Design Earthquake	0.05	0.10–0.15	0.20–0.30	0.40
Maximum Earthquake	0.28	0.50–0.72	0.90–1.20	1.40

Russia. Design spectra β_i are defined by equation (3-9) for soil type I and II (top row) and III and IV (bottom row). Values of β_i cannot be less than 0.8.

$$\begin{array}{llll} 1 + 15 T & T \leq 0.1 \text{ s} & 2.5 & 0.1 < T < 0.4 \text{ s} & 2.5 (0.4/T)^{0.5} & T \geq 0.4 \text{ s} \\ 1 + 15 T & T \leq 0.1 \text{ s} & 2.5 & 0.1 < T < 0.8 \text{ s} & 2.5 (0.8/T)^{0.5} & T \geq 0.8 \text{ s} \end{array} \quad (3-9)$$

Italy. Design spectrum is given by equation (3-10).

$$\begin{array}{llll} a_g S \eta F_0 \left[\frac{T}{T_B} + \frac{1}{\eta F_0} \left(1 - \frac{T}{T_B} \right) \right] & 0 \leq T < T_B & a_g S \eta F_0 & T_B \leq T < T_C \\ a_g S \eta F_0 \frac{T_C}{T} & T_C \leq T < T_D & a_g S \eta F_0 \frac{T_C T_D}{T^2} & T_D \leq T \end{array} \quad (3-10)$$

In equation (3-10), a_g is acceleration at bedrock, S is soil coefficient, η is defined in equation (3-3), and F_0 is maximum spectral amplification factor, depending on location and ranging between 2.40 and 2.71. Regarding periods, $T_C = C_C T_C^*$, $T_B = T_C / 3$, and $T_D = a_g / g + 1.6$. C_C depends on soil type and T_C^* depends on location, ranging between 0.15 and 0.56.

USA. Design spectrum obeys to equation (3-11), where S_{DS} and S_{D1} are design acceleration for short periods and for 1 s, respectively.

$$\begin{array}{llll} S_{DS}(0.4 + 0.6 T / T_0) & 0 \leq T < T_0 & S_{DS} & T_0 \leq T \leq T_S \\ S_{D1}/T & T_S < T \leq T_L & S_{D1} T_L/T^2 & T > T_L \end{array} \quad (3-11)$$

In equation (3-11), $S_{DS} = (2/3) F_a S_s$ and $S_{D1} = (2/3) F_v S_1$, where S_s and S_1 are design accelerations (MCE) for short periods and 1 s, respectively. F_a and F_v are site coefficients. Regarding periods, $T_0 = 0.2 S_{D1} / S_{D2}$ and $T_S = 5 T_0$. Period T_L depends on location, ranging between 4 and 16 s; noticeably, T_L is extraordinarily high, thus having little applicability.

Chile. Chilean code proposes a design spectrum which is specific for base isolation:

$$\begin{array}{llll} \frac{\alpha_A A - A}{T_b - T_a} (T - T_a) + A & T_a \leq T \leq T_b & \alpha_A A & T_b < T \leq T_c \\ (2 \pi/T) \alpha_V V & T_c < T \leq T_d & (2 \pi/T)^2 \alpha_D D & T > T_d \end{array} \quad (3-12)$$

The required parameters are listed in the code for soils I, II and III; for soil IV, a specific site spectrum is required.

Mexico: There are two types of spectra. The first one for collapse limit state and the other one for service limit state. This second one is estimated in function of the first spectra described in the equation (3-13).

$$\begin{aligned}
 & a_o + (\beta c - a_o) \frac{T_e}{T_a} & T_e < T_a & & \beta c \left(\frac{T_b}{T_e} \right)^r & & T_b \leq T_e < T_c \\
 & \beta c & T_a \leq T_e < T_b & & \beta c \left(\frac{T_b}{T_c} \right)^r \left[k + (1 - k) \left(\frac{T_c}{T_e} \right)^2 \right] \left(\frac{T_c}{T_e} \right)^2 & & T_e \geq T_c
 \end{aligned} \tag{3-13}$$

T_a , T_b are period limits of the plateau, T_c is the period corresponding to begin of zone of constant displacements, a_o is the maximum soil acceleration. K is a parameter that controls the fall in the descendent zone for large periods, β is the damping factor, c is maximum spectral ordinate.

3.1.8 Comparison among design spectra

Figure 3-3 compares spectra. All spectra correspond to damping 5%, importance factor 1, response reduction factor $R = 1$, and soil type C (according to USA codes) with $v_{s,30} = 500$ m/s (average shear wave velocity). Figure 3-3 displays spectra normalized with respect to their zero-period ordinates. Figure 3-3 shows that, for the range of periods of interest for isolated buildings, Russian spectrum has highest ordinates while Italian and ASCE 7-16 spectra have lowest.

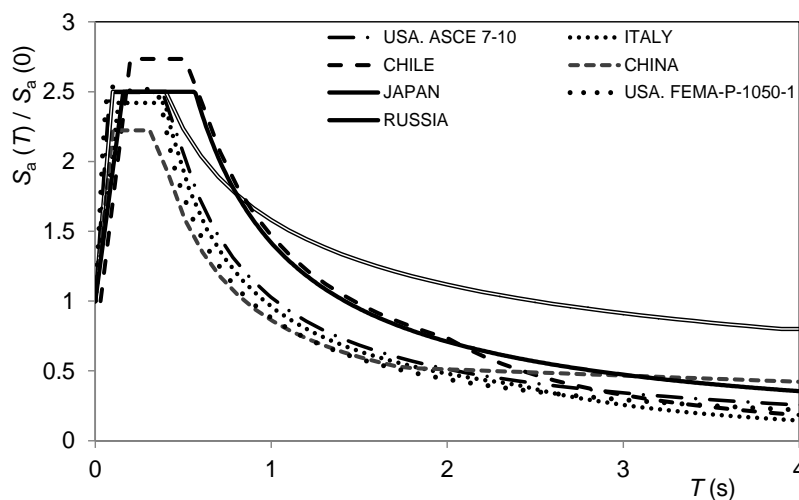


Figure 3-3. Design spectra for different codes

The differences among the spectral ordinates are due basically to the differences among the site effect factors considered for each code.

3.1.9 Design Displacements and Forces in static linear method

After the instruments discussed in the previous subsections, following major design quantities are studied: design displacement of isolators (D), total design displacement of isolators (D_T), and force (F_Δ) for obtaining drift limit (Δ_{lim}). The design displacement of isolators corresponds to expected drift in the isolation layer for a given return period; is used to determine design force for superstructure (F_{sup}), through constitutive law of isolators. Total design displacement of isolators corresponds to design displacement incremented with building torsion; is used to design isolator units and to define seismic gap. Design force for substructure (F_{sub}) is determined as for superstructure although for $R = 1$ (except in Chile, where 1.5 is allowed).

The prescriptions related to the drift limit (Δ_{lim}) are listed next.

Japan. Drift limit (level 1) in superstructure is $1/200$ for $H < 13$ m and $1/300$ for $H \geq 13$ m.

China. For frequent earthquake drift limit range between $1 / 100$ and $1 / 300$; for maximum earthquake, range between $1 / 120$ and $1 / 50$.

Russia. Drift limit coincides with Italian prescriptions.

Italy. In superstructure, drift limit is $2/3$ of the one for fixed-base buildings. In buildings with brittle partitions which are rigidly connected to the structure, this limit is 0.5%, otherwise is 1%. In unreinforced/reinforced masonry buildings, drift limit is 0.3/0.4%.

USA. Drift limit for linear/nonlinear analyses is 1.5/2%.

Chile. Drift limit in superstructure is 0.2%. It incorporates the response modification factor R .

Regarding forces for obtaining drift limit (F_Δ), in China and Chile codes $F_\Delta = F_{sup}$. In Japan and Italy, F_Δ corresponds to Level 1 and SLD, respectively. In USA, $F_\Delta = F_{sup} R$.

Table 3-5 summarizes the prescriptions of each code for D , D_T and F_{sup} .

Table 3-5. Design displacements and forces

Country	D	D_T	F_{sup}
Japan	$1.2 M F_h S_a / K_e$	$1.1 D$	$1.3 D K_e$
China	$S_a \beta M / K_e$	$D \left[1 + x \frac{12 e}{b^2 + d^2} \right]^{(*)}$	$0.85 S_a \beta M$
Italy	$S_a M / K_{esi \min}$	$D \left[1 + \frac{e}{r_x^2} x \right]^{(*)}$	$S_a M / R$
USA (ASCE 7-10)	$g S_{D1} T_D / 4 \pi^2 B$	$D \left[1 + x \frac{12 e}{b^2 + d^2} \right] \geq 1.1 D^{(*)}$	$D K_{e \max} / R$
USA (ASCE 7-16)	$g S_{M1} T_M / 4 \pi^2 B$	$D \left[1 + \frac{x}{P_r^2} \frac{12 e}{b^2 + d^2} \right] \geq 1.1 D^{(*)}$	$K_M \frac{D}{R} \left(\frac{W_s}{W} \right)^{1-2.5 \beta}$
Chile	C_D / B_D	$D \left[1 + x \frac{12 e}{b^2 + d^2} \right]^{(*)}$	$D K_{e \max} / R$

(*) These expressions correspond to x direction; relations for y direction are analogous.

The meanings and characteristics of elements in Table 3-5 are described next. Regarding D , M is superstructure mass, K_e is isolation layer effective stiffness. By assimilating dynamic behavior of isolated building to a SDOF system, K_e is related to fundamental period by means of

$$K_e = \frac{4 \pi^2 M}{T^2} \quad (3-14)$$

Reduction factor β for China is obtained after ratio between base shear under isolated and fixed-base conditions, ranging between 0.25 and 0.75. Chinese code states that F_{sup} cannot be lower than base shear of a fixed-base building under a seism with intensity 6 Table 3-4 (Pan et al., 2012). In Italian code, $K_{esi, \min}$ is minimum equivalent (secant) stiffness of isolation layer with respect to variability of its mechanic parameters. In US regulations, T_D and T_M are fundamental periods of isolated building for design and maximum displacements, respectively. In Chilean code, C_D depends on soil and seismic zone; for soil I/II/III, $C_D = 240 Z / 360 Z / 396 Z$, respectively. Z ranges between 3/4 and 5/4.

In Table 3-5, expressions for D_T represent a simplified way to take torsion effects into consideration. There, x and y are distances between the center of rigidity of the isolation system and the analyzed bearing; these distances are measured perpendicular to the input direction. Also, e is the actual eccentricity between the center of mass of the superstructure and the center of rigidity of the isolation system, plus the accidental eccentricity; it should be taken as 5%. Then, b and d are shortest and longest plan dimensions, respectively. Finally, r_x and r_y are torsional radii in x and y directions, respectively; P_r is ratio between effective translational and torsional periods. Once D_T is set, main verification criterion of isolator units is confirming that demanding factored compression and tension axial loads do not exceed the corresponding critical values. Load combinations used in compression and tension are $(1.2 + 0.2 S_{MS}) D + Q_E + L$ and $0.8 D - Q_E$, respectively. In these expressions, S_{MS} is the spectral response acceleration parameter at short periods, D and L are dead and live loads

and Q_E is the maximum considered earthquake effect. The other regulations consider some different prescriptions; for instance, European codes consider $G + \psi_E Q + E$, where G , Q , and E play the role of D , L and Q_E , respectively, and ψ_E is a combination coefficient (< 1). This points out that, regarding design of isolators, American codes are significantly more exigent than European ones. Another design criterion for isolator units is maximum allowable shear strain; since it ranges commonly between 100% and 400%, usually this condition is less demanding.

Regarding F_{sup} , in Japan and China response modification factor due to ductility is represented indirectly for drift limits. In Russia, $R = 1$. In Italy, $R = 1/1.5$ for serviceability conditions/ultimate limit state. In the USA, R is $3/8$ of value for fixed-base condition; moreover $1 \leq R \leq 2$. In Chile, $R = 2$ for any structure, except 1.6 for eccentric bracing and 1.4 for cantilevers. In former USA code, $K_{e,max}$ is maximum equivalent (secant) stiffness of isolation layer. In current USA code, K_M is equivalent stiffness of isolation layer corresponding to maximum displacement (MCE); W/W_s are seismic weights with/without base level weight. Finally, β is first mode damping factor (%).

Prescriptions for USA and Chile in Table 3-5 show that there are relevant differences in the generation of expressions for D . Chilean code assumes that fundamental period of isolated building lies in constant displacement branch ($T \geq T_d$). Conversely, in USA regulations it is assumed that 1 s period corresponds always to constant velocity branch. This circumstance is relevant, given that in some cities with soft soils, this period can correspond to constant acceleration branch.

In Japan, Italy and Chile, F_{sup} is distributed almost uniformly among stories. Chinese and old USA codes propose approximately triangular distribution; because the possible influence of higher modes. New USA code considers a distribution proportional to mass and to h^k ; h is height above the isolation interface and exponent k is given by $k = 14 \beta_M T_{fb}$ where β_M is effective damping for maximum earthquake and T_{fb} is fundamental period of building under fixed base conditions (Ryan and York, 2007).

3.1.10 Variation of Design Parameters of Isolator Units

Parameters of rubber bearings vary due to heating, rate of loading, scragging, aging, environmental conditions, and manufacturing irregularities. In static linear method, Japanese code proposes multiplying D for 1.2 (Table 3-5). Chinese and Russian regulations do not include any specific prescription. Italian code refers to the corresponding European regulation (EN 15129, 2009). This last document proposes a conservative formulation, to be used when no more specific information is

available. Major mechanical parameters of rubber bearings are modified with a factor λ that accounts for aging, heating, contamination and cumulative travel; λ factor affects stiffness and yielding force. Final value of λ is obtained by multiplying those for aging, heating, contamination and cumulative travel. Maximum value of λ for stiffness of NRB is 1.65. The old USA code deals only with variations due to manufacturing; it states that ratio between maximum and minimum stiffness of isolators shall not exceed 1.3 (FEMA 451, 2006). Conversely, new USA code contains a wider set of recommendations proposes a factor λ that accounts for all the aforementioned issues; both maximum and minimum values of λ need to be considered. In NRB, λ factor affects stiffness; maximum and minimum values are 1.83 and 0.77. In LRB, λ factor affects post-yield stiffness and yielding force; maximum and minimum values are 1.83/1.84 and 0.77 (1.83 and 1.84 correspond to post-yield stiffness and yielding force). Chilean code follows basically old USA regulation. In calculation of design displacement for isolators in old USA code (Table 3-5), T_D is obtained for the minimum value of stiffness of isolation layer; conversely, B is determined for the maximum value of such stiffness. Therefore, this approach has some inconsistency, since D is proportional to T_D and inversely proportional to B . In the new USA code, T_D and B are determined for the same stiffness. Maximum and minimum values of it are considered; among the two obtained displacements, highest one is chosen. This is considered more consistent.

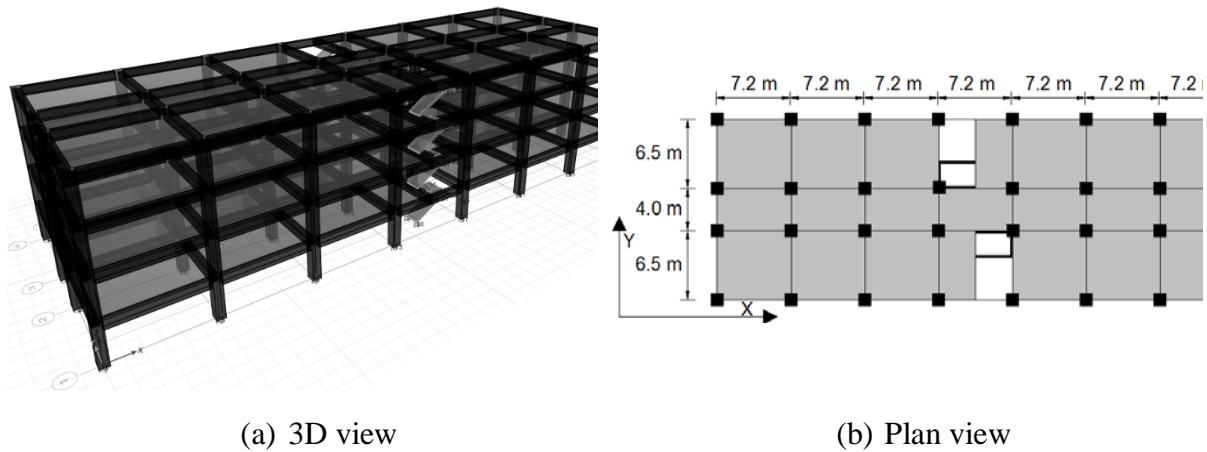
3.2 Example of sanitary building

3.2.1 General considerations

A RC sanitary prototype building is analyzed. Two versions located in Los Angeles and New Mexico are considered; represent high and medium seismicity, respectively. Superstructure and isolation layer are designed according to ASCE 7-10 and their performance is assessed for the other analyzed codes with the “Static linear analysis” and the “Nonlinear time-history analysis” methods. Since Italian code allows considering different importance factors, housing use is also considered.

3.2.2 Prototype building and isolation system

Prototype building is described in Figure 3-4; has four stories, and story height is 3 m. The structure is a 3-D RC frame.



(a) 3D view

(b) Plan view

Figure 3-4. Prototype building

Prototype building has important features of sanitary facilities (FEMA 577, 2007): (i) moderate height, (ii) horizontal architecture model, aiming to facilitate access and circulation, (iii) large span-length, for better use flexibility, (iv) redundant and spacious vertical connections (stairs, elevators, ramps), (v) wide horizontal connections (e.g. corridors) inside each story.

Two types of isolation units are used: natural rubber bearings (NRB) and lead rubber bearings (LRB); moreover, viscous dampers are incorporated in Los Angeles building. Linear and bilinear models represent the behavior of NRB and LRB, respectively. Damper behavior is described with Maxwell model $F = K_{oil} x = c v^\alpha$ (Silvestri et al., 2010); F is force exerted by device, K_{oil} is stiffness representing oil compressibility, x is displacement, c is damping coefficient, v is velocity, and α is an exponent.

3.2.3 Generation of seismic inputs for time-history analysis

Seismic inputs are pairs of artificial accelerograms fitting design spectra corresponding either to Los Angeles or New Mexico seismicity; according to (EN-1998-2, 2004), two different inputs are selected for each horizontal direction. Italian code states that a minimum of 3 pairs of accelerograms should be used and Chilean and old USA codes indicate that the number can be 3 or 7; depending on this choice, maximum or average response shall be considered. Three inputs is disregarded since some results are not satisfactory, given excessive influence of discordant results. Therefore, 7 pairs of accelerograms are generated for each country. Each pair of inputs is used for determining a design parameter: D_T , F_{sup} , F_{sub} or F_Δ . Given that Italian code allows considering different importance levels, in Italy number of inputs is doubled. The number of accelerograms is: 8 (countries) \times 7 (pairs) \times 2 (directions) \times 2 (locations) \times 4 (design parameters) = 896 accelerograms

Accelerograms are created to fit design spectra corresponding to each situation. Spectral ordinates are modified with factor $(T/T_R)^{0.3}$, where T_R is reference period (EN-1998-2, 2004). Regarding location, Los Angeles and New Mexico seismicity is represented by its zero-period spectral ordinate $S_a(0)$. Regarding design parameter, spectra are generated for return period stated in the corresponding code (Table 3-1); influence of response modification factor (R) is accounted for by dividing spectral ordinates by R . Table 3-6 displays considered values of T and R .

Table 3-6. Return periods (years) and response modification factor for generation of the accelerograms in the example

Country	D_T		F_{sup}		F_{sub}		F_Δ	
	T	R	T	R	T	R	T	R
Japan	500	1	500	1	500	1	50	1
China	2500 (0.4 g) 2000 (0.2 g)	1	2500 (0.4 g) 2000 (0.2 g)	1	2500 (0.4 g) 2000 (0.2 g)	1	2500 (0.4 g) 2000 (0.2 g)	1
Russia	1000	1	1000	1	1000	1	1000	1
Italy (hospital)	1950	1	950	1.5	950	1	100	1
Italy (housing)	975	1	475	1.5	475	1	50	1
USA (ASCE 7-10)	2475	1	475	$3/8$ $R^{(*)}$	475	1	475	1
USA (ASCE 7-16)	2475	1	2475	$3/8$ $R^{(*)}$	2475	1	2475	1
Chile	950	1	475	2	475	$1/5$	475	2

(*) R corresponds to fixed base condition; resulting value is further bounded: $1 \leq 3/8 R \leq 2$

Inputs are generated for 20 s duration (NUREG-0800, 2014). Amplitude vs. time variation responds to function described in (Saragoni and Hart, 1973); maximum amplitude corresponds to 4 s and final instant amplitude is 5% of maximum one (Saragoni and Hart, 1973). Figure 3-5 displays an example of an accelerogram whose response spectrum fits design spectrum of current USA code Design spectrum. Figure 3-5.b corresponds to design parameter F_{sup} and to New Mexico seismicity. Figure 3-5.b highlights great similarity between design and individual spectra.

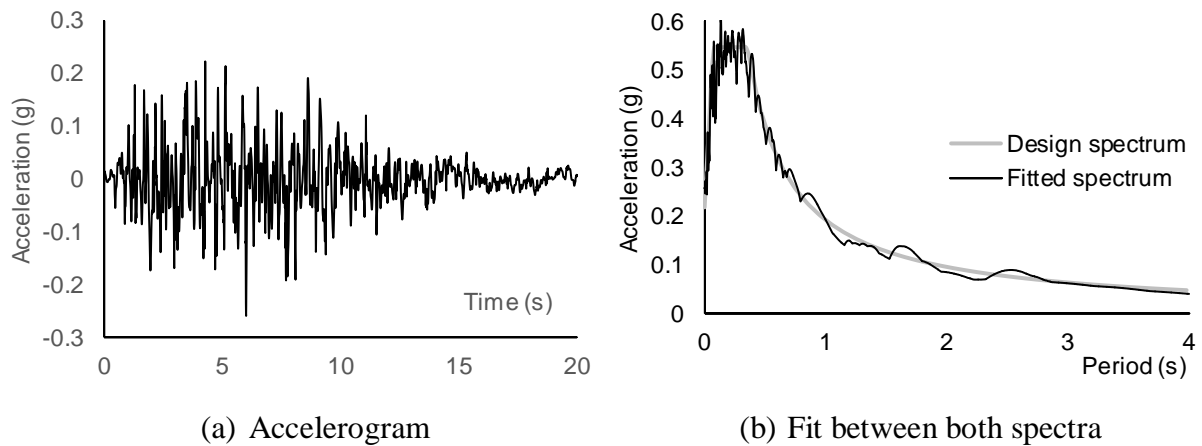


Figure 3-5. Accelerogram selected to fit a design spectrum for the analysis of the hospital buildings

3.2.4 Design of Building and Isolation Layer According to ASCE 7-10

Building and isolation system are designed with old USA code using Static Linear Analysis. Initially, it is estimated that dead load is 7 kN/m^2 for floors and 4 kN/m^2 for roof; live load is 4 kN/m^2 for surgery rooms and laboratories, 2 kN/m^2 for rooms and 5 kN/m^2 for stairs, corridors and other public areas. Soil has a shear wave velocity of 500 m/s , corresponding to type C. Parameters for Los Angeles/New Mexico seismicity are: $S_1 = 0.623/0.183$, $S_s = 1.55/0.625$, $F_a = 1/1.15$, $F_v = 1/0.621$, $T_0 = 0.08/0.082 \text{ s}$, $T_s = 0.402/0.412 \text{ s}$, $T_1 = 8/6 \text{ s}$. From this information, (ASCE 7, 2010) zero-period spectral ordinates in soil C ($S_a(0)$) are 0.4 g and 0.2 g for Los Angeles and New Mexico, respectively. Concrete strength is $f'_c = 21 \text{ MPa}$ and steel yielding point is $f_y = 420 \text{ MPa}$.

After some iterations, target values of fundamental period and first mode damping are selected: 2.69 s and 27% in Los Angeles and 2.53 s and 25% in New Mexico. Then building and isolation layer design is carried out as described previously. Seismic weight of superstructure for Los Angeles/New Mexico are $34952/32218 \text{ kN}$ ($D + 0.3 L$).

Isolation system consists of LRB and NRB for both buildings; in Los Angeles there are also viscous dampers. Figure 3-6 displays layout of these devices and the Table 3-7 describes major geometrical and mechanical parameters of rubber bearings. Figure 3-6 shows that LRB and dampers are located far from center of rigidity, to provide torsion damping and stiffness.

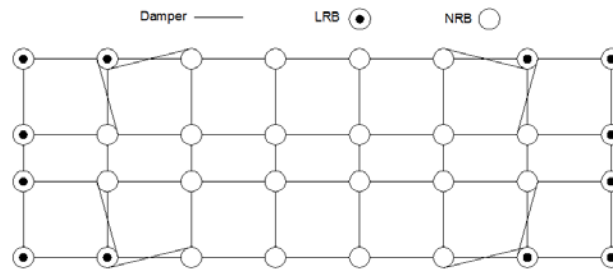


Figure 3-6. Distribution of isolators and dampers (in the Los Angeles hospital building)

Table 3-7. Characteristics of the isolators (minimum/nominal/maximum) for the hospital buildings

Property	Los Angeles		New Mexico	
	NRB	LRB	NRB	LRB
Diameter (mm)	500	600	450	450
Rubber layer height (mm)	5	6	5	5
Rubber height (mm)	125	250	110	180
Lead core diameter (mm)	-	90	-	60
Rubber shear modulus (MPa)	0.392	0.385	0.392	0.385
Lateral initial stiffness (kN/m)	616/677/800	5867/5867/5867	567/623/737	5585/5585/5585
Yielding force (kN)	-	54.91/60.40/71.38	-	26.70/29.30/34.71
Lateral after-yield stiffness (kN/m)	-	451/496/587	-	429.6/472.6/558.5
Vertical stiffness (kN/mm)	1228	1106	1043	811
Effective design stiffness (kN/m)	515	715	567	738
Effective damping (%)	-	20.38	-	25

All dampers are alike. Main parameters are: $\alpha = 0.4$, $c = 135.4 \text{ kN}/(\text{mm}/\text{s})^{0.4}$, $K_{oil} = 7144 \text{ kN}/\text{m}$, maximum stroke $\pm 30 \text{ cm}$, maximum speed $0.569 \text{ m}/\text{s}$, and maximum force 109 kN . Table 3-8 displays periods and modal mass ratios of first six modes of base-isolated buildings and of first three modes of buildings under fixed-base conditions. Since isolation layer adds three new modes, in Table 3-8 first three modes of fixed-base buildings are associated with 4th, 5th and 6th modes of base-isolated buildings, respectively. In isolated buildings, periods are calculated for effective secant stiffness (of lead-rubber isolators) that correspond to 100% and 50% shear strain for Los Angeles and New Mexico, respectively. Table 3-8 shows that, for both isolated buildings, first three modes correspond to motion along x , y and rotational directions, respectively; this indicates a high symmetry of the isolation system. Also, stiffness is highly similar in both directions. Comparison among periods of first three modes of base-isolated and fixed-base buildings, shows that base isolation elongates periods as expected.

Table 3-8. Modal parameters of the hospital buildings under isolated/fixed-base conditions

Mode	Los Angeles				New Mexico			
	Period (s)	Modal mass ratio (x)	Modal mass ratio (y)	Rotational mass ratio	Period (s)	Modal mass ratio (x)	Modal mass ratio (y)	Rotational mass ratio
1/-	2.690/-	0.993/-	0/-	0/-	2.530/-	0.998/-	0/-	0/-
2/-	2.670/-	0/-	0.999/-	0/-	2.510/-	0/-	0.999/-	0/-
3/-	2.400/-	0/-	0/-	0.990/-	2.290/-	0/-	0/-	0.999/-
4/1	0.300/0.513	0/0.820	0/0	0/0	0.350/0.706	0/0.836	0/0	0/0
5/2	0.288/0.470	0/0	0/0.820	0/0	0.330/0.697	0/0	0/0.830	0/0
6/3	0.265/0.449	0/0	0/0	0/0.820	0.310/0.617	0/0	0/0	0/0.833

3.2.5 Analyses with the Compared Codes

This subsection investigates seismic performance of buildings conducting static linear analyses according to all codes (obviously, except old USA one), and nonlinear time-history analyses using artificial accelerograms. Verification consists in comparing values of F_{sup} , F_{sub} , F_{Δ} and D_{TM} . Outside USA, locations corresponding to $S_a(0) = 0.4$ g and 0.2 g are selected. Some calculations require obtaining seismic accelerations for return periods different from reference one; as in generation of accelerograms, this operation is done through modification factor $(T/T_R)^{0.3}$.

Table 3-9 displays, for each analyzed code and level of seismicity, reduction factors due to damping, and spectral ordinates for 5% damping, 475 years return period and corresponding target fundamental period. Results from Table 3-9 show that maximum and minimum reduction for damping correspond to Japan and USA, respectively and maximum and minimum spectral ordinates correspond to Russia and Italy, respectively.

Table 3-10 displays total displacements of isolators (D_T), design forces for superstructure (F_{sup}), design forces for substructure (F_{sub}), and forces for obtaining drift limit (F_{Δ}). For each code and seismicity level, values from equivalent lateral force method and from dynamic calculations are presented.

Table 3-9 Design parameters for static linear analysis of the example hospital buildings

Country	Reduction factor due to damping		Spectral ordinate (5% damping, 475 years return period)	
	Damping 27% for high seismicity ($S_a(0) = 0.4 \text{ g}$)	Damping 25% for medium seismicity ($S_a(0) = 0.2 \text{ g}$)	High seismicity ($S_a(0) = 0.4 \text{ g}$)	Medium seismicity ($S_a(0) = 0.2 \text{ g}$)
Japan	0.405	0.429	0.2110	0.1121
China	$\gamma = 0.785, \eta_1 = 0.00259, \eta_2 = 0.570$	$\gamma = 0.789, \eta_1 = 0.0033, \eta_2 = 0.583$	0.1921	0.0974
Russia	0.559	0.577	0.3860	0.1950
Italy	0.559	0.577	0.1274	0.0720
USA (ASCE 7-10)	0.577	0.597	0.1529	0.0813
USA (ASCE 7-16)	0.577	0.597	0.1318	0.0696
Chile	0.444	0.461	0.1628	0.0920

Table 3-10. Design parameters for static/time-history analysis of the analyzed buildings

Country	D_T (mm)		F_{sup} (kN)		F_{sub} (kN)		F_{Δ} (kN)	
	$S_a(0) = 0.4 \text{ g}$	$S_a(0) = 0.2 \text{ g}$	$S_a(0) = 0.4 \text{ g}$	$S_a(0) = 0.2 \text{ g}$	$S_a(0) = 0.4 \text{ g}$	$S_a(0) = 0.2 \text{ g}$	$S_a(0) = 0.4 \text{ g}$	$S_a(0) = 0.2 \text{ g}$
Japan	269/264	141/141	7201/7092	3479/3423	7201/7092	3479/3423	3910/3870	1889/1876
China	604/387	360/237	10898/8528	5872/3790	10898/8528	5872/3790	10898/8528	5872/3790
Russia (*)	596/563	292/299	14266/13456	6927/6910	14265/13456	6927/6910	14266/13456	6927/6910
Italy (hospital)	277/244	158/138	3920/3461	2030/1780	6001/5356	3046/2713	3125/2801	1626/1485
Italy (housing)	216/193	129/115	3185/2955	1649/1524	4778/4299	2474/2209	2432/2230	1259/1167
USA (ASCE 7-10)	395/319	213/180	3939/3392	1909/1671	5908/4682	2863/2266	5908/4681	2863/2266
USA (ASCE 7-16)	346/284	182/158	5636/4838	2781/2321	8586/7242	4171/3435	8586/7242	4171/3435
Chile	267/254	140/137	3354/3197	1256/1219	4472/4213	1674/1584	3354/3197	1256/1219

(*) In Russia, results for static analysis correspond to modal spectral analysis

Figures in Table 3-10 represent the design parameters of isolated buildings corresponding to equivalent level of seismicity. Since Table 3-10 summarizes the most relevant results of this work, comprehensive interpretations are necessary. Major comparisons are discussed next:

- **Static vs. dynamic results.** Comparison among results from static and dynamic analyses shows that only in one case (D_T , Russia, $S_a(0) = 0.2 \text{ g}$) there is a slight increase, in the other cases, results from dynamic analyses are lower; as expected, given that dynamic analyses are less

simplified. Minimum and maximum reductions for D_T are 1.62/0.07% (Japan 0.4 g/0.2 g) and 35.85/33.99% (China 0.4 g/0.2 g). Regarding F_{sup} and F_{sub} , minimum and maximum decreases are 1.51/1.63% (Japan 0.4 g/0.2 g) and 21.75/35.46% (China 0.4 g/0.2 g). Concerning F_{Δ} , these quantities are 1.02/0.72% (Japan 0.4 g/0.2 g) and 21.75/35.46% (China 0.4 g/0.2 g). These comparisons show that in the Japanese and Chilean codes, static and dynamic formulations are highly adjusted; as regards Chinese code, nonlinear time-history analyses are widely used (Gao et al., 2013b, 2013a). It should be kept in mind that Japanese and Chinese regulations allow considering nonlinear behavior of superstructure; conversely, accelerograms have been generated without reducing the design spectra. Therefore, higher reductions should be expected in actual applications. In USA regulations, reductions are significant, ranging between 13.34 and 19.24 for D_T , 12.47 and 16.25 for F_{sup} and F_{sub} , and 15.66 and 20.87 for F_{Δ} . By performing nonlinear time-history analyses, American and Chilean regulations allow maximum reductions in regular buildings of F_{sup} , D_T , and F_{sub} of 40, 20 and 10%, respectively. Table 3-10 shows that these limitations are only exceeded for F_{sub} in USA codes.

- **High vs. medium seismicity.** Given that differences between static and dynamic results are already discussed, this paragraph analyzes only decreases from high to medium seismicity in static linear analysis. Minimum and maximum reductions for D_T are 40.28% (Italy for housing) and 51.01% (Russia). Regarding F_{sup} , F_{sub} , and F_{Δ} , minimum and maximum diminutions are 46.12% (China) and 62.56% (Chile). These comparisons show that, as expected, lessening percentage is close to 50%; variation among analyzed regulations is rather low.
- **Comparison among countries.** Given that differences between static and dynamic results and between high ($S_a(0) = 0.4$ g) and medium seismicity ($S_a(0) = 0.2$ g) are discussed in previous paragraphs, only figures for static analyses and high seismicity are compared herein. At a first glimpse, it is apparent that prescription of compared codes are highly uneven. Minimum and maximum values for D_T are 216 mm (Italy for housing use) and 604 mm (China). Regarding F_{sup} , minimum and maximum values are 3185 kN (Italy for housing use) and 14266 kN (Russia). Regarding F_{sub} , minimum and maximum values are 4472 kN (Chile) and 14265 kN (Russia). Concerning F_{Δ} , these quantities are 2432 kN (Italy for housing use) and 14266 kN (Russia). These comparisons show that in Italian code, differences between housing and sanitary use are significant, both for design forces and drift limits. Variations in new USA code referring to old one are -9.87% for D_T , $+43.08\%$ for F_{sup} and $+45.33\%$ for F_{sub} and F_{Δ} . By dividing F_{Δ} by drift limits, required stiffness is obtained: 65.16 kN/m (Japan), 45.41 kN/m (China), 336.40 kN/m (Russia), 78.13 kN/m (Italy for hospital use), 60.79 kN/m (Italy for housing use), 32.82 kN/m

(USA ASCE 7/10), 47.70 kN/m (USA ASCE 7-16), and 139.76 kN/m (Chile). There are extremely important discrepancies.

Table 3-11 displays D_T , F_{sup} , and F_{sub} , from static linear analysis and corresponding to levels of seismicity uniformed in terms of return period. Values for Russia are omitted, given that are outermost; also, case “Italy for housing use” is not included because is distinguished from “Italy for sanitary use” through return period.

Table 3-11. Design parameters for static analysis of the analyzed buildings under uniform return period demand for medium seismicity ($S_a(0) = 0.2 g$)

Country	D_T (mm) $T = 2475$ years	F_{sup} (kN) $T = 475$ years	F_{sub} (kN) $T = 475$ years
Japan	155	2476	2476
China	253	2517	2517
Italy	191	1312	1968
USA (ASCE 7-10)	213	1909	2863
USA (ASCE 7-16)	182	1508	2542
Chile	164	1105	1474

Figures from Table 3-11 shows significantly less scattering than Table 3-10. This circumstance indicates that part of huge disparities in Table 3-10 are due to different return period. With only two exceptions, codes for minimum and maximum values are coincident in Table 3-10 and Table 3-11. Results for both USA codes show that, once quantities are normalized with respect to return period, new code can be considered less demanding (without consider properties variation).

4. GENERATING DAMPING MODIFICATION FACTORS AFTER ARTIFICIAL INPUTS IN SCENARIOS OF LOCAL RECORDS SCARCITY

A considerable number of seismic regulations merely provide design spectra corresponding to 5% damping. This damping level can be adequate for most of the highly-damaged ordinary buildings and for some bridges, but the behavior of numerous other constructions is better described with significantly different damping ratios. Relevant examples of lower damping are: modern high-tech high-rise steel buildings, towers, masts, chimneys, racking systems, most of bridges (mainly large steel ones), steel warehouses, steel tanks, silos, arch and gravity dams, nuclear power plants, industrial facilities and buildings, pipelines, underground structures in rock or stiff soil, higher vibration modes of buildings with base isolation, and virtually all the undamaged constructions (i.e. for damage limitation state in performance-based seismic engineering), among other cases. Regarding higher damping, there is also a considerable number of situations: old masonry buildings, timber constructions, most of historic buildings, short bridges founded on soft soil, embankments, earth dams, underground structures in soft soil, buildings or bridges with base isolation, and buildings equipped with additional dampers, among other cases. Also, the substitute structure concept (Gulkan and Sozen, 1974; Shibata and Sozen, 1976), the Direct Displacement-based Design approach (Priestley, 2003) and the Capacity Spectrum Method (ATC-40, 1996), require the consideration of high damping over long period ranges. Among these circumstances, considerable variations of the damping ratio are observed, ranging approximately between 0.5% and 30%, or higher. Using 5% damping spectra for constructions with different damping ratios can lead to important errors. If the damping ratio is less than 5%, the error lies on the unsafe side. In the opposite situation, the error is on the safe side; nonetheless, in base isolated constructions and in building with energy dissipators it cannot be ignored, given that the higher damping is an essential part of the design.

To sum up, it is strongly necessary to have seismic design spectra corresponding to a wide range of damping ratios.

Numerous seismic regulations provide criteria to modify the 5% damping spectral ordinates to match other damping levels; such criteria are expressed in terms of damping modification factors. Some codes (BSL, 2009; EN-1998-2, 2004; GB 50011, 2010; NCh 2745, 2013) propose expressions or tables for modification factors to be considered for other damping ratios. The American regulations (ASCE 7-10, 2010; ASCE 7-16, 2016; FEMA P 1050-1, 2016) provide modification factors that are intended only for buildings with base isolation or additional dampers; therefore, such coefficients can be solely employed for damping ratios higher than 5%. These correction coefficients have been derived (Ramirez et al., 2001, 2002) after seismic inputs recorded in the USA; the study (Saez et al., 2012) for Chile (NCh 2745, 2013) has shown significant discrepancies with the American regulations (Piscal, 2018; Piscal and López, 2016). Although these discrepancies can be partly explained by the varying assumptions, they rely mainly on the particularities of each country seismicity. These considerations show that the criteria for modifying the 5% design spectrum derived for a specific country cannot be extrapolated to other areas. Consequently, both this paragraph and the previous one highlight that there is a strong need for developing damping modification factors in countries where they are not currently available.

The damping modification factors are commonly derived after strong historical seismic records that characterize the local seismicity. However, in numerous occasions this approach is unfeasible, given that the number of such records that are readily available is insufficient. This scarcity can be caused by moderate (or medium) and little known seismicity and by limitations of the seismological network. This last issue refers to poor or too recent networks. In such cases, representative artificial accelerograms might be used instead. This paper proposes a methodology for establishing damping modification factors, for countries, regions or cities, after artificial seismic inputs.

The proposed methodology starts by generating artificial accelerograms that are fitted to the 5% damped code design spectra. Then, dynamic analyses on SDOF linear systems using such inputs are carried out. Finally, for a given damping ratio, the damping modification factor is defined as the ratio between the individual displacement spectral ordinates corresponding to such damping ratio and to 5%. For the sake of further reliability, the obtained factors are compared with those derived after the available local historical accelerograms. Perhaps the main objection that can be made to this approach is that previous researches (Bommer and

Mendis, 2005; Hatzigeorgiou, 2010; Stafford et al., 2008) have underlined the differences among factors generated after natural and artificial inputs. However, such discrepancies are mainly due to the usual longest significant (Trifunac (Trifunac and Brady, 1975)) duration of the artificial accelerograms. Therefore, in the proposed methodology, the artificial inputs are generated caring that their duration approximates those of the available local records. In any case, even if the consideration of artificial inputs may introduce some errors, they will be significantly smaller than using damping modification factors derived for other seismic conditions.

An example of application of the proposed strategy to Colombia is presented. In such application, the country is divided in ten seismic zones according to the current design code (NSR-10, 2010), and the five most common soil types (A, B, C, D and E) are considered. Given the rather moderate seismicity of Colombia and the limitations and recentness of the seismological network, the available natural severe inputs are scarce. On the other hand, there is not enough information for selecting international records representing the Colombian hazard, such as moment magnitude and hypocentral distance. As well, it is not possible to find records that can be scaled to the design spectra for the full range of periods.

Therefore, for each zone and soil type, groups of seven artificial accelerograms fitting the design spectra for 5% damping are generated. The results obtained with these artificial inputs are compared with those for some available historical accelerograms recorded in Colombia. The sensitivity of the calculated modification factors to the soil type, period and seismic zone is investigated, and matching expressions are generated; such equations are intended to be incorporated to the Colombian regulations. These expressions are compared with previous researches and with the prescriptions of major worldwide design codes; reasonable fit is observed. Finally, a verification example on a hospital building with seismic isolation and located in Cali (Colombia) is presented and discussed. This example further endorses the proposed approach, since their results are satisfactorily compared with those using the historical records that were employed in the seismic microzonation of Cali.

4.1 Damping modification factors

4.1.1 Concept of design spectrum

The design spectra, although routinely considered for structures described with MDOF models, are initially intended for linear SDOF systems. They are plots containing, in their horizontal axes, the natural period of the system under consideration. The vertical axes represent magnitudes relevant to design; mainly, displacement, energy (in terms of equivalent velocity) and acceleration. For a given location, the code design spectra are smoothed envelopes of the individual spectra that corresponding to a number of accelerograms that represent the site seismicity.

The linear equation of motion of a SDOF system under seismic excitation can be written in any of these forms:

$$m \ddot{x} + c \dot{x} + k x = -m \ddot{x}_g \qquad m \ddot{y} + c \dot{x} + k x = 0 \qquad (4-1)$$

$$\ddot{x} + 2 \zeta \omega \dot{x} + \omega^2 x = -\ddot{x}_g \qquad \ddot{y} + 2 \zeta \omega \dot{x} + \omega^2 x = 0 \qquad (4-2)$$

In equation (4-1), m , c and k are mass, damping and stiffness coefficients, x_g is the input soil displacement, and x and y are the response relative and absolute displacements respectively; obviously, x , y and x_g are linked by the kinematic relation $x = y - x_g$. In equation (4-2), ξ is the damping ratio ($\zeta = \frac{c}{2\sqrt{km}}$) and ω is the undamped natural frequency ($\omega = \sqrt{\frac{k}{m}}$). Although the SDOF systems are actually characterized by three parameters (m , c and k), equation (4-2) shows that the response depends only on ξ and ω ; ordinarily, ω is replaced by the natural period T ($T = \frac{2\pi}{\omega} = 2\pi\sqrt{\frac{m}{k}}$).

The solution of equation (4-1) in terms of relative displacement, relative velocity, and absolute acceleration can be obtained through the Duhamel integral as

$$x = -\frac{1}{\omega_d} \int_0^t \ddot{x}_g(\tau) \sin \omega_d(t-\tau) e^{-\zeta \omega(t-\tau)} d\tau \qquad (4-3)$$

$$\dot{x} = -\int_0^t \dot{x}_g(\tau) \cos \omega_d(t-\tau) e^{-\zeta \omega(t-\tau)} d\tau + \frac{\zeta}{(1-\zeta^2)^{1/2}} \int_0^t \ddot{x}_g(\tau) \sin \omega_d(t-\tau) e^{-\zeta \omega(t-\tau)} d\tau \qquad (4-4)$$

$$\begin{aligned} \ddot{y} &= -2 \zeta \omega \dot{x} - \omega^2 x = \\ &= 2 \zeta \omega \int_0^t \ddot{x}_g(\tau) \cos \omega_d(t-\tau) e^{-\zeta \omega(t-\tau)} d\tau + \frac{1-2\zeta^2}{(1-\zeta^2)^{1/2}} \omega \int_0^t \ddot{x}_g \sin \omega_d(t-\tau) e^{-\zeta \omega(t-\tau)} d\tau \end{aligned} \qquad (4-5)$$

In equations (4-3) through (4-5), ω_d is the damped natural frequency given by $\omega_d = \omega(1-\zeta^2)^{1/2}$.

The following three response spectra are initially defined:

$$S_d(\zeta, T) = |x(\zeta, T)|_{\max} \quad S_v(\zeta, T) = |\dot{x}(\zeta, T)|_{\max} \quad S_a(\zeta, T) = |\ddot{y}(\zeta, T)|_{\max} \quad (4-6)$$

In equation (4-6), S_d , S_v and S_a , are termed relative displacement, relative velocity, and absolute acceleration response spectra, respectively.

By neglecting the difference between the two integrals with sinus and cosinus, when ζ is small, the maximum absolute value of the relative velocity in equation (4-4) can be approximated as $|\dot{x}|_{\max} \approx \omega |x|_{\max}$. Analogously, under the same assumptions, equation (4-5) provides that $|\ddot{y}|_{\max} \approx \omega^2 |x|_{\max}$. These considerations have inspired the proposal of pseudo-velocity (PS_v) and pseudo-acceleration (PS_a) response spectra:

$$PS_v(\zeta, T) = \omega S_d(\zeta, T) \quad PS_a(\zeta, T) = \omega^2 S_d(\zeta, T) \quad (4-7)$$

In ordinary force-based code-type design, the most meaningful spectrum is the pseudo-acceleration one (PS_a), since it provides the (fictitious) equivalent static forces that generate the same maximum relative displacement than the actual accelerogram does; in other words, this spectrum reports on structural damage. For that reason, the design codes contain a smoothed envelope of the corresponding individual spectra; this envelope is commonly termed as “response acceleration design spectrum”, despite being actually a pseudo-acceleration spectrum. In displacement-based design (Priestley et al., 2007), obviously, displacement spectra (S_d) are considered. Noticeably, equation (4-7) shows that both formulations utilize the same seismic information.

The acceleration spectrum (S_a) is also meaningful, since represents the maximum absolute acceleration, thus reporting on the non-structural damage.

It should be emphasized that these considerations on the utility of PS_a , S_d and S_a , hold even releasing the assumption that ζ is small. Indeed, if the damping ratio is high (e.g. above 20%), the only relevant consequence is that PS_a and S_a might differ significantly.

For multi-story base isolated buildings, (Kelly, 1999) shows that excessive additional damping does reduce displacement at the isolation layer, but at the expense of increasing floor accelerations and interstory drifts in the superstructure. This consideration further emphasizes

the need of considering not only S_d , but also S_a . Noticeably, this circumstance cannot be completely derived after the spectra, since they merely correspond to SDOF systems.

4.1.2 State-of-the-art of research on damping modification factors

A considerable number of studies on the derivation of damping modification factors have been published; only the researches that have most influenced this work are reported herein (Atkinson and Pierre, 2004; Benahmed et al., 2016; Bommer and Mendis, 2005; Bradley, 2015; Cameron and Green, 2007; Cardone et al., 2009; Cassarotti et al., 2009; Hao et al., 2011; Hatzigeorgiou, 2010; Lin and Chang, 2003, 2004; Lin et al., 2005; Mavroeidis, 2015; Mollaioli et al., 2014; Palermo et al., 2016; Pu et al., 2016; Rezaeian et al., 2014; Sheikh et al., 2013; Stafford et al., 2008). These works examine the influence of a number of issues: fundamental period of the construction, input duration, distance from the site to the hypocenter, magnitude of the earthquake, soil type, forward-directivity effect (near-fault), among others. The following general remarks spring from these studies:

- **Period.** For zero period, the stiffness is infinite; therefore, the relative displacement is null, and the absolute acceleration equals the one of the ground. Since both results are independent on damping, it is obvious that the damping modification factors for S_d and S_a , are equal to 1 for $T = 0$. At the other end of the spectrum, if T approaches infinite while the damping ratio is maintained constant, both stiffness and damping coefficients approach zero; therefore, the system becomes uncoupled from the ground, and the relative displacement equals minus the one of the ground, and the absolute acceleration is null. Given that both properties hold regardless of damping, the damping modification factors for S_d and S_a , tend to 1 when T tends to infinity. For nonzero short periods, the damping modification factors become more extreme (that is, greater/smaller for damping ratios smaller/greater than 5%) until stabilizing for periods ranging approximately between 0.25 s and 1 s. Then, for longer periods, the damping effect decreases slowly but consistently; as discussed previously, the damping modification factors approach one as the period tends to infinity.
- **Input duration.** Differences among response spectra for different damping levels should be greatest for long-lasting excitations, since have more time to develop stationary response, and thus fully undergo the damping effect. In other words, the longer the input, the more

complete the damping effect. Therefore, damping modification factors should be significantly more extreme for prolonged accelerograms.

- **Hypocentral distance.** For a given earthquake, apart from local effects, the duration of the inputs grows with the distance between the location and the hypocenter. A relevant consequence is that the damping effect increases consistently as that distance grows; hence the damping modification factor tends to be more extreme.
- **Earthquake magnitude.** Given that events with higher moment magnitude tend to generate records lasting longer, the damping effect is expected to increase accordingly. This trend is well-established only for periods greater than approximately 0.5 s; for shorter periods this tendency can be inverted.
- **Soil type.** Regarding the influence of soil type, two opposite trends collide: in rock and stiff soil the soil damping is rather low and, therefore, the role of the structural damping is more relevant, but the inputs tend to be shorter. In softer soils, the opposite happens. As a result, the balance is unclear.
- **Near-fault effects.** For pulse-like records, the damping modification factor is, in general, slightly closer to 1 (less extreme) than for ordinary inputs. This circumstance can be explained by the short duration of such inputs and, more specifically, by the even shorter duration of the pulses, being their most destructive part. However, this trend can be inverted for periods close to the pulse period, given that the paramount importance of damping in the response peak.

These remarks highlight the need of considering the influence of period, input duration, hypocentral distance, earthquake magnitude, soil type and near-fault effects in the derivation of damping modification factors for any country or region.

4.1.3 Existing methodologies for determining damping modification factors for a given country

All the proposed methodologies are based on calculating, through time-history analyses, the effects of damping on the maximum displacement response of SDOF systems subjected to recorded accelerograms or, less frequently, to artificial ones. Then the damping modification factor is defined as the ratio between the spectral ordinates (either PS_a or S_d) for the considered value of damping and for 5% damping ($S_d(\zeta, T) / S_d(0.05, T)$). A number of papers (Cardone et

al., 2009; Cassarotti et al., 2009; Lin, 2007; Sheikh et al., 2013) describe comprehensively the state-of-the-art, including the earliest studies; in view of that, this subsection discusses only the latest researches.

- (Saez et al., 2012). Among other contributions, this study proposes, for Chile, an empirical expression of damping reduction factor for different spectral ordinates. This equation is similar to the one proposed in (Lin and Chang, 2004), depending on period and damping ratio. This expression has been derived after linear dynamic analyses on SDOF systems under 28 scaled Chilean seismic records; these inputs are grouped into hard, intermediate and soft soil. The used scaling procedure is described in (Kottke and Rathje, 2008). The influence of the earthquake type (inter-plate and intra-plate), soil type and duration of motion is discussed. The obtained factor has been proposed for implementation in the Chilean design code (NCh 2745, 2013b).
- (Anbazhagan et al., 2016). This work proposes, for the Himalayan Region, damping reduction factors for pseudo-acceleration spectra obtained after inputs that have been recorded in the considered region. The main output of the study is an empirical equation providing the damping reduction factor in terms of period, moment magnitude, hypocentral distance, and site classification. The authors state that, although the influence of the input duration is significant, it is indirectly represented by the other considered parameters.
- (Mendo and Fernandez, 2017). Among other contributions, this study proposes a slight modification for Peru of the empirical expression of damping reduction factor originally proposed for Chile (NCh 2745, 2013b; Saez et al., 2012) . This expression depends on period and damping ratio, and has been derived after linear dynamic analyses on SDOF systems under 14 two-component Peruvian seismic records; these inputs are grouped into rock and intermediate soil. The obtained factors are in between those of (ASCE 7-10, 2010) and (NCh 2745, 2013b).

4.2 Proposed methodology for determining damping modification factors

The methodology proposed in this work is based on the strategy described in (Saez et al., 2012); it is intended for obtaining damping modification factors for buildings with energy dissipation devices located in Chile. The main modifications to this approach consist in extending it to any construction with damping ratios differing from 5%, and in designing a methodology for

dealing with the lack of sufficient seismic information, mainly in terms of available recorded accelerograms.

This work considers two modification factors (termed as B_a and B_d) intended to multiply the corresponding 5% damping design spectrum; B_a and B_d are generated from acceleration and displacement (or pseudo-acceleration) response spectra, respectively:

$$B_a(\zeta, T) = \frac{S_a(\zeta, T)}{S_a(0.05, T)} \quad B_d(\zeta, T) = \frac{S_d(\zeta, T)}{S_d(0.05, T)} = \frac{PS_a(\zeta, T)}{PS_a(0.05, T)} \quad (4-8)$$

After the discussion in subsection 4.1.1, it follows that B_a factor is meant to be used for S_a spectra, thus reporting on non-structural damage. Noticeably, such spectra are not readily available in the design codes; moreover, equation (4-5) shows that, for high damping ratios and short periods, differences between S_a and PS_a can be considerable. Regarding B_d factor, is meant for both S_d and PS_a spectra, thus reporting on structural damage. Concerning B_v factor (derived after velocity spectra), it is not considered highly meaningful, since S_v does not represent adequately neither input nor hysteretic energy spectra in terms of equivalent velocity (Benavent-Climent et al., 2002, 2010; López-Almansa et al., 2013).

The most recent studies for determining damping modification factors for a given country or region have been carried out for Taiwan (Lin, 2007), Chile (Saez et al., 2012), the Himalayan region (Anbazhagan et al., 2016) and Perú (Mendo and Fernandez, 2017). Some of these investigations have been conducted after scaled accelerograms obtained from locally-recorded actual ground motions. Although this strategy is considered basically correct, it requires a wide set of registers that represent the actual seismicity; if locally-recorded accelerograms are to be used, it is limited to zones with high seismicity (to ensure a sufficient number of strong ground motions), deeply-studied tectonic mechanisms, and holding a dense, long-standing and reliable seismological network. In developing countries or in areas with moderate or medium seismicity, these conditions are not commonly fulfilled; this work investigates whether the lack of available historical accelerograms can be partly compensated with artificial inputs that are generated to fit the code design spectra. The proposed approach consists in deriving the damping modification factors after a combination of natural and artificial accelerograms. If the artificial inputs are generated for all the design spectra that correspond to each soil type and seismic zone that are specified in the code, they will easily become more numerous and representative than the available historic accelerograms; therefore, the proposed strategy

consists in deriving initially the damping modification factors after the artificial accelerograms and then comparing with the recorded ones for further verification and refining.

This strategy releases, to some extent, the need of taking into consideration the magnitude of the earthquake and the hypocentral distance, since these issues are implicitly incorporated in the design spectra for each seismic zone the country is divided in. Conversely, research (Stafford et al., 2008) has pointed out that the differences between the numbers of cycles of recorded and artificial accelerograms might lead to significant discrepancies among the damping modification factors derived after them; therefore, the generation of duration-compatible artificial inputs needs to be seriously taken this consideration.

Noticeably, the proposed strategy is considered particularly suitable for determination of damping modification factors for relatively small and highly populated urban areas where microzonation studies have been carried out, given that typically only few records are readily available.

4.3 Study for Colombia

4.3.1 General description

Given the aforementioned scarcity of seismic records in Colombia, the study is mainly based on artificial inputs. Initially, fifty groups of seven artificial accelerograms are generated; each group corresponds to a given seismic zone in Colombia (ten zones, (NSR-10, 2010)) and a given soil type (five types, A through E, (NSR-10, 2010)). The inputs are created to match the 5% damping design response acceleration spectra for the corresponding seismic zone and soil type. For each accelerogram, linear dynamic analyses are conducted on SDOF systems; their natural period (T) and damping ratio (ζ) range between 0.01 and 4 s, and 0.005 (0.5%) and 0.5 (50%), respectively. In each case (for a given seismic zone, soil type, and damping ratio) the seven obtained pairs of individual S_d and S_a spectra are averaged. Then, the modification factors B_d and B_a are defined as indicated in equation (4-8). The obtained results are compared with the most relevant available accelerograms recorded in Colombia.

Nineteen values of damping ratio ranging between 0.005 and 0.5 are considered: 0.005, 0.01, 0.015, 0.02, 0.025, 0.03, 0.035, 0.04, 0.045, 0.05, 0.1, 0.15, 0.20, 0.25, 0.3, 0.35, 0.4, 0.45 and

0.5. Regarding the period, 3990 values are taken, ranging from 0.01 and 4 s. Therefore, for the artificial inputs, the number of conducted dynamic analyses is: 10 (zones) \times 5 (soil types) \times 7 (accelerograms) \times 19 (damping ratios) \times 3990 (natural periods) = 26,533,500. As well, additional analyses are performed to provide sounder basis for the derived conclusions; among them, calculations for damping ratios in the range between 0.5 and 0.8.

In each dynamic analysis, the solution of equation (4-1) is obtained as indicated in equations (4-3) through (4-5); the involved Duhamel integrals are numerically solved by assuming linear interpolation of acceleration. For systems with natural period longer than 0.04 s, the time step is 0.01 s; for systems with shorter periods, the time step is selected as $T / 4$.

4.3.2 *Design spectra for Colombia*

As discussed in the previous subsection, the Colombian code (NSR-10, 2010) divides the country in ten seismic zones, being numbered as 1 (lowest seismicity) through 10 (highest seismicity). Regarding the soil classification, it follows basically the American regulation (ASCE 7-10, 2010); six categories are considered, ranging from A (hard rock, average shear wave velocity higher or equal than 1500 m/s) through F. Given that soil type F requires routinely particular studies, and that no design spectra are provided in the code, only types A through E (average shear wave velocity lower than 180 m/s) are considered in this study. Fifty design spectra are generated, corresponding to the ten seismic zones and the five major soil types.

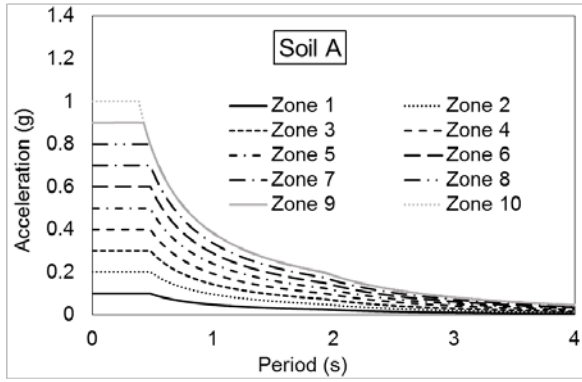
The seismic zones are classified with respect to the parameter A_a representing the design PGA (zero-period spectral ordinate) in soil type A (NSR-10, 2010). The site seismicity is also characterized by the parameter A_v affecting the medium and long period ranges of the spectrum; noticeably, the divisions of the country in seismic zones according to both parameters are not always coincident. Each zone is represented by a city having maximum values of A_a and A_v . Table 4-1 describes the main characteristics of the considered zones. Table 4-1 highlights the extreme discrepancies among the zones, ranging from low seismicity in zone 1 to high one in zone 10.

Table 4-1. Considered seismic zones

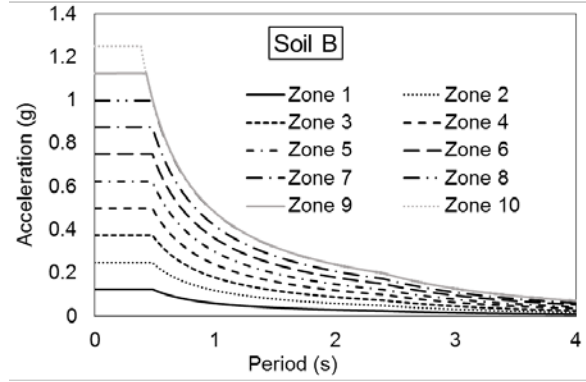
Zone	Representative city	A_a (g)	A_v (g)
1	Leticia	0.05	0.05
2	Valledupar	0.10	0.10
3	Arauca	0.15	0.15
4	Tunja	0.20	0.20
5	Manizales	0.25	0.25
6	El Carmen de Atrato	0.30	0.30
7	Quibdó	0.35	0.35
8	Alto Baudo	0.40	0.40
9	Tumaco	0.45	0.40
10	Olaya Herrera	0.50	0.40

The spectra are generated by assuming importance (I) and response modification (R) factors equal to one. The obtained spectral ordinates are intended to multiply the building seismic weight to provide the base shear force; such ordinates are termed S_a in the Colombian code, although they actually correspond to PS_a (pseudo-acceleration spectra). Each spectrum is composed of three branches: constant acceleration (short periods), constant velocity (intermediate periods) and constant displacement (long periods); the bounds among these ranges are termed T_C and T_L , respectively. Table 4-2 displays the values of T_C and T_L for each zone and soil type.

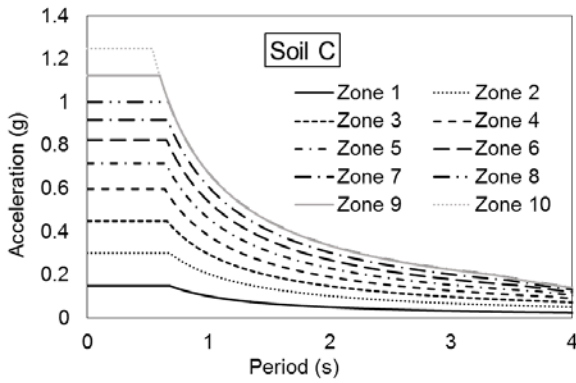
Figure 4-1 displays the pseudo-acceleration design spectra for each seismic zone and soil type. Figure 4-1.a through Figure 4-1.e contain sets of ten spectra (for each zone) corresponding to soils A through E, respectively. For the sake of further comparison, Figure 4-1.f presents, for zone 10, five spectra for each soil type. Figure 4-1.f shows that the softest soil (type E) does not always hold the highest spectral ordinates for the whole range of periods. Noticeably, the Colombian design spectra contain also an inclined initial branch; such segment is not included here, given that it is only intended for spectral analyses for other modes than the first one.



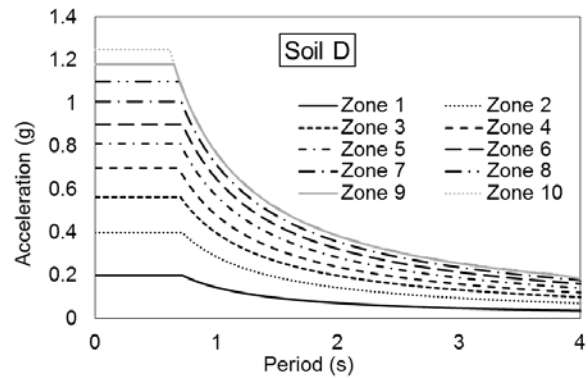
(a) Pseudo-acceleration spectra for soil type A



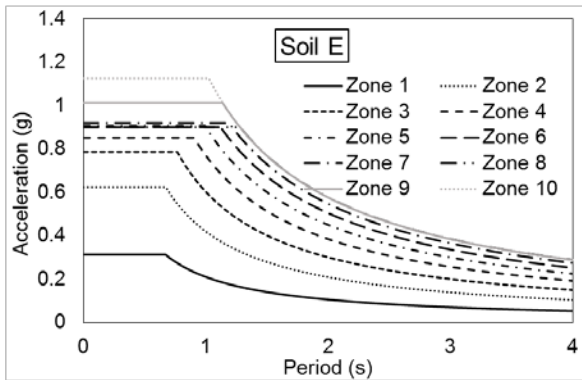
(b) Pseudo-acceleration spectra for soil type B



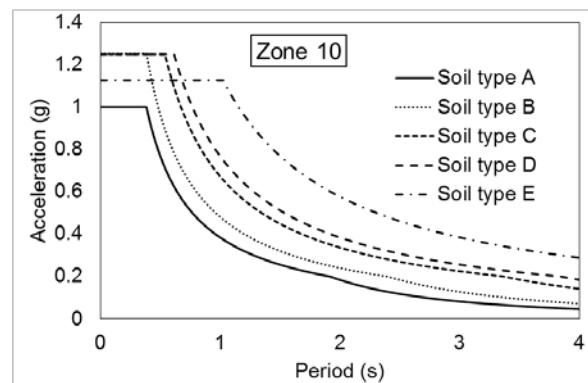
(c) Pseudo-acceleration spectra for soil type C



(d) Pseudo-acceleration spectra for soil type D



(e) Pseudo-acceleration spectra for soil type E



(f) Pseudo-acceleration spectra for zone 10

Figure 4-1 . Design spectra in the Colombian design code (NSR-10, 2010)

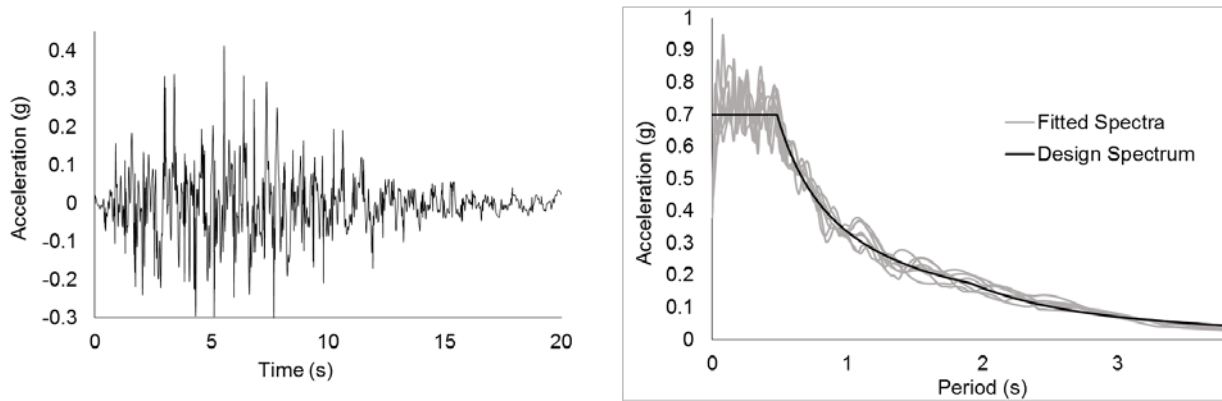
Table 4-2. T_C / T_L periods (s) in the Colombian code

Soil type	Zone									
	1	2	3	4	5	6	7	8	9	10
A	0.48/ 1.92	0.48/ 1.92	0.48/ 1.92	0.48/ 1.92	0.48/ 1.92	0.48/ 1.92	0.48/ 1.92	0.48/ 1.92	0.43/ 1.92	0.38/ 1.92
B	0.48/ 2.40	0.48/ 2.40	0.48/ 2.40	0.48/ 2.40	0.48/ 2.40	0.48/ 2.40	0.48/ 2.40	0.48/ 2.40	0.43/ 2.40	0.38/ 2.40
C	0.68/ 4.08	0.68/ 4.08	0.66/ 3.96	0.64/ 3.84	0.65/ 3.72	0.66/ 3.60	0.66/ 3.48	0.67/ 3.36	0.60/ 3.36	0.54/ 3.36
D	0.72/ 5.76	0.72/ 5.76	0.70/ 5.28	0.69/ 4.80	0.70/ 4.56	0.72/ 4.32	0.71/ 4.08	0.70/ 3.84	0.65/ 3.84	0.61/ 3.84
E	0.67/ 8.40	0.67/ 8.40	0.77/ 8.04	0.90/ 7.68	0.99/ 7.20	1.12/ 6.72	1.19/ 6.24	1.28/ 5.76	1.14/ 5.76	1.02/ 5.76

Plots from Figure 4-1.a through Figure 4-1.e show that, for each soil type, the spectra for the ten zones are approximately homothetic, namely vertically scaled with A_a coefficient (Table 4-1). Therefore, given that the artificial inputs are generated to fit these spectra, the inputs for the ten zones corresponding to a given soil type will be also approximately homothetic; therefore, since the damping modification factors are obtained after linear analyses, no extreme differences among the ten zones are to be expected.

4.3.3 Artificial seismic inputs

As discussed previously, fifty groups of seven artificial accelerograms each are created. These groups correspond to the ten seismic zones in Colombia and the five soil types that have been considered. The accelerograms are generated matching the 5% damping acceleration design response spectrum using the SeismoArtif software (Seismosoft, 2016). The inputs are generated for 20 s duration (NUREG-0800, 2014). The variation of amplitude vs. time responds to the function described in (Saragoni and Hart, 1973); the maximum amplitude corresponds to 4 s and the final instant amplitude is 5% of the maximum one. This choice is based on its superior capacity to reproduce the behavior of actual inputs (Saragoni and Hart, 1973) and accounts for the aforementioned high sensitivity of the damping modification factor to the input duration. The quadratic error and coefficient of variation averaged for the 350 accelerograms are 8.70% and 0.0997, respectively; the discretization period is 0.01 s. Figure 4-2.a displays an example of a sample accelerogram whose response spectrum fits the design spectrum of the Colombian code (Piscal, 2018). Figure 4-2.b presents comparison among such design spectrum and those of the seven corresponding artificial accelerograms (one of them is displayed in Figure 4-2.a).



(a) Generated sample artificial accelerogram

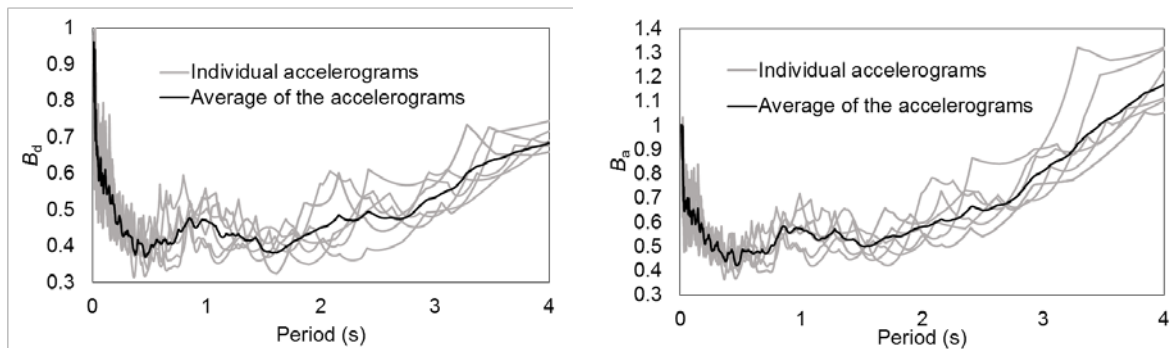
(b) Accelerogram individual spectra and acceleration design spectrum

Figure 4-2. Sample accelerogram selected to fit a design spectrum for zone 7 and soil type A

Figure 4-2.b highlights the great similarity between the target code design spectrum and the individual response spectrum of the generated artificial accelerograms.

4.3.4 Obtained results

Figure 4-3 displays sample spectra for factors B_d (Figure 4-3.a) and B_a (Figure 4-3.b); the selected case corresponds to Zone 7, soil type A and damping 30%. Each figure presents seven plots of B_d or B_a vs. period corresponding to seven individual accelerograms and their average spectrum; noticeably, only 6 individual spectra can be observed because two of them are almost coincident. Figure 4-3 shows that the dispersion of the seven spectra corresponding to the same design spectrum (i.e. same seismic zone and soil type) is rather moderate.

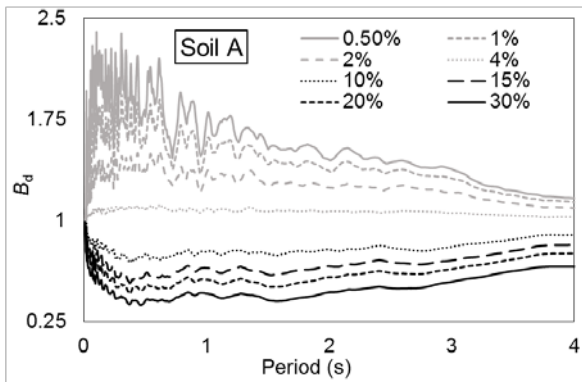


(a) B_d factor

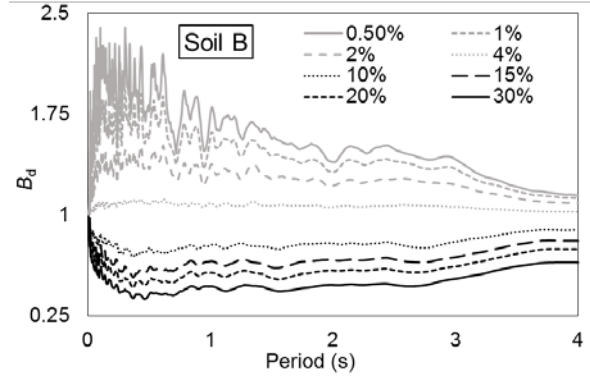
(b) B_a factor

Figure 4-3. Sample spectra for factor B_d and B_a for seven artificial accelerograms. Zone 7, soil A and damping 30%

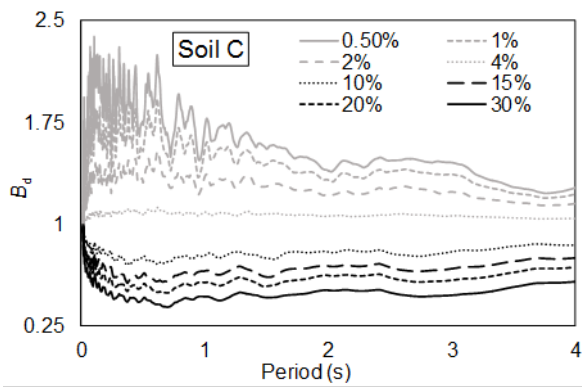
The observation of the results averaged for each group of seven accelerograms shows little influence of the seismic zone (subsection 4.3.2), this being coherent with the complexity of the tectonics of Colombia; therefore, the values of B_d are averaged for the ten zones. Figure 4-4 displays plots of factors B_d vs. period for each soil type and some representative values of the damping ratio (0.005, 0.01, 0.02, 0.04, 0.07, 0.10, 0.20 and 0.30); Figure 4-4.a through Figure 4-4.e contain plots of B_d for the soil types A through E, respectively. The observation of these plots shows also little influence of the soil type. Hence, Figure 4-4.f presents plots of B_d averaged for all the soil types. Figure 4-5 displays similar plots of factor B_a .



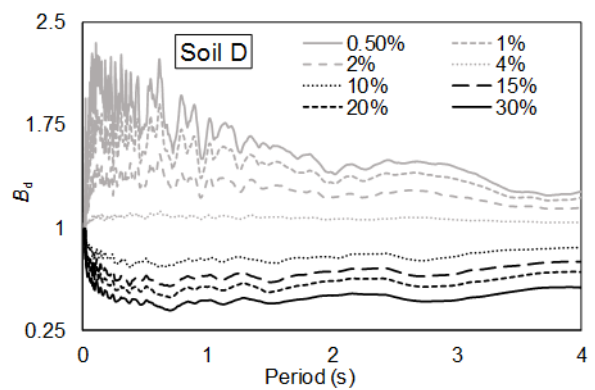
(a) B_d factor for soil type A



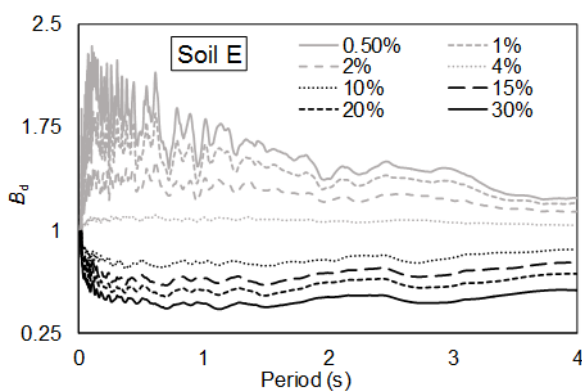
(b) B_d factor for soil type B



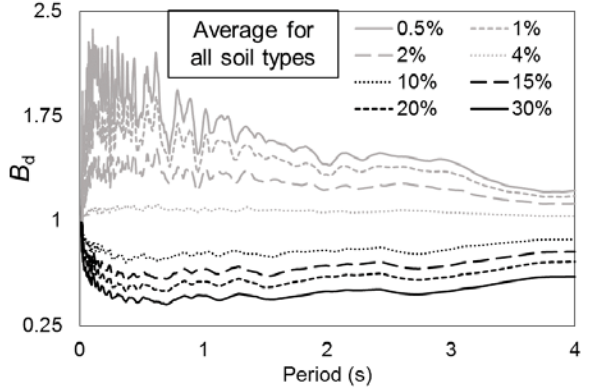
(c) B_d factor for soil type C



(d) B_d factor for soil type D

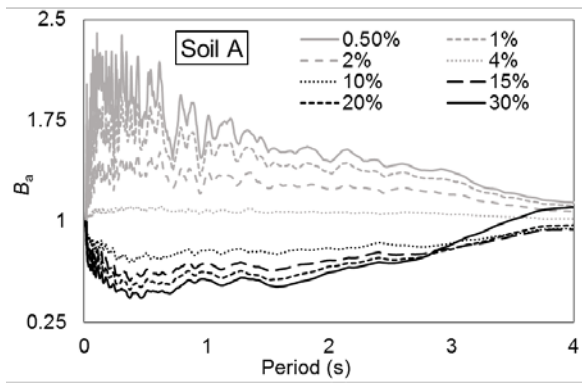


(e) B_d factor for soil type E

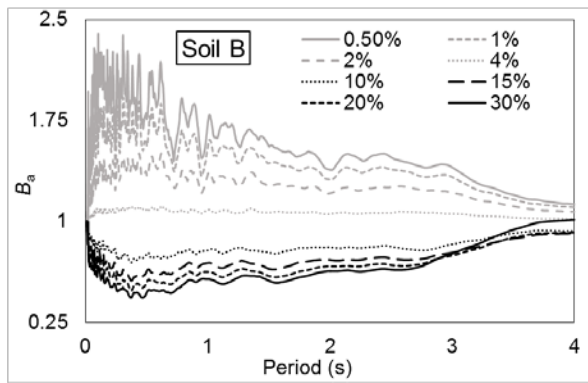


(f) B_d factor for all soil types

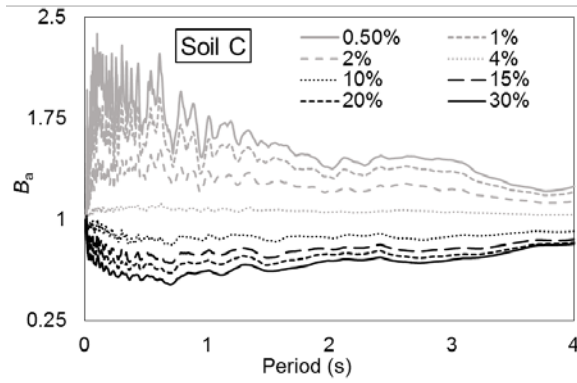
Figure 4-4. Spectra for factor B_d



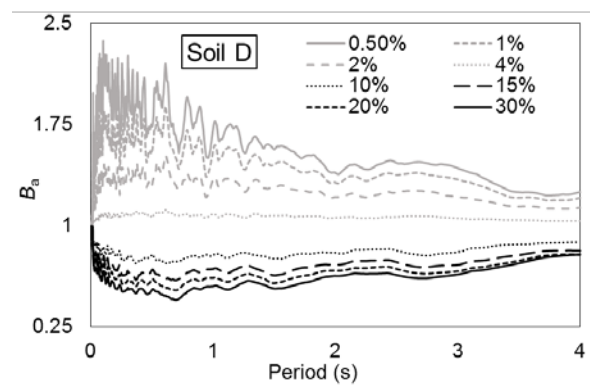
(a) B_a factor for soil type A



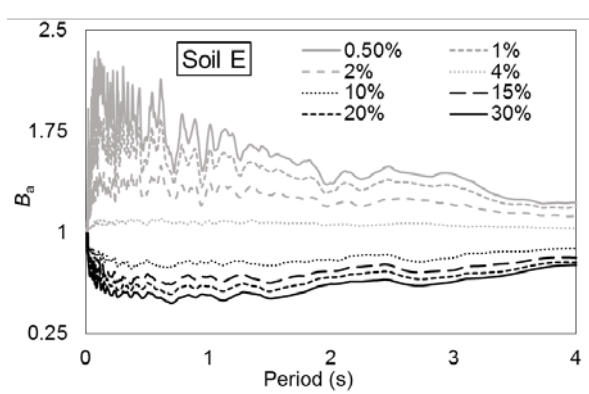
(b) B_a factor for soil type B



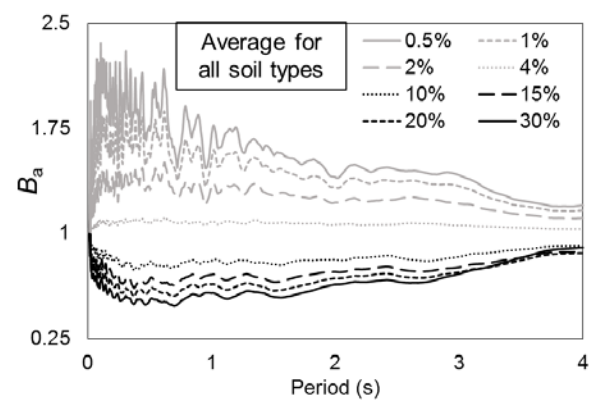
(c) B_a factor for soil type C



(d) B_a factor for soil type D



(e) B_a factor for soil type E



(f) B_a factor for all soil types

Figure 4-5. Spectra for factor B_a

Figure 4-4 and Figure 4-5 show regular behavior, with results fitting basically the previous studies (subsection 4.1.2). Noticeably, for short periods and low damping the dispersion is high.

4.3.5 Derived expressions for B_d and B_a factors

This subsection describes the genesis of the recommended fitting expressions for B_d and B_a factors. These relations are derived to match the plots in Figure 4-4.f and Figure 4-5.f, respectively. The starting point are the criteria provided for B_d and B_a in (Lin and Chang, 2004). For B_d , this work considers the expression $B_d = 1 - \frac{a T^b}{(T+1)^c}$, where coefficient a contains the influence of damping and $\frac{T^b}{(T+1)^c}$ represents the effect of period; $b = 0.3$, $c = 0.65$ and $a = 1.3033 + 0.436 \ln \zeta$. For B_a , $B_a = d + e T$; for $T > 0.2$ s, $d = 0.342 \zeta^{-0.354}$ and $e = 0.0186 + 0.368 (\zeta - 1) / 10.644 \zeta^2$, and for $T < 0.2$ s, a linear interpolation starting from $B_a = 1$ for $T = 0$ s, is suggested. The investigation (Lin and Chang, 2004) refers only to damping ratios greater than 5%; conversely, the spectra in Figure 4-4.f and Figure 4-5.f include also damping ratios smaller than 5%. The derivation of the matching expressions is described next for both cases.

Damping ratio higher than 5%. For B_d , the same expression proposed in (Lin and Chang, 2004). ($B_d = 1 - \frac{a T^b}{(T+1)^c}$) is considered. The process starts by selecting the values of coefficients a , b and c that better fit the spectra in Figure 4-4.f for damping ratio 30%; this operation is performed by nonlinear regression using the damped least-squares (Levenberg-Marquardt) algorithm implemented in Gnuplot (Williams et al., 2017). For damping ratios 50%, 45%, 40%, 35%, 25%, 20%, 15% and 10%, the values of coefficients b and c are kept constant and a is obtained with the same algorithm. For B_a , a trilinear fit is suggested; the coefficients of each linear segment ($B_a = d + e T$) are determined by linear regression. Table 4-3 displays the obtained values of coefficients a , b and c (B_d) and d and e (B_a).

Table 4-3. Coefficients of the derived expressions for B_d and B_a for damping ratio higher than 5%

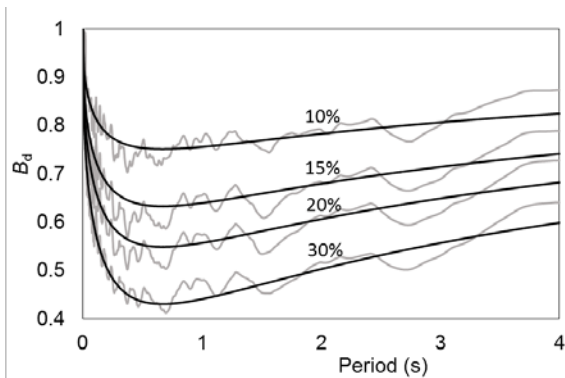
Damping ratio	$B_d = 1 - \frac{a T^b}{(T + 1)^c}$			$B_a = d + e T$					
				$T \leq 0.04 \text{ s}$		$0.04 \text{ s} < T \leq 0.5 \text{ s}$		$0.5 \text{ s} < T \leq 4 \text{ s}$	
	a	b	c	d	e	d	e	d	e
0.50	1.249	0.3683	0.9200	1.000	- 10.70	0.5873	- 0.3778	0.3184	0.1679
0.45	1.211	0.3683	0.9200	1.000	- 10.27	0.6047	- 0.3880	0.3368	0.1524
0.40	1.166	0.3683	0.9200	1.000	- 9.79	0.6241	- 0.3996	0.3585	0.1368
0.35	1.112	0.3683	0.9200	1.000	- 9.25	0.6461	- 0.3991	0.3849	0.1213
0.30	1.045	0.3683	0.9200	1.000	- 8,61	0.6716	- 0.3954	0.4178	0.1058
0.25	0.9603	0.3683	0.9200	1.000	- 7.84	0.7016	- 0.3846	0.4604	0.0903
0.20	0.8487	0.3683	0.9200	1.000	- 6.90	0.7385	- 0.3597	0.5184	0.0747
0.15	0.6912	0.3683	0.9200	1.000	- 5.65	0.7859	- 0.3027	0.6041	0.0592
0.10	0.4493	0.3683	0.9200	1.000	- 3.86	0.8528	- 0.1788	0.7496	0.0437

Damping ratio lower than 5%. For both B_d and B_a , the same expression proposed in (Lin and Chang, 2004) for B_d ($B_d = 1 - \frac{a T^b}{(T+1)^c}$) is considered. The process is analogous to the one described in the previous paragraph for B_d ; the considered damping ratios are 4%, 3.5%, 3%, 2.5%, 2%, 1.5%, 1% and 0.5%. Table 4-4 displays the obtained values of coefficients a , b and c .

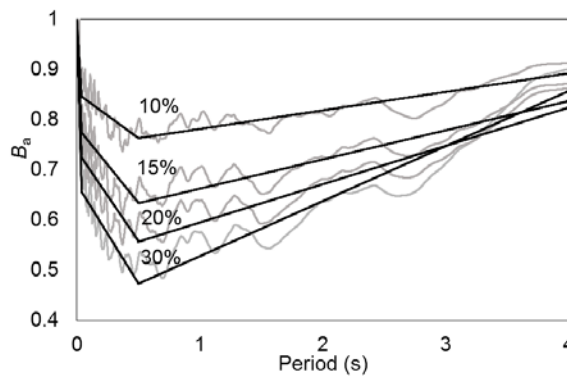
Table 4-4. Coefficients of the derived expression for B_d and B_a for damping ratio lower than 5%

Damping ratio	$B_d = 1 - \frac{a T^b}{(T + 1)^c}$			$B_a = 1 - \frac{a T^b}{(T + 1)^c}$		
	a	b	c	a	b	c
0.040	-0.2220	0.4685	1.399	-0.2449	0.4942	1.4673
0.035	-0.3632	0.4685	1.432	-0.3591	0.4828	1.4859
0.030	-0.5340	0.4685	1.472	-0.5179	0.4749	1.5022
0.025	-0.7463	0.4685	1.522	-0.7211	0.4649	1.5189
0.020	-1.017	0.4685	1.580	-0.9689	0.4471	1.5346
0.015	-1.378	0.4685	1.653	-1.2611	0.4157	1.5438
0.010	-1.905	0.4685	1.754	-1.5979	0.3652	1.5366
0.005	-2.815	0.4685	1.918	-1.9792	0.2898	1.4993

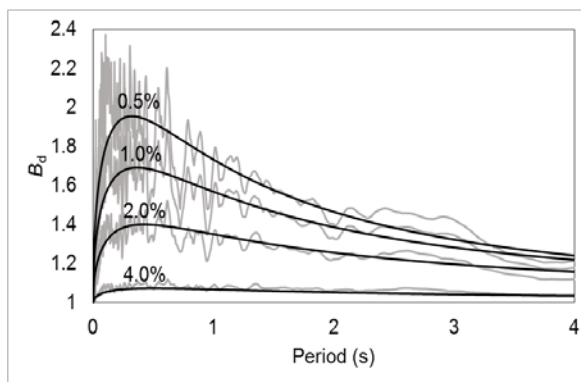
Figure 4-6 displays comparisons among the spectra represented in Figure 4-4.f and Figure 4-5.f and the fittings given by the expressions $1 - \frac{a T^b}{(T+1)^c}$, and $d + e T$, where coefficients a , b , c , d and e are given in Table 4-3 and Table 4-4.



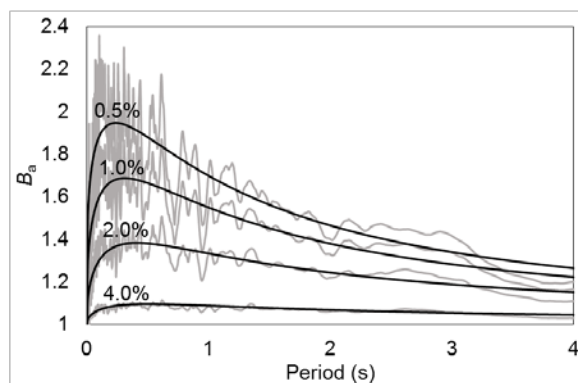
(a) Damping ratio greater than 0.05.
Factor B_d



(b) Damping ratio greater than 0.05.
Factor B_a



(c) Damping ratio smaller than 0.05.
Factor B_d



(d) Damping ratio smaller than 0.05.
Factor B_a

Figure 4-6. Comparison among the obtained spectra for factors B_d and B_a and the derived fits

Plots from Figure 4-6 confirm that the fits are correct.

The process is completed by fitting expressions of the variation of the coefficients a , b , c , d and e in terms of the damping ratio. The derived expressions are summarized in Table 4-5.

Table 4-5. Derived expressions for B_d and B_a

$\zeta > 0.05$ $B_d = 1 - \frac{a T^b}{(T + 1)^c}$	a		$1.621 + 0.4935 \ln \zeta$
	b		0.3683
	c		0.9200
$\zeta > 0.05$ $B_a = d + e T$	$T \leq 0.04 \text{ s}$	d	1
		e	$-789.9 \zeta^5 + 1445 \zeta^4 - 1071 \zeta^3 + 419.7 \zeta^2 - 100.6 \zeta + 2.938$
	$0.04 \text{ s} < T \leq 0.5 \text{ s}$	d	$-0.165 \ln \zeta + 0.4729$
		e	$-139 \zeta^5 + 248.6 \zeta^4 - 176.7 \zeta^3 + 63.64 \zeta^2 - 11.83 \zeta + 0.521$
	$0.5 \text{ s} < T \leq 4 \text{ s}$	d	$0.2202 \zeta^{-0.532}$
		e	$-0.2028 \zeta^2 + 0.4355 \zeta - 0.0026$
$\zeta < 0.05$ $B_d = 1 - \frac{a T^b}{(T + 1)^c}$	a		$3.789 + 1.238 \ln \zeta$
	b		0.4685
	c		$0.5941 - 0.2510 \ln \zeta$
$\zeta < 0.05$ $B_a = 1 - \frac{a T^b}{(T + 1)^c}$	a		$-890.2 \zeta^2 + 89.61 \zeta - 2.405$
	b		$7576 \zeta^3 - 724.6 \zeta^2 + 24.62 \zeta + 0.1839$
	c		$-274530 \zeta^4 + 32146 \zeta^3 - 1395 \zeta^2 + 23.27 \zeta + 1.414$

4.3.6 Comparison with actual accelerograms

This section presents a comparison among the results obtained in the previous section after artificial accelerograms and results derived from some available Colombian natural inputs.

Records inputs in Colombia

A number of historical accelerograms recorded in Colombia have been selected; the selection criteria are: local magnitude greater or equal than 6.0, and epicentral distance less than 210 km. This information has been retrieved from the Colombian Seismological National Network

(RSNC, 2017); noticeably, the soil type is taken from (Benavent-Climent et al., 2010). The time step is 0.005 s. For the base line correction, a constant polynomial type is used; then, a 4th order Butterworth filter with a bandpass configuration (0.1-20 Hz) is employed. Given that most of the available relevant records correspond to zone 5, the study is constrained to that area. Table 4-7 displays the main characteristics of the selected historic records.

For the sake of comparison with the historic inputs, Table 4-7 displays the average parameters of the suites of seven artificial inputs (subsection 4.3.3) that correspond to the same seismic zone and soil type. Table 4-7 shows that the available historic records are significantly less severe than the corresponding artificial ones, except for the strongest record of the Armenia earthquake (Table 4-7, 1999/01/25 event).

Table 4-6. Average parameters of the artificial inputs in zone 5

Soil type	PGA (cm/s²)	Trifunac duration (s)	Arias intensity (cm/s)	Housner intensity (cm)
A	275.89	10.29	82.52	82.68
C	408.01	10.53	207.36	156.07
D	462.58	10.51	276.52	186.32

Table 4-7. Selected Colombian records for seismic zone 5

Soil type	Earthquake epicenter	Date	Local magnitude (M_L)	Hypocentral depth (km)	Station	Epicentral distance (km)	Component	PGA (cm/s^2)	Trifunac duration	Arias intensity (cm/s)	Housner intensity (cm)
A	Calima	1995/02/08	6.6	102	CTRUJ	47.44	EW	109.36	17.36	8.15	29.21
							NS	93.09	18.04	8.11	22.53
					CSEVI	90.57	EW	80.46	22.46	11.67	9.79
							NS	45.94	25.60	4.69	9.85
	Risaralda	1995/08/19	6.5	120.90	CANSE	18.43	EW	80.93	24.85	11.24	9.35
							NS	166.51	25.10	34.15	9.99
	Córdoba	1999/01/25	6.3	0	CBOCA	38.48	EW	85.76	6.47	3.70	19.04
							NS	52.94	9.03	2.33	12.76
C	Génova	1997/09/02	6.8	230	CARME	69.58	EW	70.61	37.13	9.53	10.53
							NS	60.91	36.34	11.85	11.21
	Génova	1997/12/11	6.5	207.50	CCALI	83.48	EW	46.79	26.30	4.14	11.38
							NS	39.31	31.76	3.56	12.97
					CMAN1	137.10	EW	62.18	15.77	7.94	18.31
							NS	75.42	15.59	14.83	24.77
					CPER2	95.40	EW	46.65	22.07	6.39	12.56
							NS	45.57	22.68	6.80	12.58
	Córdoba	1999/01/25	6.3	0	CMAN1	79.45	EW	85.23	19.16	22.08	33.83
							NS	105.90	18.16	23.07	34.91
					CARME	13.02	EW	517.23	11.06	272.89	88.83
							NS	576.16	9.54	278.44	84.36
	Bajo Baudó	2004/11/15	6.7	26.2	CVERS	169.95	EW	49.76	50.85	16.14	16.28

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							NS	47.22	52.70	15.61	16.53
D	Génova	1997/12/11	6.5	207.50	CFLAN	86.32	EW	84.47	25.54	16.14	21.64
							NS	85.15	28.50	22.34	26.79
	Córdoba	1999/01/25	6.3	0	CFLAN	28.85	EW	573.67	14.32	296.59	125.21
							NS	506.67	10.13	425.71	129.84
	Bajo Baudo	2004/11/15	6.7	26.2	CCAL5	197.28	EW	54.70	35.34	9.37	57.10
							NS	31.46	49.00	6.02	40.40
					RAC03	199.16	EW	61.64	34.24	16.19	55.41
							NS	24.74	40.61	3.02	23.00
					RAC06	199.27	EW	79.48	38.60	25.32	88.11
							NS	37.34	30.21	6.29	38.32
					RAC07	195.25	EW	29.66	27.44	1.75	19.54
							NS	20.57	28.20	1.10	20.46
					RAC08	198.62	EW	46.56	48.69	11.65	55.06
							NS	29.31	37.85	4.47	37.69
					RAC10	204.07	EW	67.57	75.57	16.00	59.44
NS	23.00	63.52	4.94	34.57							
RAC11	207.25	EW	55.26	66.47	18.39	61.79					
		NS	25.20	63.58	4.62	32.44					

Comparison between spectra from artificial and natural accelerograms

Figure 4-7 displays comparisons among 30%-damped B_d spectra obtained after natural and artificial inputs. Figure 4-7.a, Figure 4-7.b and Figure 4-7.c correspond to soil A, C and D, respectively; each Figure contains plots corresponding to the strongest components of the individual records listed in Table 4-7, their average, and the average of the spectra derived after the artificial inputs that belong to the same seismic zone, damping level (30%) and soil type. Figure 4-7.d presents a similar comparison referring to the average for soils A, C and D. Figure 4-7.d. contains three types of plots: individual for all the records in Table 4-7 , their average, and the corresponding fitted expression displayed in Table 4-3.

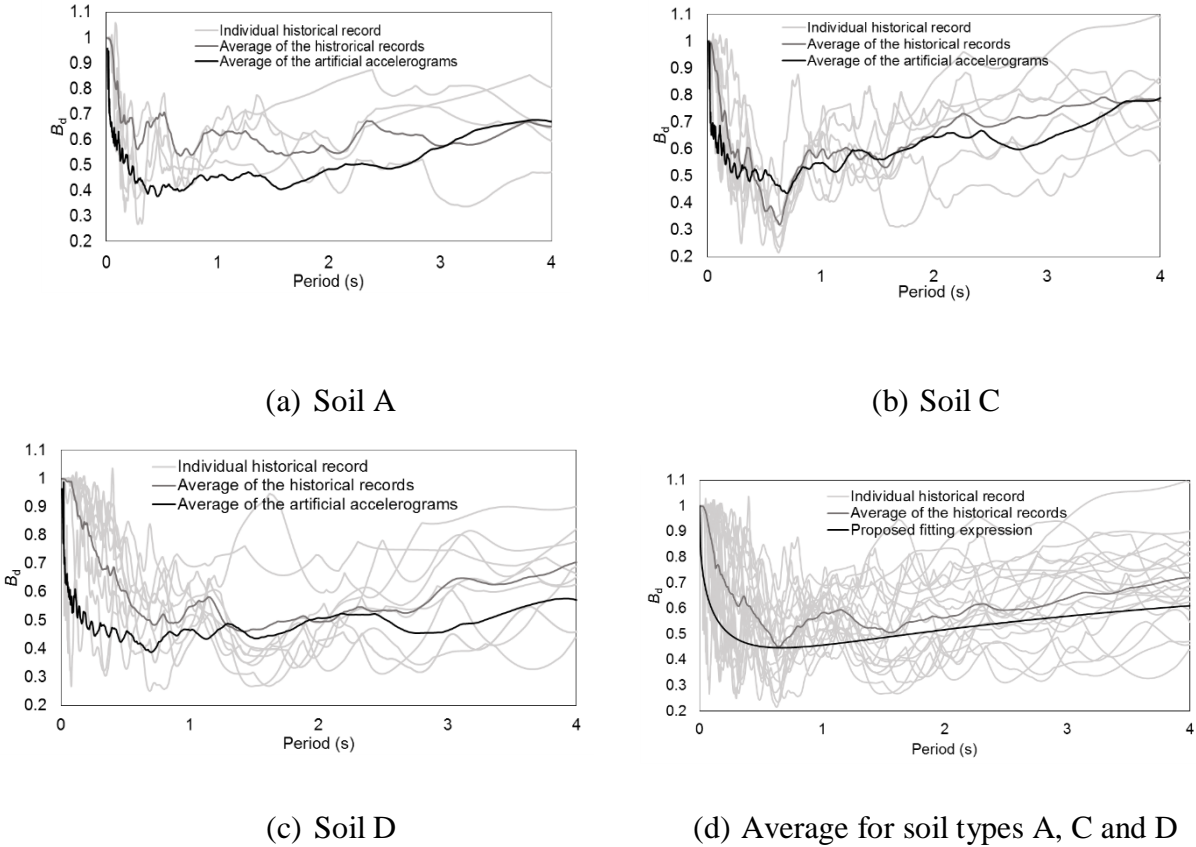


Figure 4-7. Comparison between B_d spectra for artificial and historic accelerograms in zone 5 for damping 30%

Plots from Table 4-7 show a reasonably satisfactory agreement between recorded and artificial inputs, given the extreme scarcity and high dispersion exhibited by the individual spectra obtained after the historic accelerograms. Noticeably, to assess the relevance of the observed discrepancies between both types of accelerograms, it should be kept on mind that the available historical inputs cannot be considered representative of the actual seismicity of Colombia.

4.3.7 Comparison with factors defined in codes and in the literature

This section compares the main output of this study (the matching expression for B_d factor presented in Table 4-3) with previous proposals, either from reported studies or from design codes. Figure 4-8 displays a comparison among the expression derived in this study for B_d and the results of previous researches (Lin and Chang, 2004; Saez et al., 2012) and of the codes of Chile (NCh 2745, 2013), Japan (BSL, 2009), USA (ASCE 7-10, 2010) Europe (EN-1998-2, 2004) and China (GB 50011, 2010). Figure 4-8.a, Figure 4-8.b, Figure 4-8.c, and Figure 4-8.d present B_d spectra for 30%, 20% 10% and 2% damping ratio, respectively.

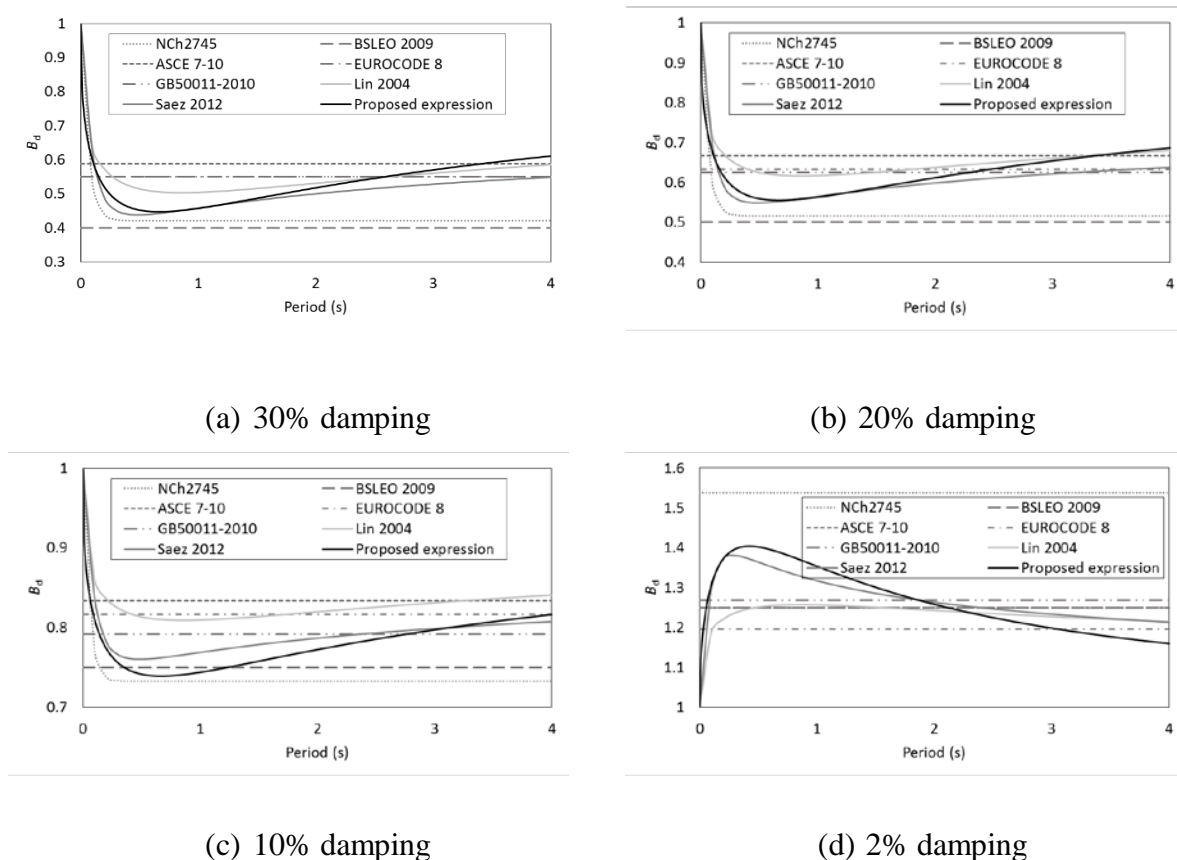


Figure 4-8. Comparison among B_d code spectra and the results of this and previous studies

Plots from Figure 4-8 show that the results of this study fit reasonably well those of the previous researches (Lin and Chang, 2004; Saez et al., 2012). Regarding the design codes, the derived expression lies within their range; noticeably, they show an important scattering.

The current Colombian design code (NSR-10, 2010) does not contain any criteria to modify the 5%-damped spectra; for seismic isolation, the USA regulations (ASCE 7-10, 2010) are used instead. Figure 4-8 shows relevant disagreements between that criterion and the results of this

study. Such differences can be due to the fact that the US accelerograms have shorter duration and are rather pulse-like (Saez et al., 2012). In any case, for the ranges of periods and damping ratios of interest in seismic isolation (period between 2.5 and 3.5 s, and damping ratio between 20% and 30%), the discrepancies are significantly smaller.

It is worth noting that the Japanese, European and Chinese codes do not allow further reducing the spectral ordinates for damping ratios higher than 30%; analogously, in the USA and Chilean regulations, such bound is 50%. This study corroborates this strategy, since, for damping ratios exceeding approximately 60%, the B_a factor can be greater than 1, thus generating an increase of the absolute accelerations (Piscal, 2018).

4.3.8 Verification example

This section discusses an application example on a 4-story RC hospital building (Piscal and López, 2016); this building is protected with base isolation and is located in Cali (Colombia). This study aims to verify the accuracy of the proposed approach by comparing the maximum design displacement of the isolation layer (D_D) determined with three different approaches: (i) equivalent lateral forces method by following the (ASCE 7-10, 2010), (ii) equivalent lateral forces method by following that regulation although considering the derived B_a factor, (iii) nonlinear time-history analysis using seven actual records that had been utilized in the microzonation of Cali (Decreto 411, 2014).

The building location refers to the “Clínica Confandi”, being the first base-isolated hospital in Colombia. The building has 4 stories and a RC framed structure; the plan is rectangular with sides 50.4 and 17 m and the seismic weight is 35021 kN. The foundation soil belongs to zone 4D according to the Cali microzonation (Decreto 411, 2014), being equivalent to soil D (NSR-10, 2010).

The isolation layer consists of 32 bearings; two types of isolator units are employed: natural rubber bearings (NRB) and lead-rubber bearings (LRB). Three isolation solutions have been investigated; they are differentiated by the number and characteristics of the isolators, and by the target values of the fundamental period and the damping ratio of the isolated building. Table 4-8 displays the main features of the analyzed isolation solutions.

Table 4-8. Isolation solutions for the example hospital building

Solution No.	LRB			NRB		Damping ratio (%)	Fundamental period (s)
	No.	Diameter/Height* (mm)	Lead plug diam. (mm)	No.	Diameter/Height* (mm)		
1	20	450/96	90	12	450/60	16.69	2.15
2	24	400/126	78	8	400/78	19.76	3.00
3	32	500/210	120	-	-	29.84	2.42

*Rubber height (sum of the thickness of each rubber layer)

In the aforementioned static equivalent approaches, the isolators displacement is determined after the expression $D_D = \frac{S_a T^2}{4\pi^2 B}$, where T is the fundamental period of the isolated building in the direction under consideration, S_a is the spectral ordinate corresponding to this period, and B is the reduction factor due to damping. S_a is determined after the corresponding design spectra in the Cali microzonation (Decreto 411, 2014). In the first approach, $B = 1.40, 1.49$ and 1.70 for solutions 1, 2 and 3, respectively. In the second approach, B is represented by the derived coefficient B_d (Table 4-3 and Table 4-5); $B_d = 0.662, 0.658$ and 0.543 for solutions 1, 2 and 3, respectively.

In the third (time-history) approach, seven historic accelerograms that were considered in the Cali microzonation (Decreto 411, 2014) have been selected. Such inputs were recorded in rock; they have been scaled (Kottke and Rathje, 2008) to correspond to soil D and to match the aforementioned design spectrum as required by the Colombian regulation (NSR-10, 2010).

Table 4-9 displays the maximum displacements in the isolation layer corresponding to the three analyzed isolation solutions (Table 4-8) and the three aforementioned analysis approaches. Results in Table 4-9 correspond to any horizontal direction.

Table 4-9. Maximum displacement (D_D) in the isolation layer (cm)

Solution No.	Equivalent forces ASCE 7-10	Equivalent forces method using the proposed formulation	Time-history analysis
1	26.41	24.20	22.32
2	24.81	24.27	21.93
3	21.93	20.22	15.52

Table 4-9 shows that the proposed formulation provides better agreement with the allegedly more exact dynamic results than the formulation contained in the American regulations (ASCE 7-10, 2010). This can be read as a verification of a better suitability of the proposed approach to the particular Colombian conditions.

5. RELEVANT ASPECTS TO TAKE INTO ACCOUNT IN THE DEVELOPMENT OF SEISMIC ISOLATION CODES OF BUILDINGS FOR COLOMBIA AND LATIN AMERICA.

Base seismic isolation is a technique that consists of decoupling the foundation soil of the structure, in order to avoid that the energy coming from a seismic event will be transmitted directly to the building and causes damage in it. Currently, approximately 14,000 buildings with this technology exist worldwide, many of them have been exposed to severe earthquakes without structural damage, so demonstrating the effectiveness of the technique. However, the rise of base isolation application is relatively recent, so the codes that govern the design of this type of building are relatively new, have undergone considerable changes in recent years or are in the process of being created. In Latin America, only Chile and Mexico currently have regulations for buildings with base isolation. Countries such as Colombia, Ecuador and Peru have been working to develop local regulations. Latin American countries have in their norms an important influence of their American counterparts, which in the case of base isolation have been questioned by several authors (Kelly, 1999; Mayes, 2014; Naaseh et al., 2002), due to the possible overcharges they could generate and the possible degree of conservatism that they could possess. This suggests that to promote this technology in Latin America, it is necessary to study in detail each of the requirements contained in international standards, to define whether each of them must be adopted or adapted to the conditions of developing countries. This chapter presents an analysis of the key philosophical aspects stipulated in the seismic isolation codes, with emphasis in those that are most influential for the region, namely American (ASCE 7-10, 2010; ASCE 7-16, 2016) and Chilean codes (NCh 2745, 2013b). The analysis includes a comparison with the philosophy used in design of conventional structures and finally a general proposal containing relevant considerations for the generation of seismic isolation codes at Latin American countries and specially in Colombia is established.

5.1 Analysis of design philosophy stipulated in codes for seismic isolated buildings

In fixed based buildings (without base isolation) the objective of the earthquake resistant design is usually dual (combining two performance levels for different earthquakes), to avoid both loss of human life and global collapse of structures (Tsompanakis, 2015). Reaching this performance level, implies for the structures inelastic behavior and therefore damage. In buildings with base isolation, it is possible reach the same aforementioned performance levels; however, taking advantage of the elastic forces' reduction obtained with this technique the design is carry out to higher level of security (higher performance), therefore the international codes want significantly to reduce the level of damage of the structures and its contents, making that structural behavior will be closer to elastic than inelastic (ASCE 7-16, 2016; NCh 2745, 2013b). This is true except in the case of Mexico, where a called partial isolation is allowed, with the presence of moderate structural damage in the building. To reach a specific performance level, both in buildings with seismic isolation and in fixed based buildings, the codes are supported on the following definitions:

- Seismic hazard level
- Importance factor
- Earthquake´s Return period and performance
- Forces reduction factor
- Structural damping
- Ductility requirements
- Interstory Drift

5.1.1 Seismic Hazard level

The seismic hazard is expressed in the earthquake resistant design codes, through the denominated design spectra. Because a lot of projected buildings generally have natural vibration periods less than 3 seconds, the spectra in some cases are constructed from records whose low frequencies have been filtered. In the case of base isolation, the important periods generally range between 2 and 4 seconds, so the definition of the hazard for each country must be carefully studied. A relevant case are the Chilean spectra, where there is currently a spectrum for isolated buildings, characterized by greater demands for flexible structures (NCh 2745,

2013b) and a spectrum for fixed based buildings, calibrated with the response of the type of buildings most used in its time (Almazán, 2012; NCh433, 1996) Figure 5-1 shows the differences between both spectrum, calculated for the same conditions, namely: soil type B (II), seismic zone I and importance factor equal to 1.

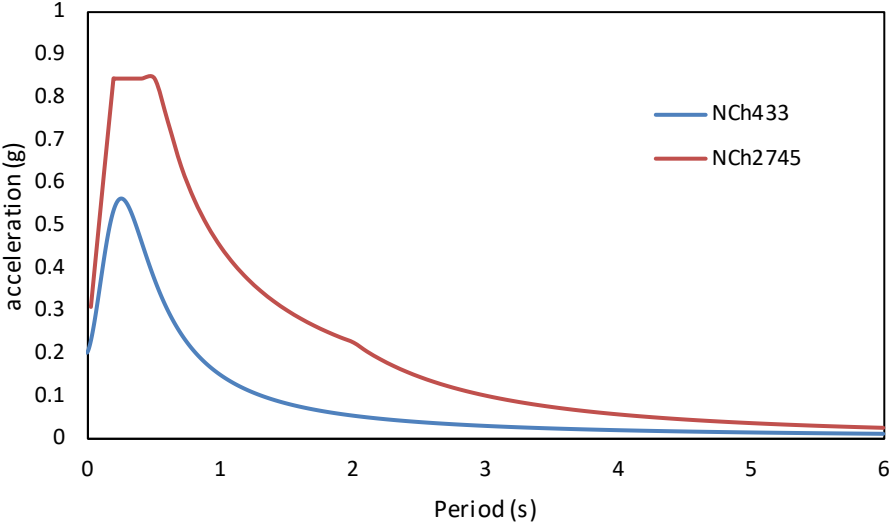


Figure 5-1. Comparison between Chilean spectra

5.1.2 Importance factor

ASCE 7-16 defines the risk categories (*RK*) (related to building use, South American codes) namely: ***RK I***. Building and structures uninhabited or with low occupation. Its fail means a low risk for the population. ***RK II***. Residential, commercial, industrial buildings. Here are included a lot of typical structures. ***RK III***. Buildings and structures with high occupancy, buildings accommodating people with limited mobility. Structures containing hazard substances and structures with high economical losses and high interruption of civil activities when they are not functionally. ***RK IV***. Structures with essential services for the population in emergency cases

Risk categories are defined in function of number of people at risk of losing their lives, due to lack of structures' operability. Figure 5-2 shows an approximate relationship among lives' number in risk and the risk categories.

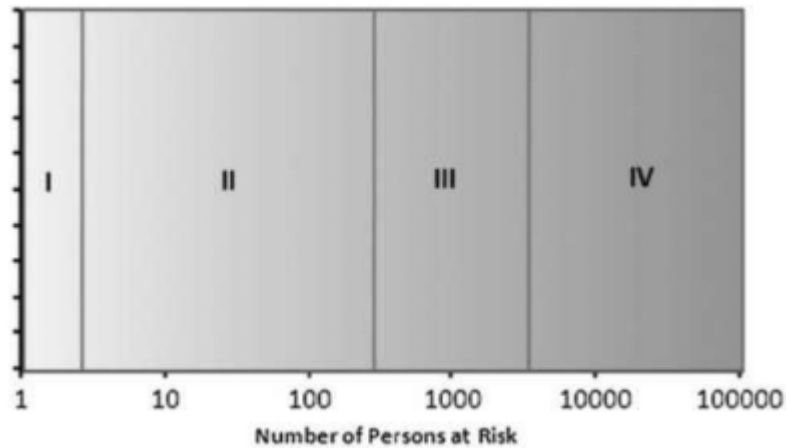


Figure 5-2. Relation among lives' number in risk and the risk categories. (ASCE 7-16, 2016)

For each risk category (building use in South American codes) there is a design earthquake with a defined return period. Essential structures (*RK IV*) are designed with the highest earthquake, while low risk structures (*RK I*) are designed with the lowest one, in USA case. To obtain aforementioned return periods, the codes use the importance factors (*I*). Table 5-1 shows importance factors used in USA (ASCE 7-16, 2016), Europe (EN-1998-2, 2004) and some of the used in South America (Peru/Chile/Ecuador/Colombia) (E030, 2014; NCh 2745, 2013b; NEC, 2015; NSR-10, 2010)

Table 5-1. Importance factors

Grupo de uso	USA	EUROPA	SOUTH AMERICA
IV	1.50	1.40	1.50/1.50/1.50/1.20
III	1.25	1.20	1.25/1.30/1.30/1.20
II	1.00	1.00	1.10/1.00/1.00/1.00
I	1.00	0.80	1.00/1.00/1.00/0.60

Importance factors multiply spectral ordinates (accelerations), therefore these affects basically the return period off the design earthquake considered (EN-1998-2, 2004; Tsompanakis, 2015). Figure 5-3 shows for Eurocode 8 case ($k=3$) how importance factor affects the considered earthquake return period. It is evident that an importance factor value equal to 1.0 represent an earthquake return period near to 500 years, while an importance factor value equal to 1.6 represent an earthquake return period near to 2000 years. In USA code $I=1$ and $I=1.5$ correspond to earthquakes with return period of 475 and 2475 years respectively.

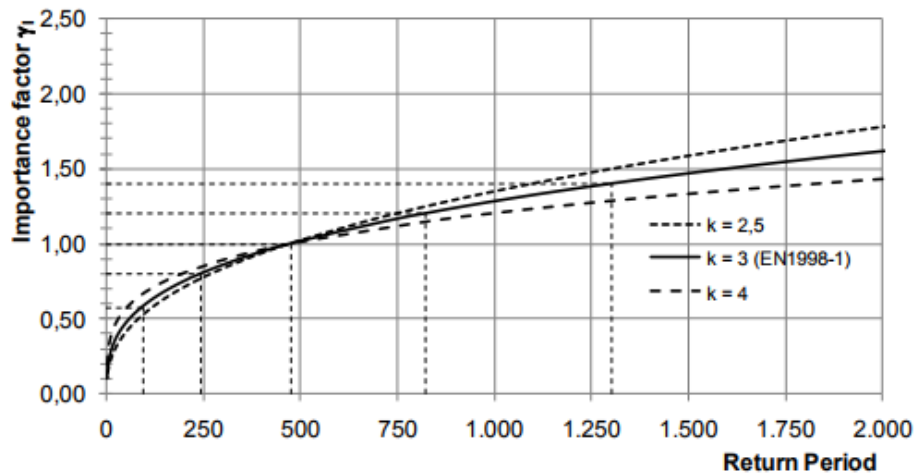


Figure 5-3. Relation among importance factor and earthquake return period considered (EN-1998-2, 2004)

Regard isolated structures some codes like Chilean (NCh 2745, 2013b) and American (ASCE 7-16, 2016) specify for this type of structures an importance factor always equal to one. Previous consideration is founded on: a) the philosophy of an equal performance level for all types of isolated structures, without have into account differences due to risk categories, b) higher certainty in the seismic request for the structure (ASCE 7-16, 2016; NCh 2745, 2013).

5.1.3 Earthquake return period (T_R) and performance

In the codes a defined earthquake return period is associated to a specific performance level in fixed based buildings. The return periods generally used in some codes are described in the Table 5-2. Each code or document denote the earthquake associated to a specific return period with a different name, for instance design earthquake or rare for $T_R=475$ years, and maximum considered earthquake or maximum possible earthquake for $T_R=2475$ years (ASCE 7-16, 2016; NCh 2745, 2013; VISION, 2000). In this work the earthquakes are called basically like in the Chilean and Colombian code, it means frequent, moderate or severe. The values corresponding to each type of earthquake are the typically specified in international codes.

Table 5-2. Earthquakes types considered in codes for seismic design of buildings

Earthquake	Return period (years)	Exceedance probability
Frequent	43	50% in 30 years
Small	72	50% in 50 years
Moderate	475	10% in 50 years
Severe	2475	2% in 50 years

It is important to note that in Chilean case, the return period of the severe earthquake corresponds to a probability of exceedance of 10% in 100 years ($T_R=950$ years), which is the same value stipulated for the rare earthquake in VISION2000 document.

The structural performance levels post-earthquake are described briefly below (ATC-40, 1996; FEMA 356, 2000; Tsompanakis, 2015; VISION, 2000):

- **Operational.** Continuous service. Negligible structural and nonstructural damage.
- **Immediate Occupancy.** The structure retains its pre-earthquake characteristics like design strength and stiffness for vertical and lateral force resisting system. Very low structural damage may occur, but without high risk of life-threatening injury. Some structural repairs may be appropriate without affect safe to occupy the building. None permanent drift is observed.
- **Life safety.** Moderate structural damage, injuries but with low overall risk of life-threatening may occur. Permanent drift may be encountered. The structure retains a margin against partial or total collapse, therefore it may be repaired prior to occupancy, if it is possible from economical point of view.
- **Collapse prevention.** Substantial damage with significant degradation of stiffness and strength of the lateral force resisting system, but a more limited degradation in vertical load capacity that makes that the structure has still a margin against collapse. There is risk of injury, the structure is not safe for reoccupancy and neither may be practical to repair it. Aftershock activity could induce collapse. The structure has large permanent drift.

The performance expected for fixed base buildings depends on use of it and type of design earthquake considered. ASCE 7-16 shows this criteria through Table 5-3, for ordinary, high occupancy and essential buildings. It is evident that for ordinary buildings, a performance level of Life Safety (*LS*) is expected for a moderate earthquake ($T_R=475$ years), while in Essential buildings this performance level is reach for a severe earthquake ($T_R=2475$ years). It means that for the same earthquake the performance level is higher in essential structures regard conventional ones.

Table 5-3. Expected performance as related to risk category and level of ground Motion

Ground Motion	Performance level			
	Operational	Immediate Occupancy	Life Safety	Collapse Prevention
Frequent (43)	■	▲	●	Inadequate performance
Moderate (475 years)		■	▲	
Severe (2500 years)			■	

To satisfy dual performance level, for earthquakes greater than design earthquake a series of exigent requirements regard structural detailing are defined in the codes.

In isolated structures, the performance expected does not depends on neither use of it nor type of design earthquake considered, because the same performance level is expected for any type of structure with seismic isolation and therefore only one return period of the design earthquake is considered. Important changes have been presented in the two more recent versions of ASCE 7 regard to the return period of design earthquake, ASCE 7-10 stipulated a design earthquake for superstructure and substructure with return period (T_R) of 475 years, while ASCE 7-16 stipulated $T_R=2475$ years

Codes like ASCE 7 indicates the higher performance of isolated structures regard fixed base structures, through Table 5-4. In the Table 5-4 both for moderate and severe earthquakes the performance level is more exigent in isolates structures (*i*). In the case of moderate earthquakes, for instance, isolated structures must not have significant nonstructural or content damage, to reach this performance level the structural behavior must be close to elastic.

Table 5-4. Performance expected for isolated structures

Performance Measure	Earthquake Ground Motion level		
	frequent	Moderate	Severe
Life Safety: Loss of life or serious injury is not expected	<i>f, i</i>	<i>f, i</i>	<i>f, i</i>
Structural damage: Significant structural damage is not expected	<i>f, i</i>	<i>f, i</i>	<i>i</i>
Non structural damage: Significant nonstructural or content damage is not expected	<i>f, i</i>	<i>I</i>	<i>i</i>

f = fixed building, i= isolated building

5.1.4 Forces reduction factor (R)

In fixed base buildings, the generally important incursion of these structures in the inelastic range, implies damage and therefore dissipation of energy. Therefore, in some codes, there are R values that can reach up to 7. Due to in seismic isolated structures the target performance expected wants to avoid damage of structural and nonstructural components, it is necessary an elastic behavior for the superstructure and the substructure, and therefore its stiffness and strength must be related to aforementioned behavior (NCh 2745, 2013). For this reason, the factors used to calculate design forces in isolated structures are lower than in structures with fixed base.

ASCE 7 recommends using the following force reduction factor in isolated structures:

$$2 \geq \frac{3}{8}R \geq 1 \quad (5-1)$$

Where R is the value used in fixed base buildings

ASCE 7-16 in the exception of the chapter 17.5.2 allows the use of R values greater than 2, however from the point of view of the author this would go against the performance level expected in this type of buildings.

5.1.5 Structural damping

Structural damping in fixed base buildings traditionally has been considered like 5% of critical damping, these value used to represent high deformation without stability loss, appear coherent with the ductility and tenacity needed in the current buildings (Chopra, 2001; NSR-10, 2010; Sarria, 2008). These values have been obtained of experimental measures, with high consistency for these type of buildings (Chopra, 2001). For isolated structures a higher structural damping corresponds to isolation system, generally range between 5% and 50% of critical damping, while for superstructure and substructure low values (< 5%) of this damping are required to be coherent with the expected behavior (small deformation, near to elastic)

5.1.6 Ductility requirements

Although the behavior expected both for superstructure and substructure is essentially elastic, and therefore ductility requirements would not be necessary to support design earthquake. Codes like ASCE 7-16, let us understand that for isolated structures the ductility requirements must be the same that for conventional ones. This consideration can be explained only like a safety factor due to doubts about that structures will have a perfectly elastic behavior under design earthquake and a safety factor to support earthquakes bigger than design to reach possible a dual objective such as in conventional structures. Chilean code defined in former version the ductility like of special category while in the current version this requirement has been changes to intermediate category.

5.1.7 Interstory drift

The drift limits in the codes correspond to a specific performance level expected and therefore these are associated to specific damage level. The damage level can be defined for structural elements, nonstructural drift sensitive elements and nonstructural acceleration sensitive elements. Different authors had studied the relationship between the drift and the damage or performance for different types of elements, (Ghobarah, 2004) studied the relationship between the drift and various damage levels for different types of concrete structures, namely moment resisting frames (MRF) both ductile and nonductile, MRF with infills, ductile and squat walls Table 5-5.

Table 5-5. Drift limits (%) associated with various damage levels (Ghobarah, 2004)

Damage state	Ductile MRF	Nonductile MRF	MRF with infills	Ductile walls	Squat Walls
No damage	<0.2	<0.1	<0.1	<0.2	<0.1
Repairable damage					
a) Light damage	0.4	0.2	0.2	0.4	0.2
b) Moderate damage	<1.0	<0.5	<0.4	<0.8	<0.4
Irreparable damage (yield point)	>1.0	>0.5	>0.4	>0.8	>0.4

Sever damage- Life Safe- partial collapse	1.8	0.8	0.7	1.5	0.7
Collapse	>3.0	>1.0	>0.8	>2.5	>0.8

(Aslani and Miranda, 2005) studied, among other things, relationship of the drift and peak floor acceleration with various damage level, for nonstructural drift sensitive and acceleration sensitive components (Table 5-6)

Table 5-6. Statistical parameter for fragility functions of generic nonstructural drift-sensitive and acceleration-sensitive components

Damage state	Peak Interstory Drift ratio		Peak Floor Acceleration	
	Median (%)	Dispersion ¹	Median (g)	Dispersion ¹
Slight	0.4	0.5	0.25	0.6
Moderate	0.8	0.5	0.50	0.6
Extensive	2.5	0.5	1.00	0.6
Complete	5.0	0.5	2.00	0.6

¹Defined as the logarithmic standard deviation of the demand

In conclusion, all the authors agree with the fact that for low or not damage in structural and nonstructural elements the drift limit must be small, range between 0.2 and 0.6 which is coherent with the recommendations gave by VISION2000 document (Table 5-7)

Table 5-7..Drift limits related to performance levels

Performance level	Drift limit %	Permanent drift limit %
Operational	0.2	--
Immediate Occupancy	0.5	--
Life Safety	1.5	0.5
Collapse prevention	2.5	2.5

It's important to note that some codes considered for seismic isolated structures drift limits near to 1.5% and does not near to 0.5% for control damage.

5.2 New considerations for seismic isolated buildings codes for Colombia and Latin America

It is important that in Latin American Countries an adequate adaptation of ASCE 7-16 code will be developed. The direct application of aforementioned code could impose isolated structures with technic and economical inconsistencies. The follow aspects must be having into account for the future Latin American codes of isolated buildings, additionally a new proposal is here developed to incentive this technology.

5.2.1 Importance factor

Any seismic isolated buildings must have the same performance level, however it does not necessary for the same earthquake's return period. The proposal is to consider the same importance coefficients used in fixed base buildings to define different earthquake return period for each use group. These consideration is coherent with both the performance level proposal of the numeral 5.2.2 and with the earthquake's return period defined in some Latin American codes, included Colombia, to stablish drift limits.

5.2.2 Earthquake return period and performance

Seismic isolation technique should not be defined like a way to reduce buildings' costs regard to conventional ones (fixed base). This technique should be catalogued like a solution to reduce design seismic force to reach higher performance in buildings. Table 5-8 and Table 5-9 show the autor's proposal about the expected performance in base isolated structures with different group uses. Like a way to compare, the performance stipulated in codes for fixed base buildings has been incorporated in these tables. It is important to note that these tables are applicable to sub and superstructure and not being applicable to the insulation layer

Table 5-8. Expected performance for both isolated buildings and fixed base buildings (Use group I)

Earthquake	Performance level			
	Operational (FO)	Immediate Occupancy (IO)	Life Safety (LS)	Collapse prevention (CP)
Frequent*	●	●	Inadequate performance	
Moderate**	●	●	●	
Maximum***	●	●	●	●

(*) $T_R = 72$ years; (**) $T_R = 475$ years; (***) $T_R = 2475$ years

Table 5-9. Expected performance for both isolated buildings and fixed base buildings (Use group IV)

Earthquake	Performance level			
	Operational (FO)	Immediate Occupancy (IO)	Life Safety (LS)	Collapse prevention (CP)
Frequent*	●	●	Inadequate performance	
Moderate**	●	●	●	
Maximum***	●	●	●	●

(*) $T_R = 72$ years; (**) $T_R = 475$ years; (***) $T_R = 2475$ years

For intermediate cases, performance expected is showed in Table 5-10.

Table 5-10. Expected performance for both isolated buildings and fixed base buildings (Use group intermediate)

Earthquake	Performance level			
	Operational (FO)	Immediate Occupancy (IO)	Life Safety (LS)	Collapse prevention (CP)
Frequent*	●	●	Inadequate performance	
Moderate**	●	●	●	
Maximum***	●	●	●	●

(*) $T_R = 72$ years; (**) $T_R = 475$ years; (***) $T_R = 2475$ years

Like summary, the proposal consists in increment in one level the performance expected for fixe base buildings. It's mean, for a specific earthquake, in all aforementioned tables, the performance of the isolated structure is obtained moving to the left the performance of the fixed base building

Regarding to return period of design earthquake, it is a very important issue and it is possibly the issue with more repercussion in the application of seismic isolation in Latin American countries. The selection of return period for design earthquake in substructure, superstructure

and isolation system here it is discussed. For superstructure and substructure the proposal is accord to Table 5-8, Table 5-9 and Table 5-10, considering the same earthquake return period used for fixed base buildings, which is a function of group use. These solution is accord to seismic isolation philosophy” *the same performance for any isolated building*”, but define in front of what earthquake is expected such behavior. On the contrary if immediate occupancy is selected as expected performance in isolated structures, and an earthquake return period of 2475 years is used, an economical cost overrun could be induced in normal use buildings. If an earthquake return period of 475 years is used, neither additional benefits would be obtained in essential buildings, this consideration is showed in Table 5-9. The use of a single return period for design of any isolated structures could not be very consistent from economical and technical point of view. (Morgan and Mahin, 2011) affirm “*the requirements defined for essential structures, could be inappropriate or expensive for ordinary buildings whose functionality or damage state following a major earthquake is not critical to either the public welfare or the financial solvency of an organization*”. For isolation system, it is recommended a design earthquake return period of 2475 years.

5.2.3 Forces reduction factor

Due to in seismic isolated structures the target performance expected wants to avoid damage of structural and nonstructural components, it is necessary an elastic behavior for the superstructure and the substructure, and therefore its stiffness and strength must be related to aforementioned behavior (NCh 2745, 2013). For this reason, the factors used to calculate design forces in isolated structures are low, and it does not correspond to reduction due to ductility with damage associated but to overstrength due to way how the structures are designed nowadays, hence in this work forces reduction factor is not called R but Ω to avoid generate confusion in the performance expected. Seismic codes define, for this type of structures, reduction factors that range between 1 and 2 with the following justifications.

- Factors near to 2, can cause that some structural element enter to inelastic behavior, however all the lateral resistant system does not change notably its behavior or principal characteristic (NCh 2745, 2013).
- Taking into account that design of structural systems is based on strength design procedures. A factor of at least 2, is assumed between calculated design force and the true yield level of the structural system. An investigation of 10 specific buildings, indicate that these value

range between 2 and 5 (ATC-10, 1982). Thus, a reduction factor of 2 is appropriate to ensure that the structural system remains essentially elastic for the design earthquake (ASCE 7-16, 2016; FEMA 450, 2004).

Although expected performance for substructure is the same than superstructure, and therefore the Ω factors could be the same, some codes like ASCE 7-16 stipulate that $\Omega=1$ for substructure. These criteria look for a major security level for this part of the building, knowing that the fall of it implies fail of the isolation system and superstructure.

5.2.4 Structural damping

Spectra included in Latin American codes generally has been developed for 5% of critical damping, the addition of isolation systems to buildings increase these values of damping, generally ranging between 5-50% of critical damping. Codes like ASCE 7 has estimates a series of factors that convert the 5% spectrum to spectrum with different damping values, these factors are used in some countries like Colombia, without previous studies that demonstrate its applicability, since American factors are calculated in function of local seismic hazard. It is important that each country develop your own factors in function of its local earthquakes characteristics.

5.2.5 Ductility requirements

Following the performance expected for isolated structures (Immediate Occupancy IO), a behavior close to elastic is necessary in the buildings, therefore ductility requirements must not be the same used in fixed base buildings.

The proposal is to reduce in one level the requirements stipulated for fixed base buildings, it means for instance, in the case of fixed base buildings with especial ductility requirements, the corresponding seismic isolated buildings will have moderate ductility requirements. It is important to note that minimum ductility requirements always must be used in seismic isolated buildings.

5.2.6 Interstory drift

Interstory drift is a parameter to control damage in the structures, following the values stipulated in the numeral 5.1.7, for isolated structures drift's limits coherent with all here discussed are showed in Table 5-11. The drift limits correspond to moderate earthquake

Table 5-11. Drift limits for isolated structures with different use groups

Parameter	Use group	
	I	IV
Drift limit %	0.5	0.2

5.2.7 Expression for FHE analysis

ASCE 7 defines the expression used for static linear analysis or equivalent horizontal forces methodology (FHE) as follow:

$$S_d = \frac{S_a}{\omega^2} \quad (5-2)$$

In these expression S_d y S_a are: displacement spectrum and accelerations pseudo spectrum respectively, angular frequency ω is defined in function of building's period (T) as follow:

$$\omega = \frac{2 \pi}{T} \quad (5-3)$$

Replacing ω in S_d and dividing by B (Coefficient of reduction of response by damping) it is obtained:

$$S_d = \frac{g S_a T^2}{4 \pi^2 B} \quad (5-4)$$

Equation (5-4) is a general expression and could be used for any country interested in design isolated buildings using FHE procedure. The expression stipulated in ASCE 7 is the equation (5-4) simplified with the particularity of isolated structural periods always corresponding to velocity constant zone in the spectrum.

6. SUMMARY, CONCLUSIONS AND FUTURE INVESTIGATIONS.

6.1 Summary

This work presents a study of different parameters used in the design of seismic isolated structures, analyze them and makes a proposal about how could be adequately used in future Latin American codes, especially in Colombian case. The study is divided in three principal chapters, the first one makes a comparison between international codes requirements about seismic isolation, and a numerical example on a prototype building analyzed both with static linear analysis and nonlinear dynamic analysis is developed. The second chapter presents a new proposal to estimate damping modification factors used to modify 5% spectrum due to high damping imposed by isolation systems, the novelty obey to the use of artificial accelerograms in zones with local records scarcity. The third chapter presents a general proposal about important parameters required in design of isolated structures, this proposal gives a new conception about the expected performance in these types of buildings and aims to encourage this technique for any kind of building in Latin American countries, especially in Colombia.

Next sections discuss the most important conclusions of this study. The conclusions are organized in three categories: international buildings codes requirements, damping modification factors and proposal for future Latin American codes. Finally, the generalizability of the results of this study is discussed.

6.2 Conclusions

6.2.1 International buildings codes requirements

This subsection presents a comparison among design codes for base isolation of countries where this technology is most spread: Japan, China, Russia, Italy, USA Chile and Mexico. Design of a sanitary building, located in high and medium seismicity zones, according to analyzed codes is compared.

This study shows that there are enormous discrepancies among compared codes. Russian code is the conservative, apparently mainly because of its low specificity for base isolation. Chinese code is also highly conservative, mainly the simplified analysis strategy. American regulations exhibit a considerable degree of conservatism, and even new version is more demanding. Level of conservatism of Japanese regulations is comparable to USA codes. Chilean code is significantly less demanding than American ones. Italian regulation is least demanding, mainly for non-essential facilities; this conclusion can be extended to all countries whose regulations are based in Eurocodes. If code prescriptions are normalized with respect to return period, three major changes are observed: dispersion among analyzed countries is significantly reduced, Chilean code becomes more conservative than Italian one, and new USA code is less demanding than old one. Regarding Chinese and Japanese regulations, consideration of nonlinear behavior of superstructure in time-history analyses, might generate less demanding conditions when such approach is utilized.

More detailed conclusions are discussed next. They are separated into general (e.g. applicable to any building) and particular (e.g. applicable to the prototype sanitary buildings).

General conclusions are:

- **Seismic hazard.** Return period for designing superstructure ranges between 475 years (Japan, former US and Chile) and 2500 years (China and new US). Regarding isolators, it ranges between 500 years (Japan) and 2500 years (China and USA).
- **Importance factor.** Italian code proposes coefficients equal to those for fixed-base buildings. In the other codes, it is equal to one.
- **Reduction factor due to damping.** Factors for Japan Chile and Mexico are significantly smaller than the other ones.
- **Design spectra.** For the range of periods of interest for isolated buildings, spectra for Russia, Mexico and Japan have highest ordinates while spectra for Italy and new US code have lowest.

- **Load combinations.** Load combination for USA codes is quite demanding.
- **Maximum allowed reductions after time-history analysis.** Only USA and Chilean codes contain these limitations. In old USA range between 10% (for substructure) and 40% (for superstructure); in new code these limitations are more restrictive.
- **Reduction factor due to ductility.** This factor in Italy is 1/1.5 for serviceability conditions/ultimate limit state, in US code cannot exceed 2, and in Chile is always 2; in Russia, is 1. Chinese and Japanese codes do not consider this coefficient.
- **Drift limits.** These bounds must be judged with respect to corresponding demanding force; strictest requirements come from Russian code and least strict ones from old USA one.
- **Particular requirements.** Chilean and old USA codes oblige a deep review of any base isolation project; noticeably, requirements are slightly less strict in new US regulation.

Particular conclusions for the prototype sanitary building are:

- **Static vs. dynamic analyses.** In Japanese and Chilean codes, static and dynamic formulations are highly adjusted; maximum differences are observed in China, where dynamic analyses are extensively used. In most cases, using seven pairs of accelerograms has provided better results than using only three.
- **Superstructure.** Design forces are highest in Russian code and smallest in Italian one (for housing use). However, regarding design forces for same return period, differences are less exaggerated (highest demands correspond to China and Japan and lowest to Chile). Differences in required stiffness for drift limit verification are extremely important; value for Russia is more than ten times higher than the one for old USA code. In Italian code, differences between housing and sanitary use are significant, both in terms of design forces and drift limits.
- **Isolation system.** Highest requirements correspond to China; lowest ones to Chile and Japan. Highest and lowest displacements for same return period correspond to China and Japan.
- **Substructure.** Requirements are extremely unbalanced, being most demanding for China and least for Chile. After normalizing for same period, most demanding prescriptions are in old USA code, and least one in Chilean regulation.

6.2.2 *Damping modification factors*

This subsection proposes a strategy to derive damping modification factors after linear dynamic analyses by using artificial inputs that are generated to match the 5%-damped code design

response spectra of the area under consideration. Given the sensitivity of such modification factors to the significant (Trifunac) duration, the considered artificial inputs are generated caring that this duration does not exceed significantly the one of the available local records. This approach is intended for any country, region or city where a sufficient number of representative seismic records is not readily accessible. The derived expressions are aimed to modify 5%-damped displacement (or pseudo-acceleration) and acceleration response spectra according to the actual value of the damping ratio of the analyzed construction; values both higher and smaller than 5% are considered.

- The obtained expressions depend on the period; conversely, the influence of the soil type and the site seismicity is less relevant, being neglected in this study. The proposed modification factor for acceleration response spectra is greater than one for long periods and damping ratios higher than approximately 50%; this circumstance corroborates that, in buildings with base isolation, overdamping can lead to serious nonstructural damage.
- An application example of the proposed strategy to Colombia is presented; expressions for scaling displacement and acceleration spectra are obtained. Such expressions match reasonably well those of the most relevant previous studies. The derived expressions are compared with other major seismic regulations, being concluded that lie within their range; noticeably, such codes show important scattering. The results obtained with these artificial inputs are compared with those for some available historical accelerograms recorded in Colombia; the agreement is rather satisfactory, accounting that such actual inputs are too scarce to represent the actual seismicity. Concerning seismic isolation, the current Colombian code refers to the USA regulations; although there are relevant discrepancies with the results of this study, the differences are only moderate for the ranges of period and damping of interest. The suitability of the proposed formulation to the Colombian situation is further analyzed in a verification example of a base-isolated hospital building; comparison with results derived after time-history analyses using actual inputs shows better agreement than using the US regulations.
- The satisfactory performance of the derived expressions shows that artificial accelerograms can be a convenient option to estimate the damping modification factor in countries without enough actual seismic records, like Colombia.

6.2.3 *Proposal for future Latin American codes*

In this subsection the direct applicability of ASCE 7-16 in Latin American countries to design isolated structures is discussed. A new proposal about relevant aspects required in the design of this type of structures is developed, with the purpose to promote this technology for any kind of building in Latin America and especially in Colombia.

- The direct applications of ASCE 7-16 in Colombia and Latin America induce technical and economical inconsistencies.
- It is necessary make an adequate adaptation of ASCE 7-16 to local regulations (NSR-10 in Colombian case)
- The proposal here developed is coherent with NSR-10, and is a practical and understandable methodology that can promote this important construction technique in the country in a better way.
- It is very important that each Latin American country developed its own code for seismic isolation of buildings, to adapt it to its local conditions both economical and technics

6.2.4 *Scope of this study*

The conclusions of this research, specifically these of the chapter 5 can be broadly generalized to other Latin American countries. Among them, Perú, Ecuador, Bolivia, Argentina, etc.

6.3 **Future investigations**

Given the potential professional interest of this research for Colombia, from the obtained results, the following future subjects of researches are envisaged:

- **Drift limits for isolated structures:** Drift limits used in isolate structures must be carefully analyzed and research, because the performance expected in this type of buildings is different to conventional buildings (without isolation) whose drift limits has been clearly defined.
- **Forces reduction factor.** If the behavior of isolated structures must be near to elastic, a forces reduction factor near to 1 should be used. However international codes permit use factors values until of 2. These criteria require more research

- **Ductility requirements.** If the inelastic behavior of isolated structures is minimum, ductility requirements should be less than the used in conventional structures, whose has a high inelastic behavior. The uncertainty and distrust that currently still poses the technic could induce this consideration.
- **Pounding.** Pounding of seismic isolation system could induce higher forces on superstructure, which are not being considered in any kind of analysis or design procedure.
- **Isolated structures with different use groups.** Currently seismic isolation technique in American codes is developed target to essential buildings principally, however this technique can benefit considerably all the great number of normal use buildings
- **Performance check.** The performance of both seismic isolation system and the structure exposed to different types of earthquake must be careful analyzed. In the isolation system to guarantee its activation, its vertical load capacity (related to its lateral deformation capacity) and to avoid or control pounding. In the structure to guarantee elastic behavior or define possible inelastic incursions

The aforementioned researches are intended to be performed in the Salle University in Bogotá, Colombia.

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Appendix A. PUBLICATION GENERATED DURING THIS RESEARCH

This appendix lists the main publications generated during this research.

Publications in Proceedings of Conferences:

- López Almansa F., Piscal A. C. (2017). New design approach for base-isolation of essential sanitary facilities in high seismicity zones of Colombia. *16 World Conference of Earthquake Engineering, Santiago de Chile*.
- Piscal A. C., López Almansa F. (2017). Comparación de códigos de aislamiento sísmico a nivel mundial. Implicaciones para Colombia. *VIII Congreso Nacional de Ingeniería Sísmica, Barranquilla, Colombia*.
- Piscal A. C., López Almansa F. (2017). Herramienta computacional para el predimensionamiento de sistemas de aislación elastoméricos empleados en edificaciones. *VIII Congreso Nacional de Ingeniería Sísmica, Barranquilla, Colombia*.

Publications in Journals indexed by the Journal of Citation Reports:

- López Almansa F., Piscal A. C. (2018). Survey on major world-wide regulations on base isolation of buildings with rubber bearings. *Soil Dynamics and Earthquake Engineering*. En revisión.
- Piscal A. C., López Almansa F. (2018). Generating damping modification factors after artificial inputs in scenarios of local records scarcity. *Bulletin of Earthquake Engineering*. 2018. DOI: 1007/s10518-018-0406-9.

Other publications:

- Piscal A. C., López Almansa F. (2018). Propuesta para la futura norma de aislamiento sísmico en Colombia. *Revista Dyna, Universidad Nacional de Colombia*. En revisión.
- Piscal, C., López Almansa F. (2018). Aplicación de la nueva metodología de ASCE 7-16 para el diseño sísmico resistente de edificaciones con aislamiento de base en Colombia. *Revista Científica de Ingeniería y Desarrollo, Universidad del Norte, Colombia*. En revisión.
- Piscal A. C., López Almansa F. (2016). Consecuencias de la posible aplicación a Colombia de las normas más actuales sobre aislamiento sísmico de edificios. *Revista Internacional de Ingeniería de Estructuras*. ...