EVALUATION OF THE REGULATIONS USED IN COLOMBIA FOR THE SEISMIC VULNERABILITY ASSESMENT OF EXISTING CONCRETE STRUCTURES

A Dissertation Submitted in Partial Fulfilment of the Requirements
for the Master Degree in

Earthquake Engineering

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The dissertation entitled "Evaluation of the Regulations Used in Colombia for the Seismic Vulnerability Assessment of Existing Concrete Structures", by Jaime H. González, has been approved in partial fulfilment of the requirements for the Master Degree in Earthquake Engineering.

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ABSTRACT

Colombia is a quite high seismicity country, but only starting from 1984 seismic forces were taken into account in structures design. It clearly indicates that the study of the behaviour of existing structures and the development of an effective seismic assessment code are extremely important topics of investigation and should become urgent goals for Colombian government.

In this thesis, these investigations have brought to study several codes that present guidelines for the seismic vulnerability assessment of existing structures (FEMA356, Eurocode-8 and the New Zealand Recommendation) and compare them with the regulations of the Colombian standards (NSR-98). The outcome of this comparison exhibits the weakness of the Colombian code and addresses possible modifications that could be performed in order to update it.

Among the limitations found in the Colombian Code, the main one is related with the use of a global ductility factor that depends on the structural system and detailing. This factor is defined entirely by the engineer judgement instead of the use of analytical local ductility factors as the one used by the other codes studied.

Other aspect considered has to do with the minimum requirements imposed by the NSR-98 for the collection of the required information of the structure which are not clearly established. It is concluded that this feature must be improved including levels of necessary information taking into account the type of structure analyzed.

It was also found that the NSR-98 does not include guidelines for the performance of non-linear analysis. This contradicts some considerations made by the other codes studied which state that these types of analysis produce more accurate and reliable results and that they must be compulsory for certain type of structures. According to this, the Colombian entities in charge of the development of the NSR-98 must make big efforts in this topic in order to incorporate the minimum requirements for non-linear procedures (static and dynamic).

Keywords: existing concrete structures; codes; seismic vulnerability assessment; Colombia; FEMA 356; NSR-98; Eurocode-8; New Zealand Recommendations.

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1 INTRODUCTION

The development of new structural codes has made possible that the design of structures guarantee a better performance of the buildings when these are subjected to different kind of loads such as gravitational, seismic or wind forces. They include provisions that encourage the development of designs with features important for good performance, including regular configuration, structural continuity, ductile detailing and materials of materials of appropriate quality. Unfortunately, this can't be said about the structures that were designed before the implementation of these codes, especially when seismic forces are concerned.

This is the reason why big efforts have been carried out in order to understand the performance of existing buildings when these are subjected to seismic load, taking into account aspects such as material properties, structural system and configuration, foundation, site conditions, soil-structure interaction, quality of workmanship and structure's age among others [Otani, 2000]. The investigation made have permitted the development of different methodologies that allow the designer to estimate the most probable behaviour of an existing building when this is subjected to an earthquake but there is still important questions that haven't been solved. In addition, the results obtained from this effort haven't produced a unified method, being common that the use of one or other methodology produces different results.

This has major relevance in countries where the use of adequate seismic codes is recent. In Colombia, for example, the first seismic code was released in 1984, which implies that most of the structures existing in the country has been designed with poor, if any, seismic requirements. Furthermore, the most recent Colombian code (1998) specifies that the design of the structures built according to the requirement of the 1984 code, must be reviewed considering the updates present in the latter code. It says that only the structures designed and built after 1998 can be considered as earthquake resistant (according to Chavarría et al [2001] just the 20% of the Colombian structures can be considered in this group).

In Colombia, the national requirements for the seismic vulnerability assessment of existing structures are constantly evaluated in the country by professionals that considered that it requires an update according to the state of the art. The main objective of this work is born in this need.

This document studies different approaches used nowadays to asses the seismic vulnerability of existing RC building present in standards used worldwide such as FEMA356, Eurocode-8 and the New Zealand Recommendations and compares them with the Colombian standard.

An overview of the Colombian tectonic environment and the effect that recent earthquakes have had in the country was also considered, as useful to understand the consequences that not improve seismic specifications could have on the country. It was also included a brief description of the evolution of the Colombian seismic codes and the way they are applied today.

The outcome of this work shows the limitations that the Colombian standard for seismic vulnerability assessment contains and presents some recommendations that could improve the reliability of the results obtained, according to the principles used in the different codes studied.

2 COLOMBIAN SEISMICITY

2.1 Colombian Seismic Environment

Colombia is located in the north-west corner of South America and its tectonic enviroment is characterized by the interaction of the Nazca, Southamerican and Caribe plates. The figure 2.1 presents the location of Colombia and the borders of the plates mentioned above.

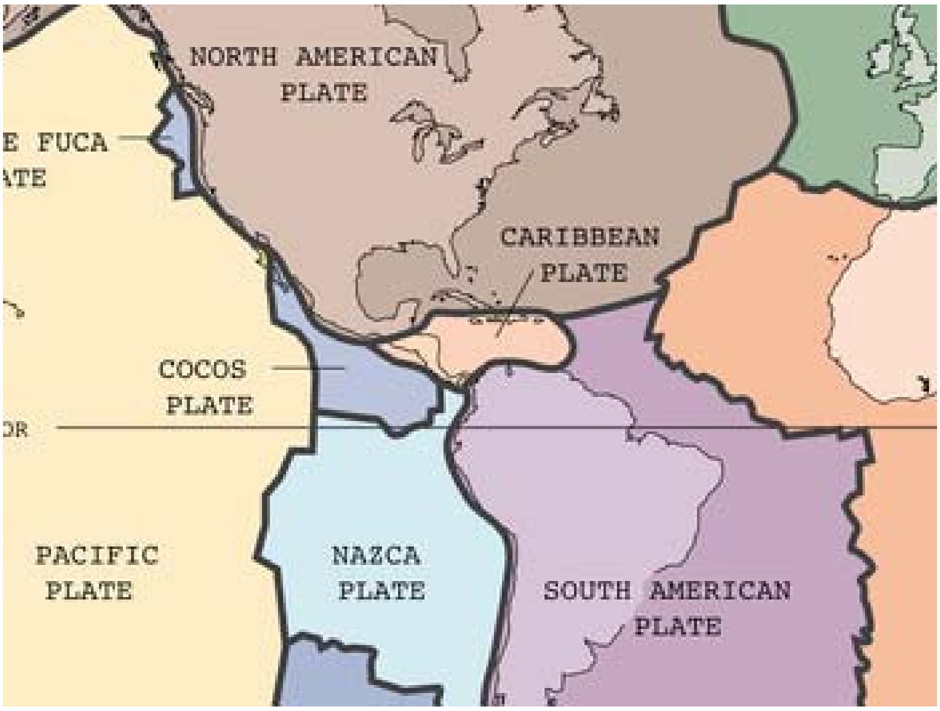


Figure 2.1. Tectonic plates in South America.

The relative displacement velocities of these plates can be described as high, intermediate and low respectively. The first one moves in west-east direction with a velocity that ranges between 60mm to 80 mm/year. In the other hand, the South American plate moves with an average velocity of 30mm/year in an opposite direction. The Caribbean plate, finally, moves with a slower velocity in south-east direction [Pujol et al., 1999].

This complex interaction creates several geological failures than can be considered as actives or likely actives, even if some of them haven't shown activity in past years. The following figure presents the 32 main Colombian failures:

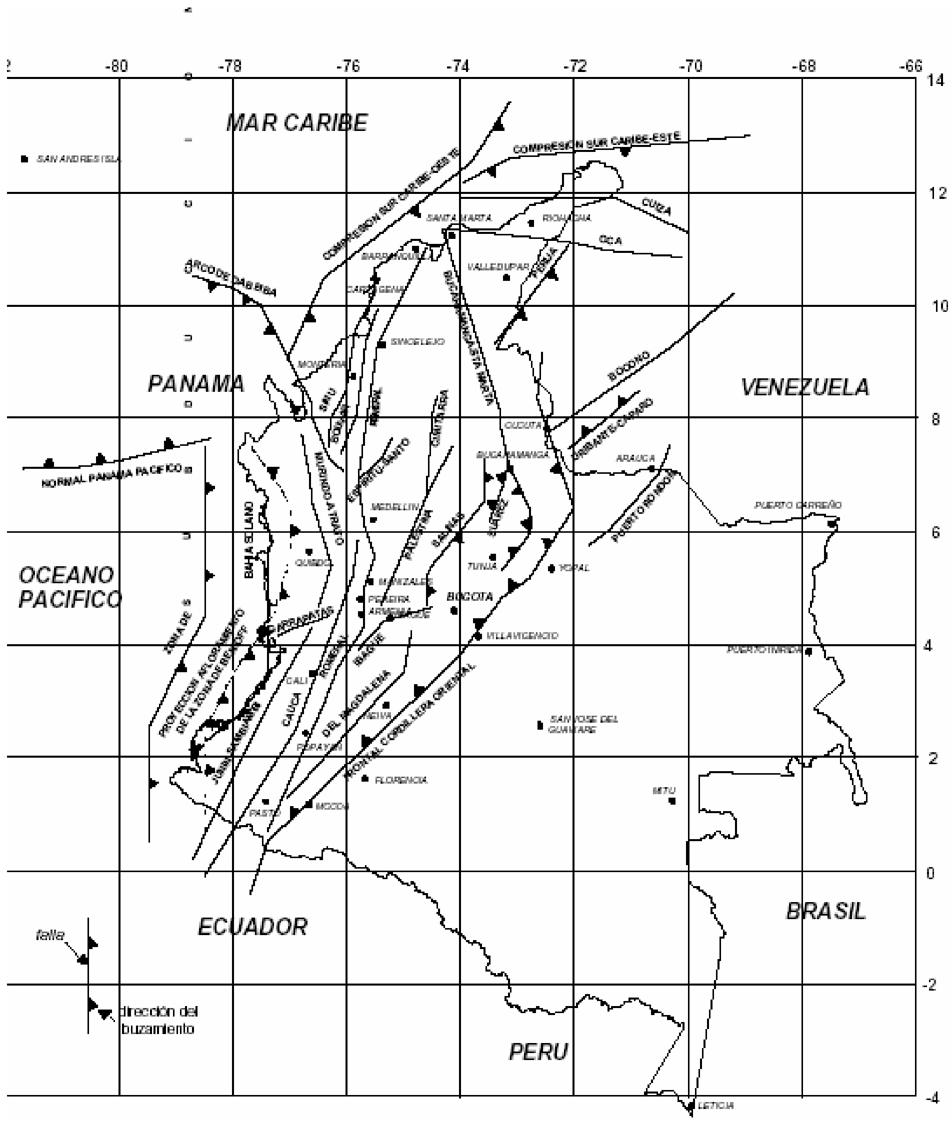


Figure 2.2. Colombian active geological faults

The figure 2.3 presents the epicentre of several seismic events occurred in Colombia and also includes the faults mentioned before. It is seen that the most active part corresponds to the called Andean region, where the direction of the faults is north-south which coincides with the alignment of the three mountain chains that cross the country [AIS, 1998]. In the east, the displacement environment is related with the Bocono Fault that could correspond to the border between the Caribbean and South American Plates not yet defined. In the left hand of this figure, it is possible to observe that the activity of the subduction zone presents a complex behavior product of the bending of the Nazca plate when it goes under the South American plate [AIS, 1998], with a greater concentration of events in the borders with Ecuador and Panama. Research has been carried out in order to clarify this pattern but there is not an accepted theory yet, even if the subduction zone is clearly defined. When the Benioff zone is considered, it has been found that the inclination of the subduction zone can vary with the latitude and it generates seismic events of important magnitude with a decreasing of this magnitude with the movement of the epicenter towards east with focal depths that can range from 40 to 700 km and maximum magnitudes of 8.6 [AIS, 1996].

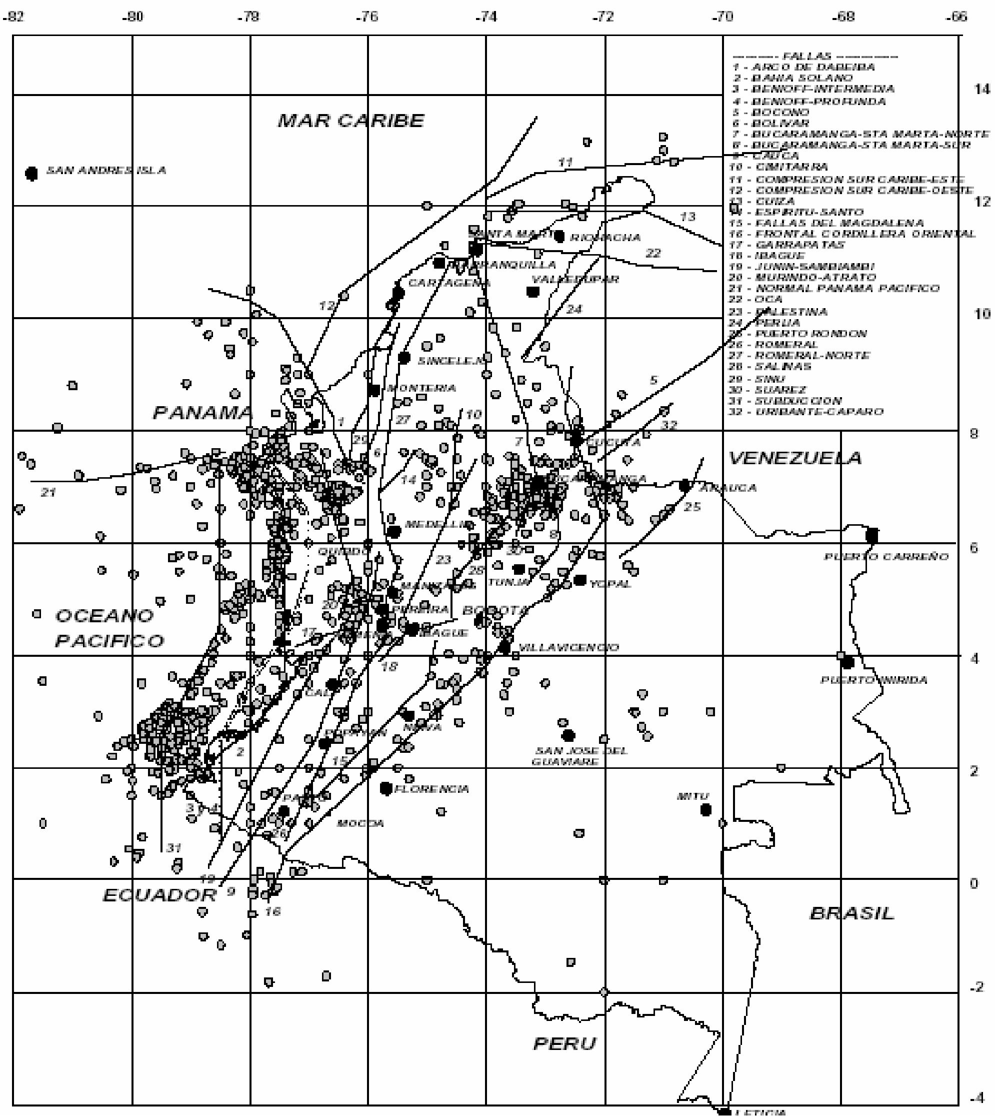


Figure 2.3. Epicentres of Colombian earthquakes Ms≥ 4 (1566-1995)

The most important active faults correspond to Romeral and Guaicaramo. The first one has a length of 1000 km with a width of 10 km in some parts where several parallel micro-faults of the main fault are found, with a possible higher seismicity rate in the south. The Popayan earthquake of 1983 (see 2.4.1) was produced by the rupture of one of these micro-faults. This fault doesn't produce high magnitude earthquakes but these earthquakes can be considered as shallow and because of its proximity from important urban centres (Pasto, Popayan, Pereira, Armenia, Manizales and Medellin) high intensities and important damages are expected [Bedoya, 2005].

The Guaicaramo fault, in the other hand, divides the Andean territory of the Country from the plane one called "*Llanos*". For some authors, this limit corresponds to the south-east border of the Caribbean plate. Three main events occurred in the past have been associated with this fault (1827, 1834 and 1967) with a highest magnitude of 7.0. This fault has been divided in three parts: the south one generates seismic events with high or moderate magnitude, the central one with high microseismic activity and just one historical event with moderate magnitude and finally, the north one with seismic events with a maximum magnitude of 6.0. Besides this geological fault system, the country presents and important feature that is called "Bucaramanga Seismic Nest" characterized by its unusual concentration of seismic activity, clustered within a small volume of 13x18x12 km, at a depth of 160 km [Pulido, 2003].

2.2 Seismic History

The first report of an earthquake occurred in Colombian territory is dated in 1566. It took place in the recent founded cities of Cali and Popayan, causing important damage. After this report, it is possible to find several attempts to recollect information about seismic events. Among the documents available, the most important are: The Diary by Don Luis Vargas Jurado (1703-1764), The Catalogue by Don Santiago Valencia (1785 – 1843), The chronicle by Don Jorge Maria Caballero (1813 – 1819) and the Seismic Chronology by Don Francisco Javier Vergara y Velasco (1898) [Baquero, 2003]. The information contained in these documents is considered an important source of data about relevant seismic events occurred in the Colombian territory. Furthermore, it has been the starting point of the most recent seismic hazard studies developed in the country. The next table contains the main seismic events occurred in Colombian territory and reported by the documents mentioned before [AIS, 1996]:

Table 2.1. Principal historic seismic events

DATE	LOCATION	DESCRIPTION
1566	Popayan and Cali	Important damages reported.
1736	Popayan	City partially destroyed. No reports about casualties or injuries.
1785	Felt in all the country	-------
1805	Honda (Region of Tolima)	More than 100 casualties reported.
1827	Bogotá	More than 250 casualties reported. Evidence of a big number of aftershocks.
1834	Bogotá	No reports about casualties or injuries. Felt in an extense area. Evidence of remarkable topographic changes.
1875	Cucutá	More than 460 casualties reported (approximately the 10% of the population of the city). City almost completely destroyed.
1906	Tumaco	Killed 500 to 1500. Generated a strong tsunami that was observed as far as Japan and San Francisco.

2.3 Instrumental Seismicity

In 1922, the first seismograph was installed in the country. After this, an important effort was carried out by the Priest Emilio Ramirez in order to install seismologic stations all across the country through the "Colombian Geophysical Institute of the Colombian Andes". This institute operated for several years but after the 1983 earthquake, the national government recognized the importance of having a modern national network of seismographs and it created the national seismological network of Colombia (RSNC). This was sponsored by several Colombian agencies as well as the Canadian Development agency. It started its operation in June 1993 and consist of 20 seismic stations countrywide connected by a satellite system [Pulido, 2003]. In addition to the RSNC, a seismological network in the South-west of Colombia (OSSO) operates since 1987.

Additionally, some institutions have carried out important investigations in order to complete the Colombian earthquake catalogue. This information has been used in the elaboration of the Colombian seismic hazard report and, more recently, the seismic microzonation of the most important Colombian cities.

2.4 Recent Colombian Eartquakes

2.4.1 Popayan Earthquake (March 31st, 1983)

The earthquake with a magnitude of 5.5 m_b occurred 40 km away of the city of Popayan with a focal depth of 10 kilometers. It produced 250 casualties and the estimated cost of the damages has been calculated as U$500 million. The 90% of the buildings of the historical quarter was destroyed and 2470 masonry houses, owned by low income families, were destroyed. Another 6900 suffered major structural damage.

2.4.2 Murindo Earthquake (October 18th, 1992)

A strong earthquake (M_w 7.1) hit the north-west of Colombia. Fortunately it took place in a low density population area and the damage to buildings and infrastructure was small [Pulido, 2003]. It triggered landslides and had some affectation in cities as far as Medellin (150 km away).

2.4.3 Paez Earthquake (June 6th, 1994)

It affected 35 towns in the regions of Cauca and Huila (south-western of the Colombian territory). There were 556 casualties, most of them natives. At least 5.276 houses were destroyed and 8.341 suffered some structural damage. It had a magnitude of 6.4 in Richter scale. It produced more than 3000 landslides that generating an avalanche that destroyed nearly 1664 houses along the river banks [Pulido, 2003].

2.4.4 Tauramena Earthquake (January 19th, 1995)

The Tauramena earthquake, M_w=6.5, occurred on January 19th, 1995, in the Andean Eastern Cordillera foothill region, the so-called Piedemonte Llanero. It caused minor structural damage in Bogota and 4 fatalities were reported.

2.4.5 Coffee Region Earthquakes

The departments of Caldas, Quindio and Risaralda, located in the central part of Colombia, are known as “Colombian growing coffee region” and contribute greatly to Colombia´s economy. These departments present the highest seismicity of the country and have been stroked by several important earthquakes. The following table presents the main events occurred in the region in the last years:

Table 2.2. Main seismic events in the “coffee region” in recent years

DATE	MAGNITUD	DEPTH (KM)	DESCRIPTION
25/11/1979	6.4	(---)	Caused important damage in the cities of Armenia, Manizales and Pereira [Espinosa, 1999]
08/02/1995	$6.4(m_b)$	90	970 edifications collapsed, 30 casualties and 6500 injured.
25/01/1999	$5.9(m_b)$	5-15	See description below

The 1999 earthquake produce devastation in the city of Armenia, but, as it is shown by Pujol *et al* [1999], the event affected 35 towns, caused more than 1200 casualties and injured 4000. The consequences of the catastrophe were devastating, mainly because of the loss of people's property and belongings: around 70% of Armenian houses vanished, and in some neighbouring towns the percentage was more than 80%. In the rural area, the proportion of lost houses was also 80%.

The maximum intensity reported was X (MMI) but an average value of IIX can be used in order to have an idea of the situation of the region after the earthquake [Espinosa, 1999] Preliminary estimates indicate that the reconstruction of the infrastructure of the cities affected cost more than 500 million dollars. This is about 0.5% of the 1996 gross national product of Colombia.

Most of the structural damage observed in concrete structures after the occurrence of this earthquake was produced by one of the following aspects: short columns[1] (see figure 2.4), lack of transverse reinforcement, interaction between structural and non-structural components and deficient detailing [Pujol et al, 1999]. There were many total collapses, mainly of buildings with poor construction built before 1984 and located on poor soils. Structures built using the 1984 code behaved better structurally, although damage to the infill masonry ranged from heavy to moderate [EERI, 1999]. It was also observed severe damage in unreinforced masonry walls that was the typical structural system used in the historical buildings.

[1] It was a common practice in Colombia the use of unreinforced masonry walls as partitioning system. The interaction between the walls and the structure was often ignored in the design.

Figure 2.4. Damage in building in the city of Armenia (Colombia)[2]

It also must be noticed that some buildings that should have withstand the earthquake without important damages, such as hospitals, police and fire stations, collapsed or had important structural affection.

2.4.6 Quetame Earthquake (May 24th of 2008)

This recent earthquake was felt in a large area of the Colombian territory. It produced 9 casualties, structural damage in the localities of Quetame and Puente Quetame, the nearest to the epicentre, which reconstruction has been evaluated in U$ 10 million.

At Bogota, it produced generalized panic and minor damage regarding non-structural components. After the event, many government and private institutions have shown their concerned about the possible damage scenario that the Colombian capital would present if it was stroke by an important earthquake.

[2] http://nisee.berkeley.edu/lessons/colombia.pdf.

3 COLOMBIAN SEISMIC STANDARD (NSR-98)

3.1 History of the Colombian Seismic codes

The Colombian Association of Earthquake Engineering (AIS) has been an important part during the development of the Colombian National standards for earthquake resistant constructions. In 1976, its first effort was related with the translation of the provisions of the Structural Engineers Association (SEOAC) with the aim of provide comprehensive information about how the seismic forces influence the reinforcement detailing.

In 1978, the ATC-3, that contained information about the state of the art of the Seismic resistant design, is known in Colombia and the evaluation of the document showed that it was possible to adapt it to the conditions of the country. But, before this process was performed, it was considered that the first step to carry out should be its divulgation and study.

While the Colombian engineers were getting close with the ATC-3, the AIS had several meetings with the American engineers that had developed the document. A proposal for the Colombian code of seismic resistant design (AIS-100 81 standard) that could be considered as a translation of the ATC-3 adapted to the Colombian regional conditions was obtained as result of these meetings. The use of this document wasn't compulsory at that time but numerous engineers in all the country decided to use it during the development of their design projects.

Due to the occurrence of the Popayan earthquake (see 2.4.1), it was clear that the scope of the AIS 100-81 should be increased, including a chapter for structural masonry and a simplified method for the design of buildings of one and two stories that had shown large damage during the earthquake. The response was the AIS 100-83 standard that also included the national seismic hazard map, produced by the National Development Department (DANE).

Another consequence of this earthquake was that the National Government delegated in the AIS, the elaboration of a technical code of seismic design that should be used in the Country and that would be based on the AIS 100-83, the *ICONTEC 2000* norm (based on the ACI 318-77) and the code of Steel Structures of *FEDESTRUCTURAS*. The final document should included a compendium of these norms and other subjects that were considered as relevant.

The final version was subjected to an evaluation carried out for several national organizations and universities and was finally accepted in June 7th of 1984 and called "*Codigo Colombiano de Construcciones Sismoresistentes*" (in the following CCCSR-84).

The aim of this code was focus in avoid some aspects that were observed during the earthquakes of 1979 and 1983. These can be resumed as:

- Damage and collapse of building with 5 stories or less. It should be noticed that it was a normal practice consider that this type of building shouldn't be designed to withstand seismic forces.
- Excessive flexibility of the buildings that produced damage to non-structural elements, due to the fact that most of the affected buildings weren't designed for seismic forces and didn't pass drift limits.
- Failure of columns due to lack of confining steel hoops and deficient design for lower shear forces.
- Failure and collapse of reinforced and non-reinforced masonry buildings.

The solution for other problems weren't included in the 1984 standard, because it was understood that was impossible to do it with a single document but it was also clear that future actualizations should be carried out in order to obtain a national standard with the adequate provisions. Some of the aspects that were postponed are:

- Change in the structural systems: Colombia had been characterized by the extended use of reinforced concrete frames as traditional structural system but they present excessive flexibility. For this reason, the standard should encourage the use of different systems such as structural walls.
- Limitation to irregularities: The CCCSR-84 aware engineers about the best performance of regular buildings but didn't include formal provisions about it or limitations for accidental torsion.
- Non-structural elements: The main objective of the CCCSR-84 was to improve the performance of the structural system because these elements have showed major damage during previous earthquakes and it was considered to postpone the addition of a chapter dedicated to non-structural elements.
- Different structural materials: The CCCSR-84 included provisions for concrete, steel and masonry structures because they were more common than other different materials as aluminium, timber, etc.

This standard was used in the country for thirteen years and its influence was more noticeable in regions of high seismicity, where the structural walls started to be more used in order to obtain the drift limits imposed by the code and, in general, an increase in the cost was perceived. In moderate seismic hazard regions, the effect wasn't felt as in high seismic hazard regions but the most important change was related with the use of the walls of the elevators as structural walls in order to obtain the drift limits. In the other hand, in low seismic hazard regions, some engineers felt that the new code was less strict than some provisions that they were using before. In addition to these changes, architects were affected because they had to start to use bigger columns and slab thicknesses.

Since the year 1992, the AIS had worked in the actualization of the code, with 8 sub-committees, formed by expert engineers and in 1997 the AIS 100-97 standard that corresponds to the technical content of the "Norma Colombiana de Diseño y Construcciones Sismo Resistente", in the following NSR-98, was approved in the National Congress on 19th august 1997 and it is of obligatory use in Colombia today.

In addition, some actualizations have been performed to the NSR-98 in order to take into account the advance in the state of the art in earthquake and structural engineering or perform corrections to the mistakes found by the AIS or by the engineers that have used the code since it was approved (1999, 2000 and 2002).

3.2 Content of the NSR-98

The NSR-98 presents a preface that includes a brief description of the code. In this preface it is expressed that the aim of the NSR-98 is focused in life protection although its use can diminished the probability of total or partial loss of the structure. It must be said that the NSR-98 doesn't include comments and it implies that the regulations can be interpreted in a personal way [Goméz, et al. 2005]. The NSR-98 is divided in eleven chapters that include a specific subject. Of these chapters, six correspond to updates of the CCCSR-84 and five are completely new. In this standard, the IS is used with exception of the chapter F (steel structures). Another difference with CCCSR-84 is related to the inclusion of three different levels of energy dissipation capacity in order to obtain a better inelastic performance of the structures. The standard restrains the use of each energy dissipation capacity level according to the seismic hazard region where the structure is going to be built, as can be seen in the following table:

Table 3.1. Allowed energy dissipation capacities according to seismic hazard regions

Energy Dissipation Capacity	Seismic Hazard Region		
	High	Intermediate	Low
Minimum (DMI)	Not Allowed	Not Allowed	Allowed
Moderated (DMO)	Not Allowed	Allowed	Allowed
Special (DES)	Allowed	Allowed	Allowed

According to the definition given by the code, the dissipation of energy is measured by deformation energy that the system or the element is capable to dissipate in consecutive hysteretic cycles. This dissipation energy is taken into account by the factors $\mathbf{R_0}$ provided by the standard that must divide the seismic forces used during the design stage. The dissipation energy is achieved, for each material, with the detailing level prescribed by the code according with the structural system used.

The basic energy dissipation factors (**R_0**) are presented for all the structural systems considered as well as for their combinations, for DMI, DMO or DES design. The factors that must affect these coefficients are also included in order to take into account height (ϕ_a) or plan (ϕ_p) irregularities (see Tables 3.4 and 3.5). The following table presents the values of **R_0** recommended by the NSR-98 for moment-resistant concrete frames:

Table 3.2. Factor R_0 for moment resisting concrete frames.

Earthquake and vertical loads resistant system	R_0 Factor	Seismic Hazard Region					
		High		Medium		Low	
		Use	Max. Height	Use	Max. Height	Use	Max. Height
Moment resisting concrete frames with special dissipation energy capacity (DES)	7.0	Yes	No limit	Yes	No limit	Yes	No limit
Moment resisting concrete frames with moderated dissipation energy capacity (DMO)	5.0	Not allowed		Yes	No limit	Yes	No limit
Moment resisting concrete frames with minimum dissipation energy capacity (DMI)	2.5	Not allowed		Not allowed		Yes	No limit

There are new structural systems included and it is also possible to find a detailed description of the way these structural systems can be combined in height or/and plan. Another important aspect included in the update of the standard is related to the drift limits because, as it was expressed above, the performance of existing structures during past earthquakes had showed that these presented excessive flexibility.

Table 3.3. Drift limits expressed as percentage of storey height.

Structural system:	Drift Limit
Reinforced concrete and steel frames, timber buildings.	1.0%
Structural masonry.	0.5%
Existing buildings (any structural system).	0.5%

The following are the chapters contained in NSR-98. It is only presented a description of Chapters A, B and C because the other ones are beyond the scope of this document:

CHAPTER A *"General Design and Construction Seismic Requirements"*: It is based on the ATC- 3 (1978) but recent codes were studied in order to update it with the information obtained during the earthquakes that have occurred since then. Some of the additional codes used are:

- ANSI/ASCE 7-95
- NEHRP-94
- UBC-97
- SEAOC-96
- EUROCODE 8
- AIJ-90 (Japan).
- NZS-4203 (New Zealand).

This chapter includes the seismic hazard maps and the regulation to obtain the seismic design spectrum with special attention to soft soils due to the performance observed during the 1985 Mexican earthquake. In addition, the requirements useful to perform the micro-zonation of the Colombian cities are outlined.

It is permitted to perform the evaluation of the seismic loads with the equivalent horizontal force method or with a dynamic analysis but further information regarding this subject will be presented in 3.3.3.

This chapter presents the A.10 section "Structures built before the release of the standard" that describes the provisions and the procedure that must be followed to carry out the seismic vulnerability assessment and structural reinforcement of an existing building. This procedure is described in detailed in 3.4.

CHAPTER B *"Loads"*: The CCCSR-84 was based on the ANSI A.58 (1982) that presented the load requirements for structures designed by allowable stress method. The NSR-98 has evolved from this to the ANSI/ASCE 7-95 (1996) that presents load requirements than can be used for structures designed by allowable stress method or ultimate limit state method.

CHAPTER C *"Structural Concrete"*: It is based on the ACI 318 (1978) but it has been updated taking into account the new versions of the American code that have been released (1983, 1989, 1995). The chapter presents the requirements for designing concrete structures according to the required energy dissipation capacity. At the moment, a process to update the NSR-98 taking into account the new specifications presented in the ACI 318 (2002) is being carried out.

The section C.21 of this chapter includes the minimum design requirements (member size, minimum and maximum reinforcement bar ratios for beams and columns, etc.) and the description of the detailing of the structural elements (minimum bar size for stirrups, stirrups spacing, etc.) in order to guarantee that the building will be able to withstand the seismic forces according to its energy dissipation capacity.

CHAPTER D *“Structural Masonry”*

CHAPTER E *“One and Two Stories Buildings”*

CHAPTER F *“Steel Structures”*

CHAPTER G *“Timber Structures”*

CHAPTER H *“Geotechnical Studies”*

CHAPTER I *“Technical Supervision”*

CHAPTER J *“Fire Protection Requirements”*

CHAPTER K *“Additional Requirements”*

3.3 Seismic design according to the NSR-98

3.3.1 General methodology

The NSR-98 presents the following steps in order to perform the seismic design of buildings:

- Definition of the seismic hazard level and the value of the peak ground acceleration (A_a) of the structure for the location of the structure (see 3.3.2).

- Definition of the seismic design spectrum taking into account factors such as local effects and importance coefficient (see 3.3.2).

- Definition of the energy dissipation capacity (DES, DMO and DMI) and factor $\mathbf{R_0}$ of the structure according to the seismic hazard region, the structural system and the used material (see 3.2).

- Definition of the structural irregularity of the building (see Tables 3.10 and 3.11). The factor $\mathbf{R_0}$ must be multiplied by the factors ϕ_a and ϕ_p given in these tables in order to find the factor **R** that must be used during the design stage.

- Structural analysis of the structure using some of the methods allowed by the NSR-98 (see 3.3.3).

- Evaluation of storey-drifts and comparison with the limits accepted by the NSR-98 (see Table 3.5).

- Design of the structural elements.

Table 3.4. Plane irregularities

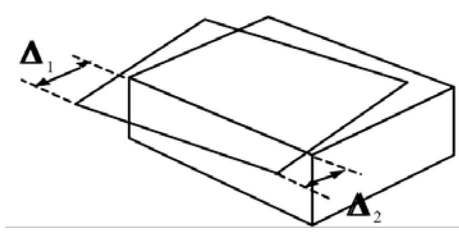
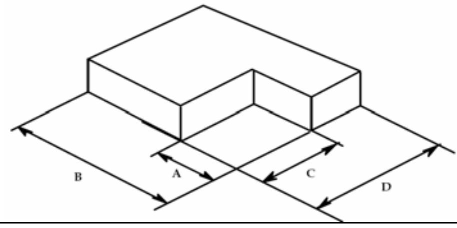
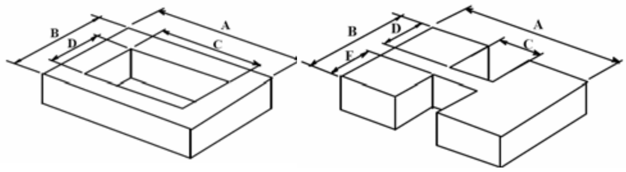
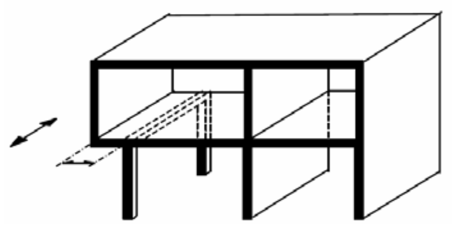
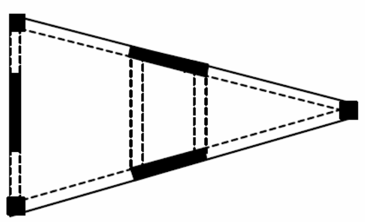
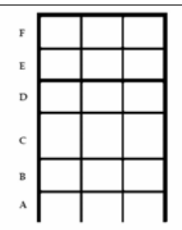
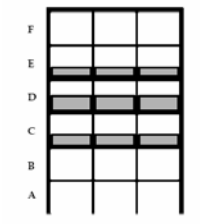
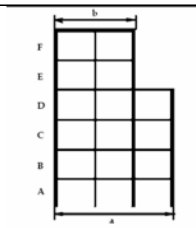
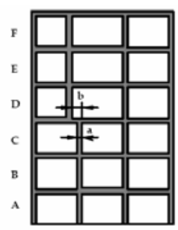
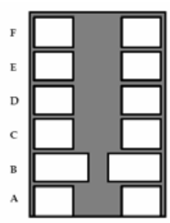
PLAN IRREGULARITY			
TYPE	**DESCRIPTION**	ϕ_p	**FIGURE**
TORSIONAL IRREGULARITY			
1P	$\Delta_1 > 1.2\frac{\Delta_1+\Delta_2}{2}$	0.9	
L-SHAPE IRREGULARITY			
2P	$A > 0.15B$ and $C > 0.15D$	0.9	
DIAPHRAGM IRREGULARITY			
3P	$CxD > 0.5AxB$ or $Cx(D+E) > 0.5AxB$	0.9	
VERTICAL ELEMENTS IRREGULARITY			
4P	The structure presents discontinuities in the seismic load paths due to the geometric displacement of vertical elements.	0.8	
NON-PARALLEL SYSTEMS IRREGULARITY			
5P	The orientations of the vertical elements of the lateral resistance system are not parallel or symmetrical with the main orthogonal horizontal axis.	0.9	

Table 3.5. Elevation irregularities

ELEVATION IRREGULARITY			
TYPE	**DESCRIPTION**	ϕ_p	**FIGURE**
FLEXIBLE STOREY IRREGULARITY			
1A	K_i : Stiffness storey i $K_C < 0.70K_D$ or $K_C < 0.80(K_D\ K_E\ K_F)/3$	0.9	
MASS DISTRIBUTION IRREGULARITY			
2A	m_i : mass storey i $m_D > 1.50m_E$ or $m_D > 1.50m_C$		
GEOMETRIC IRREGULARITY			
3A	a>1.30b	0.9	
VERTICAL ELEMENTS IRREGULARITY			
4A	b>a	0.8	
SOFT STOREY IRREGULARITY			
5A	Strength storey B < Strength storey C	0.8	

3.3.2 Seismic design spectrum

- **Seismic Hazard Level:** It must be defined according to the geographical location of the structure. This must be done taking into account the values of A_a (peak ground acceleration) included in the seismic region hazard maps of the NSR-98. Some cities have already the microzonation and the values of A_a given there should be used instead.

  The Figure 3.1 corresponds to the seismic hazard map of the NSR-98. In this map "*alta*" corresponds to a high seismic hazard, "*intermedia*" to intermediate and "*baja*" to low seismic hazard.

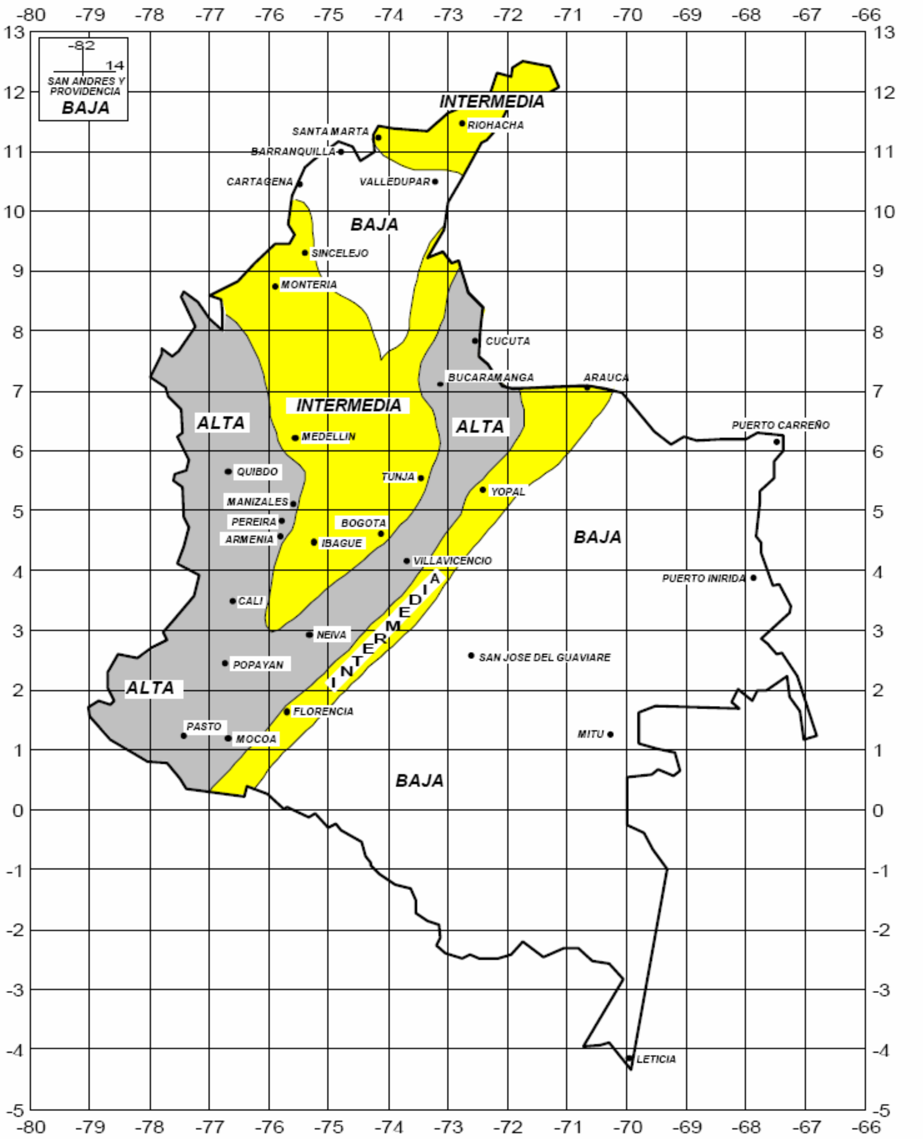


Figure 3.1. Colombian seismic hazard map

The Table 3.6 presents the values of the ranges of A_a for the seismic zones considered by the code. In the Table 3.7, it is possible to observe the values of A_a prescribed for the main Colombian cities.

Table 3.6. Values of Aa for the different seismic regions

Aa (g)	Seismic Zone
0.05-0.10	Low
0.10-0.20	Intermediate
>0.20	High

Table 3.7. Values of Aa for the main Colombian cities

City	Seismic Zone	Aa (g)
Armenia	High	0.25
Bogotá	Intermediate	0.20
Cali	High	0.25
Cartagena	Low	0.10
Medellin	Intermediate	0.20
Pasto	High	0.30
Popayan	High	0.25
Villavicencio	High	0.30

- **Local effects:** The site coefficient (S) must be selected taking into account the soil profile. The code presents four different types of soils and four different coefficients that range from 1.0 (for rock or hard soils) to 2.0 (for thick soft soil layers).

The following table presents the layers considered by the standard and their respective site coefficient:

Table 3.8. Description of soil profiles prescribed by NSR-98

Soil Profile	Description	Site Coefficient
S1	Rock with shear wave velocity larger than 750m/s or hard and dense soils with thickness smaller than 60 m with a shear wave velocity larger than 400 m/s	1.0
S2	Hard and dense soils with thickness larger than 60 m with a shear wave velocity larger than 400 m/s or intermediate density soils with a thickness smaller than 60 m with a shear wave velocity that ranges from 270 m/s to 400 m/s	1.2
S3	Intermediate or soft clay layers with thickness larger than 20 meters from the surface to the rock layer underneath with a shear wave velocity that ranges from 150 m/s to 250 m/s. In this profile the summation of the depths of the layers that contain soft clays must be smaller than 12 m.	1.5
S4	The summation of the depths of the layers that contain soft clays found from the surface to the rock layer underneath must be larger than 12 m with a shear wave velocity smaller than 150 m/s.	2.0

- **Importance Coefficient:** Considering the use of the structure, an importance coefficient (I) must be selected. The standard divides in four types the structures, with coefficients that range from 1.3 (for indispensable buildings) to 1.0 (for normal occupation buildings). The following table presents the type of use considered by the code and their respective importance coefficient:

Table 3.9. Description of type of use prescribed by NSR-98

Group	Description	Site Coefficient
IV	Indispensable buildings such as hospitals, telecommunication stations, power lines,	1.3
III	Community attention buildings such as firemen stations, army buildings, emergency car garages	1.2
II	Special occupation buildings such as schools, universities, shopping centres with more of 500 m^2 per floor	1.1
I	Normal occupation structures (not included in the other three groups).	1.0

- **Design Spectrum:** The shape of the elastic acceleration spectrum for a damping of 5% is given by the following equation:

$$S_a = \frac{1.2 A_a S I}{T} \tag{3.1}$$

Where T is the period of the structure

For structural periods lower than $T_c = 0.48S$, the value the next expression for S_a must be used instead:

$$S_a = 2.5 A_a I \tag{3.2}$$

For structural periods larger than T_L=2.4S, S_a can be computed as follows:

$$S_a = \frac{A_a I}{2} \tag{3.3}$$

For dynamic analysis, the value of S_a for periods lower than 0.3 s, can be computed as:

$$S_a = A_a I (1.0 + 5.0T) \tag{3.4}$$

The following graph presents the shape of the spectrum, where the dotted line corresponds to the spectrum that can be used when a dynamic analysis is carried out:

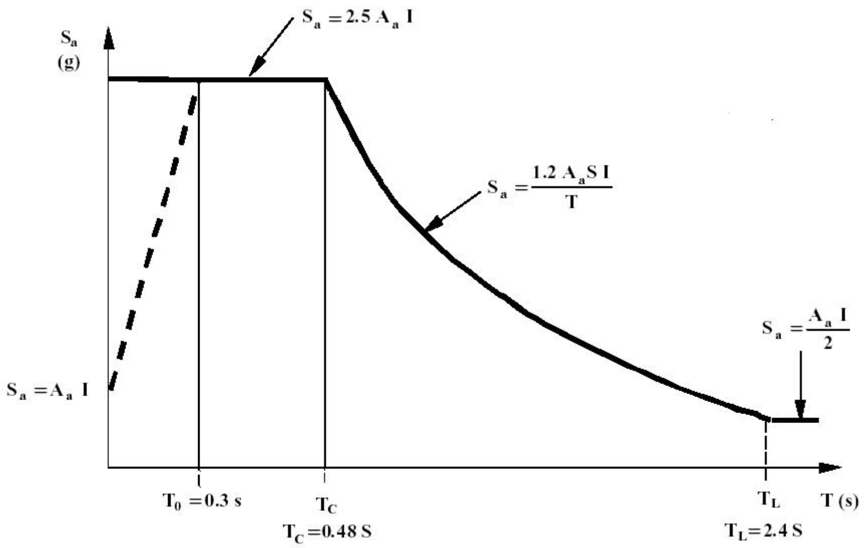


Figure 3.2. Elastic Design Spectrum (NSR-98).

3.3.3 Analysis Methods

The NSR-98 allows the use of the following analysis methods:

<u>Horizontal equivalent force:</u> It can be used in the design of the following structures:

- Regular or irregular buildings of any height located in areas with low seismic hazard level.
- Regular or irregular buildings of use I of any height located in areas with intermediate seismic hazard level.
- Regular buildings, located in areas with any seismic hazard level with height lower than 20 stories or 60 m, with the exception of the buildings located in a S4 soil profile with a structural period larger than 0.7 seconds.
- Irregular building with height lower than 6 stories or 18 m.
- Regular flexible structures supported by stiffer ones.

According to this method, the seismic base shear must be computed as:

$$V_s = S_a Mg \tag{3.5}$$

Where

M: Total mass of the structure. It must included the mass of elements such as partition walls, permanent equipment, tanks, etc. For storage buildings, it also must considered a 25% of the live load.

g: gravity acceleration (9.8 m/s^2)

The period of vibration of the structure is computed according to the following methods:

- Empiric: It is done using the following expression:

$$T_a = C_t h_n^{0.75} \tag{3.6}$$

Where

h_n : Height of the building, in meters, from its bottom to the top of the highest storey (n-storey).

The following values of C_t might be used:

- Concrete moment resisting frames and structural steel frames with eccentric diagonals: $C_t = 0.08$.
- Steel moment resisting frames: $C_t = 0.09$.
- A different structural system from the ones mentioned above: $C_t = 0.05$.

- Approximate Rayleigh-Ritz method: The value found by this method can't be larger than 1.2 times the one calculated with the expression above (equation 3.6).

When the seismic base shear is computed, the seismic forces must be distributed as follows[3]:

$$F_x = C_{vx} V_s \tag{3.7}$$

$$C_{vx} = \frac{m_x h_x^k}{\sum_{i=1}^{n} m_i h_i^k} \tag{3.8}$$

Where

F_x : Seismic force acting on storey x.

C_{vx}: Force distribution coefficient for the storey x.

m_x, m_i : part of M located in the x-storey or in the i-storey.

h_x, h_x: Height of the masses m_x, m_i, above the level of application of the seismic action.

The value of *k* is related with the fundamental vibration period as follows:

a.) For T lower or equal to 0.5 seconds, k=1.0.

[3] Fundamental mode shape is approximated by horizontal displacements increase linearly along the height.

b.) For T ranging from 0.5 to 2.5, k= 0.75 + 0.5T.

c.) For T larger than 2.5 seconds, k=2.0.

There are not limitations about the use of 3D or 2D model when this method is used but the following recommendations must be taking into account, among others:

- Support conditions.
- Effects of diaphragm flexibility.
- Change in the axial load of the elements due to overturning moments.
- Torsional effects.

The linear dynamic method (with response spectrum and modal superposition) must be used for:

- Building with height higher than 20 stories or 60 m. It excepts buildings in low seismic hazard region and buildings of Use I in intermediate seismic hazard regions which analysis can be performed with the horizontal equivalent force method, as it was expressed before.
- Buildings with irregularities 1A, 2A and 3A (see Table 3.5).
- Buildings with irregularities not described in the code.
- Buildings with height higher than 5 stories or 20 m located in high seismic hazard level zones.
- Regular or irregular buildings, located in a S4 soil profile with a structural period larger than 0.7 seconds.

A detailed analysis of the spatial distribution of the mass and stiffness must be performed, including the characteristic that will influence the dynamic performance of the structure. The following models can be used:

- 3D-Model with stiff diaphragm: The mass is considered located in the center of the diaphragm (infinitely stiff) with adequate modification in its location in order to take into accounts the torsional effects. The code recommends that this method must be used for structures with 1P, 4P or 5P (see Table 3.4 for definitions) or when the designer considers that it is necessary.
- 3D-Model with flexible diaphragm: The mass is considered in each joint of the structure and this can move and rotate in any horizontal or vertical component. This method must be used when there is a lack of diaphragm, when irregularities 2P or 3P (see Table 3.4 for definitions) are present when the designer considers that it is necessary.

- 2D Models: These can be used when the structure characteristics are not included in the ones of the previous models. In this model the movement is constrained to one horizontal direction. The torsional effects must be computed and added to the ones obtained in the plane as well as the directional ones.

Inelastic Dynamic Analysis: The NSR-98 only states that a inelastic dynamic analysis must be performed when, according to the engineer criteria, there might be significant variations in the dissipation energy capacity of the structure only identifiable through this method. The standard does not provide further information about the manner this analysis must be carried out.

Alternative methods: The NSR-98 allows the use of these methods but it states that they must have general acceptation for engineers, include dynamic characteristics of the structure and the inelastic properties of the used materials. The standard does not present additional information, descriptions or examples of the methods considered as alternatives.

3.4 Seismic Vulnerability Assessment Procedure

The methodology proposed for the seismic vulnerability assessment of the structures that were built before the released of the NSR-98, are presented in the chapter A.10 and includes the criteria that must be used in order to assess, add, modify or rebuilt the structural system of existing structures.

It is understood that the main goal of the methodology proposed is to preserve human lives and achieve an acceptable performance of these structures when they are subjected to seismic loads. This methodology is based on the comparison between the demands and strength of the elements of the structure.

The chapter presents a series of steps that must be followed and that are presented and detailed here:

3.4.1 Step 1

In this part, it must be verified that the structure and the aim of the intervention is covered for the scope of the section A.10. This is, as was explained before, that the structure was built before of the release of the code (1998).

In this part, it is also expressed that minor interventions that don’t affect the structural system of the building don’t require a seismic vulnerability assessment unless they are made in order to change the use of the structure (change from normal occupation to special occupation for example, see 3.3.2 for definitions).

It must be said that the standard is not clear about which structures should be evaluated but in the country studies of seismic vulnerability assessment and design of structural reinforcement have been carried out for buildings of type of use IV to II.

3.4.2 Step 2

This step is related to the research of existent information about the design and construction process of the structure. In this part, there are also some requirements about the structural survey that must be carried out. When drawings and structural design reports of the building can be found, this information must be verified with the actual configuration of the building. This also implies that information about modifications performed after the construction of the building must be considered.

The survey must take into account all the required information, as local failures, excessive deflections, corrosion of the rebars, etc., to give a judgement of the quality of the original design and construction as well as the present condition of the building, as it is expressed in Step 3.

3.4.3 Step 3

A judgment about the quality of the design, the construction process and the condition of the structure must be given based on the information collected in the previous step.

The standard emphasizes that even though this opinion depends on the engineer criterion, aspects as the irregular distribution of the mass and stiffness or lack of diaphragms, among others, must be taken into account. At the end of the step, the structure is classified as in good, regular or bad condition according to the design and construction process as well as for the situation of the structure.

3.4.4 Step 4

Evaluation of the seismic loads is performed according to the lateral horizontal method or a dynamic analysis described in more detail in other chapters of the code. In this stage, the loads different from the seismic ones must be computed.

The seismic forces obtained are affected by the energy dissipation factor ($\mathbf{R_0}$) that must be selected by the engineer on the base of the information obtained in step 2 due to the fact that the standard only provides the value of these coefficients for new structures. In any case, because the determination of this coefficient represents a key factor during the development of a seismic vulnerability assessment of an existing structure, the chapter gives some indications about the way its value can be estimated:

- When there is good information about the original design of the structure, it is allowed the use of a value of $\mathbf{R_0}$ obtained from the comparison of the values given by the NSR-98.

- When there is partial fulfilment of the requirements that the standard prescribes for a new structure with the same material and structural system, the value of $\mathbf{R_0}$ can be estimated by interpolation of the data provided by the code. This must be done taking into account the required dissipation energy capacity according to the seismic hazard region and the actual capacity of the structure, deduced from the information gathered about the design and detailing of the existing building.

- When the information gathered about the original design is considered as incomplete or not satisfactory the engineer must use his judgement in order to estimate the best value of $\mathbf{R_0}$ for the structure, but this value can't be higher that the one prescribed by the standard for a new structure with similar characteristics.

- When there is not information about the original design, the standard allows the use of a value of $\mathbf{R_0}$ equal to the 75% of the one prescribed by the NSR-98 for a new structure with similar characteristics.

- It is not required the use of factor **R** lower than one. For unreinforced masonry buildings, it is compulsory the use of **R** equal to one.

It must be noticed that the selection of the factor **R** is defined entirely by the engineer judgement based on the available information of the building.

3.4.5 Step 5

All the loads calculated must be combined according to the specifications given in the code and an elastic analysis of the structure must be carried out in order to obtain the internal forces that the structure must withstand.

It must be said that the standard just prescribes a non-linear analysis for the assessment of indispensable structures. The NSR-98 does not state if the analysis must be static or dynamic but the outcome must identify the main failure mode.

3.4.6 Step 6

The strength (N_{ex}) of the structural elements (axial force, bending moment, shear force) is evaluated taking into account the information obtained in the step 2. Strength is defined as the level of stress that implies a non-elastic response of a given element, fracture of fragile materials or yielding of ductile ones. The code emphasizes that the experience and criterion of the engineer has a major role in this stage. In general, the existent strength corresponds to the values that are obtained using the strength models that the standard prescribes for each material.

3.4.7 Step 7

The value of strength previously obtained must be affected by the coefficients ϕ_c (a factor related to quality of design and construction) and ϕ_e (a factor related to condition of the structure) that depend on the evaluation performed on step 3 in order to obtain the effective strength of the elements (N_{ef}). The values proposed by the standard for these factors are presented in the Table 3.10.

Taking this into account, the effective strength must be computed as:

$$N_{ef} = \phi_c \phi_e N_{ex} \tag{3.9}$$

Table 3.10. Values of ϕc and ϕe

	Quality of design and construction or condition of the structure		
	Good	**Regular**	**Bad**
ϕ_c or ϕ_e	0.9	0.8	0.7

It must be noticed that the highest value of strength that can be assigned to an element of an existing structure corresponds to the 81% of the strength that the same element would have in a new structure.

3.4.8 Step 8

The over-stress index must be computed as the ratio between the internal forces obtained (step 5) and the strength of a given element (step 7). It must be said that the over-stress index of a given element corresponds to the maximum obtained for each solicitation considered (axial force, shear force, bending moment, etc.)

3.4.9 Step 9

The horizontal and vertical displacements of the structure when this is subjected to the loads computed in step 4 and combined according to step 5, must be obtained.

3.4.10 Step 10

In order to obtain the horizontal flexibility index, the maximum horizontal displacement obtained in step 9 must be compared with the maximum allowed by the code (Table 3.3). The same procedure must be carried out for vertical displacements in order to obtain the flexibility index in that direction.

3.4.11 Step 11

Global over-strength and a flexibility index must be computed as the maximum obtained for the structure. If these values are lower than one, the structure doesn't require intervention. If they are higher than one a structural reinforcement must be considered and a new analysis of the reinforced structure considering must be performed.

4 FEMA 356

The FEMA 356 presents provisions for the rehabilitation of buildings in order to improve their seismic performance. According to the standard, it is expected that most buildings rehabilitated taking into account the requirements content in it, would perform within the expected objectives.

The standard is based on performance-based design methodologies, according to which a building rehabilitated with FEMA 356 will be able to satisfy a variety of performance levels considering the different levels of seismic input.

A seismic assessment performed based on FEMA 356 must be carried out taking into account the type of analysis procedure used (see 4.5), the type of structural component (see 4.3.1) and the target building performance level chosen (see 4.1) among other factors [Olarte et al., 2005]. The methodology, as it will seen in this chapter, includes the use of four different analysis procedures and the selection of the most appropriate for a given structure depends on the non-linearity and irregularity level of the building as well as the affectation that higher mode effects can have on its performance to withstand seismic actions.

4.1 Target building performance levels

The first step of the methodology proposed by FEMA 356 consists in the selection of a rehabilitation objective (see 4.2). Some definitions are needed in order to do that.

First of all, the terminology used by the standard for “target building performance levels” is intended to represent goals of design. It is related with the damage of the vertical and horizontal lateral-force-resisting elements for structural performance and with architectural, mechanical and electrical components as far as non-structural elements is concerned.

The targets presented in FEMA 356 are discrete damage states chosen from an infinite spectrum of possible damages that a given building can present when it is subjected to a seismic event.

The definitions of the discrete performance levels are done taking into account the presence of identifiable consequences found in buildings stroked by earthquakes (damage to structural and non-structural elements, permanent horizontal displacements, etc.). It is assumed that an existing building, retrofitted according to the provisions given by the standard, will present a post-earthquake configuration similar to the ones described by the performance level but, as it is expressed in the standard, it is impossible to guarantee that the building will perform exactly in that way.

4.1.1 Structural performance levels and ranges

The standard includes four discrete structural performance levels (S-1, S-3, S-4 and S-6) and two intermediate structural performance ranges[4] (S-2 and S-4). The FEMA-356 only presents the acceptance criteria for the structural performance levels but the acceptance criteria for the S-2 performance range shall be obtained by interpolating the acceptance criteria provided for the S-1 and S-3 performance level. In the other hand, acceptance criteria for S-4 performance range shall be obtained by interpolating the acceptance criteria provided for the S-3 and S-5 performance levels.

- **IMMEDIATE OCCUPANCY STRUCTURAL PERFORMANCE LEVEL (S-1).** The post-earthquake configuration of the building allows to its occupation because it retains the pre-earthquake strength, stiffness and configuration of the structure presenting very little damage.

- **DAMAGE CONTROL STRUCTURAL PERFORMANCE RANGE (S-2).** It can be understood as the continuous range of damage states between the Life Safety Structural Performance Level (S-3) and the Immediate Occupancy Structural Performance Level (S-1). Its use is recommended when some undesirables consequences that could be prevent with the S-1 can be accepted in order to reduce the cost of design.

- **LIFE SAFETY STRUCTURAL PERFORMANCE LEVEL (S-3).** The post-earthquake configuration presents damage to structural components but the structure is still far from collapse. Some human injuries can be presented but it is expected a low risk of death. The repairing of the structural should be possible but it may not be practical.

- **LIMITED SAFETY STRUCTURAL RANGE (S-4).** It can be understood as the continuous range of damage states between the Life Safety Structural Performance Level (S-3) and the Collapse Prevention Structural Performance Level (S-5).

- **COLLAPSE PREVENTION STRUCTURAL PERFORMANCE LEVEL (S-5).** In this level, the structure is considered to be at the limit of total or partial collapse. Significant degradation in strength and stiffness is observed in the lateral-force-resisting system with large and permanent lateral deformations. The vertical-load

[4] The inclusion of the structural performance ranges is done in order to allow customizing their building rehabilitation objectives according with the user requirements.

carrying system must continue carrying the gravity loads but with significant degradation.

- **STRUCTURAL PERFORMANCE NOT CONSIDERED (S-6).** This is considered when the rehabilitation of the structure includes only non-structural vulnerabilities.

The following table, obtained from FEMA-356, presents the expected damage for the levels S-1, S-2 and S-3 for vertical members of concrete frames. It must be said that the drift values presented in the Table 4.1 correspond to an estimate of the post-earthquake drift and are not the drift limits that must be used during design stage.

Table 4.1. Structural performance levels and damage for vertical elements

		Structural Performance Levels		
Structural system	**Element**	Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns	Extensive damage to beams. Spalling of cover and shear cracking (<1/8" width) for ductile columns. Minor spalling for nonductile columns. Severe damage in short columns.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints<1/6" width.
	Drift	4% transient or permanent.	2% transient; 1% permanent	1% transient; negligible permanent

The following table, obtained also from FEMA-356, present the expected damage for the levels S-1, S-2 and S-3 for horizontal members of concrete frames for three different types of diaphragms: metal deck, concrete and precast.

Table 4.2. Structural performance levels and damage for diaphragms

Elements	**Structural Performance Levels** Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Metal Deck diaphragms	Large distortion buckling of some units and tearing of many welds and seam attachments.	Some localized failure of welded connections of deck to framing and between panels. Minor local buckling of deck.	Connections between deck units and framing intact. Minor distortions.
Concrete Diaphragms	Extensive crushing and observable offset across many cracks.	Extensive cracking (<1/4" width). Local crushing and spalling.	Distributed hairline cracking. Some minor cracks of larger size (<1/8"width).
Precast Diaphragms	Connections between units fail. Units shift relative to each other. Crushing and spalling at joints.	Extensive cracking (<1/4" width). Local crushing and spalling.	Some minor cracking along joints.

4.1.2 Non-structural performance levels

These performance levels shall be selected from five categories (N-A to N-E) intended for architectural partitions, exterior cladding and ceilings; and mechanical and electrical systems.

- **OPERATIONAL NONSTRUCTURAL PERFORMANCE LEVEL (N-A):** In this level, the non-structural components must be able to function as they did before a seismic event.

- **INMEDIATE OCCUPANCY NONSTRUCTURAL PERFORMANCE LEVEL (N-B):** In this stage, there is some damage to some non-structural components but systems as doors, stairways, elevators, emergency lightening, etc., remain operable. Assuming that the structure is safe, occupants could stay in the building.

- **LIFE SAFETY NONSTRUCTURAL PERFORMANCE LEVEL (N-C):** The damage produced in the non-structural components is not-life threatening although some injuries may occur.

- **HAZARDS REDUCED NONSTRUCTURAL PERFORMANCE LEVEL (N-D):** The post-earthquake state includes large damage to non-structural components that could create falling hazards but high hazard non-structural components are secured.

- **NONSTRUCTURAL PERFORMANCE NOT CONSIDERED (N-E):** The building rehabilitation does not include non-structural components.

4.1.3 Designation of target building performance levels

In order to understand in a better way the feasible rehabilitation objectives, the FEMA 356 presents an alphanumerically designation for the target building performance levels that includes the number and the letter of the structural and non-structural performance levels (e.g., 1-B, 3-C). The Table 4.3, taken from the standard, presents the possible combinations of "target performance levels". The Table 4.4, on the other hand, gives a brief description of the behaviour expected for the most common target building performance levels and ranges.

Table 4.3. Target building performance levels and ranges for non-structural components

	Structural Performance Levels and Ranges					
Nonstructural Performance Levels	Immediate Occupancy S-1	S-2 Damage Control Range	S-3 Life Safety	S-4 Limited Safety Range	S-5 Collapse Prevention	S-6 Not Considered
N-A Operational	Operational 1-A	2-A	Not recommended	Not recommended	Not recommended	Not recommended
N-B Immediate Occupancy	Immediate Occupancy 1-B	2B	3-B	Not recommended	Not recommended	Not recommended
N-C Life Safety	1-C	2-C	Life Safety 3-C	4-C	5-C	6-C
N-D Hazards Reduced	Not recommended	2-D	3-D	4-D	5-D	6-D
N-E Not Considered	Not recommended	Not recommended	Not recommended	4-E	Collapse prevention 5-E	No rehabilitation

Table 4.4. Expected post-earthquake damage state

HIGHER PERFORMANCE LESS LOSS

Operational (1-A)	Immediate Occupancy (1-B)	Life Safety (3-C)	Collapse Prevention (5-E)
Backup utility services maintain functions; very little damage.(S1+NA)	The building remains safe to occupy; any repairs are minor.(S1+NB)	Structure remains stable and has significant reserve capacity; hazardous nonstructural damage is controlled. (S3+NC)	The building remains standing, but only barely; any other damage or loss is acceptable. (S5+NE)

LOWER PERFORMANCE MORE LOSS

4.2 Selection of a Rehabilitation Objective

A rehabilitation objective consists of a target building performance level and an earthquake hazard level and shall include one or more rehabilitation goals that can be chosen taking into account basic, enhanced or limited objectives.

The selection of a rehabilitation objective, as shown in the FEMA commentary [FEMA, 2000], determinates the cost and feasibility of any rehabilitation project, but unfortunately the standard doesn't present recommendations regarding the selection of the rehabilitation objective required for a given building.

The following table presents the range of rehabilitation objectives proposed by FEMA 356.

Table 4.5. Rehabilitation objectives

		Target Building Performance Levels			
		Operational Performance Level (1- A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (10%/50 year)	i	j	k	l
	BSE-2 (2%/50 year)	m	n	o	p

The following table presents an explanation of the Table 4.5, considering that each cell represents a discrete rehabilitation objective.

The definition of the rehabilitation objectives include in the standard is presented after Table 4.6.

Table 4.6. Definition of rehabilitation objectives according to Table 4.5

“k”+ “p” =	Basic Safety Objective (BSO)
“k”+ “p”+ any of “a”, “e”, “i”, “ b”, “f”, “j”, or “n” =	Enhanced Objectives
“o” alone or “n” alone or “m” alone =	Enhanced Objectives
“k” alone or “p” alone =	Limited Objectives
“c”, “g”, “d”, “h”, “l” =	Limited Objectives

4.2.1 Basic Safety Objective (BSO)

The BSO includes achievement of life safety and collapse prevention performance for earthquakes with 500 and 2500 years of return period respectively. According to the comments of the standard, when this rehabilitation objective is achieved, the structures are expected to suffer little damage from relatively frequent, moderate earthquakes but significantly more damage and potential economic loss from the most severe and infrequent earthquakes.

4.2.2 Enhanced Rehabilitation Objectives

When this objective is selected, the structures retrofitted present a better performance than the ones retrofitted according to a BSO. It can be achieved designing for a higher target building performance or by using higher earthquake hazard levels.

4.2.3 Limited Rehabilitation Objectives

When this objective is selected, the structures retrofitted present a poorer performance than ones retrofitted according to a BSO. It includes “reduced rehabilitation objective” and “partial rehabilitation objective”. The first one is understood as the rehabilitation performed to the entire building using a lower earthquake hazard level or target building objective than the BSO. In the other hand, the partial rehabilitation objective is obtained when a portion of the lateral-force-resisting system is rehabilitated.

4.3 Collection of as-built information

It is required information about the structural configuration, strength, condition, detailing, etc. of the building. They can be obtained from structural drawings, specifications and technical documents but must be verified with on-site investigations. This information must also include data for all non-structural elements that could affect the forces and deformations experienced by the structural elements during their response to an earthquake ground motion.

The following table gives some regulation about the data collection requirements according to the rehabilitation objective intended. This table includes the value of the knowledge factor κ that accounts for uncertainty in the collection of as-built data and, as it is explained on the commentaries of the standard, it is used to express the confidence with which the properties of the building components are known. The lower the knowledge level, the more conservative the applied assessment procedure should be [Mihaylov, 2006].

Table 4.7. Data collection requirements

Data	**Level of Knowledge**							
	Minimum		**Usual**				**Comprehensive**	
Rehabilitation Objectives	BSO or Lower		BSO or Lower		Enhanced		Enhanced	
Analysis Procedures	LSP, LDP		All		All		All	
Testings	No tests		Usual Testing		Usual Testing		Comprehensive Testing	
Drawings	Design drawings	Or equivalent	Design drawings	Or equivalent	Design drawings	Or equivale nt	Construc-tion document	Or equivalent
Condition Assessment	Visual	Compre-hensive	Visual	Compre-hensive	Visual	Compre-hensive	Visual	Compre-hensive
Material Properties	From Drawings or Default values	From Default values	From Drawings and Tests	From Usual Tests	From Drawings and Tests	From Usual Tests	From Document and Tests	From Compre-hensive Tests
Knowledge Factors (κ)	0.75	0.75	1.00	1.00	0.75	0.75	1.00	1.00

The standard gives a brief description of the main information required:

4.3.1 Building Configuration

It shall include the structural configuration of the gravity and lateral-load-resisting systems. The structural components must be divided in primary or secondary taken into account if the elements are able or not to provide capacity to the structure to resist collapse under seismic forces.

4.3.2 Component Properties

The information obtained must be enough to compute strengths and deformation capacities of the elements. The knowledge factor κ is considered in order to take into account the uncertainty associated with the as-built information gathered.

4.3.3 Site characterization and geotechnical information

Data on foundation configuration and soil surface and subsurface conditions must be obtained. It can be gathered from original construction information but for enhanced performance objectives a subsurface investigation must be carried out.

4.3.4 Adjacent buildings

The interaction with adjacent buildings must be studied. This includes building pounding, shared element conditions and hazards from adjacent buildings such as falling debris, aggressive chemical leakage, fire or explosion.

4.4 Rehabilitation methods

The selected rehabilitation objective can be undertaken using two different rehabilitation methods. The first one, named as simplified by the standard, can be use only for a life safety building performance level with a probability of 10% in 50 years. In addition to this restriction, it must be used for certain buildings of regular configuration that do not require advanced analytical procedures [Lupoi, 2003]. The systematic rehabilitation method, in the other hand, as it is expressed on the commentaries of the standard, may be applied to any building for any target building performance level.

4.5 Analysis procedures

As it was expressed before, four different analysis procedures are allowed for the evaluation of the seismic vulnerability assessment: Linear static (LSP), Linear dynamic (LDP), Non-linear static (NSP) and Non-linear dynamic (NDP).

The selection of the adequate procedure that must be used should be done taking into account the methodology that the standard presents. In first place, it is required to verify the "regularity" of the structure under study. This is done by means of geometrical requirements and the calculation of the demand-capacity ratios (DCR) that intends to quantify the non-linearity of the structure. The following correspond to the irregularities considered by the standard.

- In-plane discontinuity irregularity: One lateral-force resisting element is present in one story, but does not continue, or is offset within the plane of the element, in the story immediately below.

- Out-of-plane discontinuity irregularity: An element in one story is offset out-of-plane relative to that element in an adjacent story

- Severe weak story irregularity: The ratio of the average shear of any story exceeds 125% the one of an adjacent story in the same direction.

- Severe torsional strength irregularity: The diaphragm above the story under consideration is not flexible and, for a given direction, the ratio of the critical element DCRs for primary elements on one side of the centre of resistance of a story, to those on the other side of the centre of resistance of the story, exceeds 1.5.

The definition for DCR given by the standard is as follows:

$$DCR = \frac{Q_{UD}}{Q_{CE}} \tag{4.1}$$

Where:

Q_{UD} : Force due to the gravity and earthquake loads.

Q_{CE} : Expected strength of the component or element (axial, shear, bending).

If for all the components the value of DCR is lower than two, linear procedures can be used. If one or more components present DCR greater than two but the building presents no irregularities, linear procedures are also applicable. If neither of the previous conditions are satisfied, linear procedures are not applicable. It must be noticed that values of DCR lower than one imply that the component respond elastically.

In addition to these requirements, linear procedures can not be used in the evaluation of structures where the effect of higher modes is relevant.

4.5.1 Linear static procedure (LSP)

This corresponds to the horizontal equivalent force procedure contain in the NSR-98. In this procedure, the design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacement shall be determined using a linearly elastic, static analysis.

This method shall not be used when the fundamental period of the building is greater than or equal to 3.5 times T_s, the ratio of the horizontal dimension at any story to the corresponding dimension at an adjacent story exceeds 1.4, the building has a severe torsional stiffness irregularity in any story, a severe vertical mass or stiffness irregularity or a non-orthogonal lateral-force resisting system. In these cases the linear dynamic procedure must be used.

The FEMA 356 presents three different methods in order to obtain the structural period.

METHOD 1 – Analytical

The period is obtained using a dynamic analysis of the mathematical model of the building.

METHOD 2 – Empirical

It is done using the following expression, similar to the one used by the NSR-98:

$$T_a = C_t h_n^{\beta} \tag{4.2}$$

A values of C_t equal to 0.08 must be used for concrete moment resisting frames and structural steel frames with eccentric diagonals. The value of β that must be used is 0.90 for the same type of structures.

METHOD 3 – Approximate Rayleigh-Ritz method

FEMA-356 permits the use of the Rayleigh-Ritz method for any building. The standard includes some expression for one-story buildings. The following expression, for instance, can be use for one-story buildings with single span flexible diaphragms:

$$T = (0.1\Delta_w + 0.078 * \Delta_d)^{0.5} \tag{4.3}$$

Where:

Δ_w: In-plane wall displacement, due to a lateral load in the direction under consideration, equal to the weight of the diaphragm (inches).
Δ_d: In-plane diaphragm displacement, due to a lateral load in the direction under consideration, equal to the weight of the diaphragm (inches).

The standard allows computing the forces and deformations using the pseudo lateral loads. The distribution of the pseudo lateral load must be carried following the next steps:

$$V = C_1 C_2 C_3 C_m S_a W \tag{4.4}$$

Where:

V : Pseudo lateral load

C_1: Factor that relates the inelastic displacement to the elastic one computed with the linear procedure. This can be taken as:

- C_1 : 1.5 for $T < 0.10$ sec
- C_1 : 1.0 for $T \geq T_s$, with T_s equal to the period associated to the change from constant acceleration to constant velocity in the response spectrum.

C_2 : It represents the pinched hysteris shape, strength deterioration and stiffness degradation at maximum displacement response. For linear procedures this can be taken as 1.0.

C_3: Modification factor to represent P-Δ effects.

C_m : Factor that takes into account the higher modes participation effect.

S_a : Response spectrum acceleration at the fundamental period and damping ratio.

W : Effective seismic weight of the building.

The vertical distribution of seismic forces is done using an expression equal to the one used by the NSR-98 (see equation 3.8).

4.5.2 Linear dynamic procedure (LDP)

When this method is selected, either response spectrum method or time history method can be used. The forces and deformations computed must be affected by the factors C_1, C_2 y C_3 described above.

4.5.3 Non-linear static procedure (NSP)

This can be used for structures with low high modes effects. It can be verified comparing the value of the shear at any story for the first mode and for the number of modes that guarantee that the 90% of the effective mass is participating. If the value of this combination is 130% than the one for the first mode, it can be considered that the higher modes effects are important and NSP can not be used.

The nonlinear relationship between base shear and displacement of the control node must be constructed taking into account the lateral load patterns distribution selected and additional model considerations used. This curve must be idealized in a bilinear relationship that includes the lateral stiffness K_e (secant stiffness at a base shear force equal to the 60% of the effective yield strength of the structure), effective yield strength V_y and the post-yield stiffness α according to a graphical process balances the areas above and below the curve.

After this, the effective fundamental period must be determined as:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \tag{4.5}$$

Where

T_i and K_i : Elastic fundamental period and elastic lateral stiffness respectively.

K_e : Effective lateral stiffness.

In general, the target displacement must be computed as:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{4.6}$$

With:

C_0 : Modification factor that relates the spectral displacement of an equivalent SDOF system to the roof displacement of the MDOF system.

4.5.4 Non-linear dynamic procedure (NDP)

It can be used for any structure but the standard specifies that the outcome must be revised by an independent engineer, with certified experience in seismic design and NDP, in order to be accepted.

When NDP is used the mathematical model must include the nonlinear load-deformation characteristics of individual components and elements of the building. This shall be subjected to earthquake shaking represented by ground motion time histories with the characteristics prescribed in the standard. The force-displacement behaviour of all components included in the model must contain strength degradation and residual strength, if any.

The internal forces and displacements shall be compared with nonlinear acceptance criteria.

4.6 Acceptance criteria

The acceptance criteria used depends on the analysis procedure, the classification of the elements (primary or secondary, see 4.3.1) and the classification of the actions (deformation-controlled or ductile and force-controlled or non-ductile).

4.6.1 Linear procedures

For deformation-controlled actions, the design actions Q_{UD} must be computed as:

$$Q_{UD} = Q_G \pm Q_E \tag{4.7}$$

Where:

Q_E : Action due to design earthquake loads.

Q_G : Action due to design gravity loads.

The acceptance criteria specify that:

$$m\kappa Q_{CE} \geq Q_{UD} \tag{4.8}$$

Q_{CE} : Expected strength of the component.

m : Element demand modifier to account for expected ductility (see 7.3.1).

κ : Knowledge factor (see 4.3).

For force controlled actions, Q_{UF}, the maximum action that can be developed in a component, shall be computed as:

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3 J} \tag{4.9}$$

Where:

J : Force delivery factor that can be obtained from the evaluation of DCR or from a range that goes from 1.0 up to 2.0 and depends on the seismicity zone and target building performance level.

For force-controlled actions, it must be satisfied that:

$$\kappa Q_{CL} \geq Q_{UF} \tag{4.10}$$

Where:

Q_{CL}: The lower bound strength considering all coexisting actions in the element.

4.6.2 Non-Linear procedures

For deformation-controlled actions, the elements shall have capacities larger than the demands obtained at the target displacement. In addition, the target base shear (V_t) shall not be less than the 80% of the effective yield strength.

For force-controlled actions, the internal forces of the elements shall not be larger than the lower-bound strengths. In addition, the flexural plastic hinges shall not form away from components ends unless it was considered in that way during analysis.

5 RECOMENDATIONS OF THE NEW ZEALAND SOCIETY FOR EARTHQUAKE ENGINEERING

Draft guidelines for seismic assessment and retrofitting were released in 1996 by the New Zealand National Society for Earthquake Engineering. The 2006 edition (not longer a draft) includes two different levels of assessment: an initial evaluation process that is intended to be used by local authorities in order to identify high risk buildings and a detailed assessment. The present document is focused on the detailed assessment but a brief description of the initial evaluation process is presented.

The recommendations present the procedure that must be followed if a displacement-based method or a force-based method is selected to perform the assessment. The inclusion of the two procedures is done, according to the standard, because even though "*it is generally considered that displacement-based methods produce more rational and less conservative assessment outcomes, it is acknowledged that most designers are currently more familiar with force-based approaches*".

5.1 Initial evaluation process

These guidelines are intended to be a coarse screening involving a few resources as reasonably possible to identify potentially high risk buildings. It is expressed in the New Zealand recommendations that this process must be carried out by registered engineers with experience in seismic design of buildings.

This process makes an initial assessment of the performance of an existing building and compares it against the standard required for a new building: the outcome is a value named *percentage new building standard* (%NBS). This is then reduced due to significant vulnerability features and deficiencies (termed "critical structural weaknesses") and hence the structural performance score (SPS) is obtained. According to the recommendations when a value of SPS lower than 33 is obtained, the building is assessed as potentially "not safe in earthquake" and should be retrofitted. With values between 33 and 67, the structures must be evaluated according to detailed assessment.

In order to compute %NBS some factors as the code used for the original design, soil type, structural period, assessed ductility of existing structure, risk scaling factor, importance and occupancy of the building and the structural performance factor are taking into account.

The critical structural weaknesses considered are irregularity in plan and in elevation, short columns, site and soil issues and pounding with adjacent buildings. These factors on the structure are classified as severe, significant and insignificant depending on their effects on the structural performance of the building.

5.2 Detailed assessment of earthquake performance

This procedure allows the engineer to evaluate with greater accuracy the capability of an existing building to withstand a seismic event. This is made by *"the determination of the demands an earthquake places on structural elements and the capacity of such elements to meet those demands"*.

The recommendations define the potential risk of a building in terms of the percentage of the design earthquake required to attain the ultimate limit state (ULS). According to this, the following definitions are presented:

- New buildings: Attain ULS when subject to no less than 100% of the design earthquake at the site.
- Existing buildings – low risk: Attain ULS when subject to no less than 67% of the design earthquake at the site.
- Existing buildings – medium risk: Attain ULS when subject to no less than 33% of the design earthquake at the site.
- Existing buildings – high risk: Attain ULS when subject to less than 33% of the design earthquake at the site.

The definition of ULS used in the code is taken from AS/NZS 1170.0:2002 and NZS1170.5. This definition states that the ULS is reached "*when the building or any part of it:*

- *Loses stability*
- *exceeds prescribed displacement limits*
- *is strained to the accepted limits of the structural materials involved"*.

It is reported that in order to consider sources of overstrength and overcapacity such as material overstrength and contribution of non-structural elements, analysis and assessment should be made considering only two-thirds of the seismic action. This recommendation does not apply when in the analysis the sources of overstrength and overcapacity have already been accounted for.

The standard includes a brief description of the major structural problems of existing buildings. According to the document, they can be summarized as:

- Inadequate anchorage of transverse reinforcement due to poor anchorage details.
- Inadequate shear strength of beam-column joints due to insufficient transverse reinforcement.
- Inadequate anchorage of longitudinal reinforcement due to poor anchorage details.
- Inadequate strength of footings and/or piles and their connections.
- Uncertain behaviour of the structure as a result of the presence of non-structural elements, typically infill walls, which can significantly alter the structural behaviour of the frame.

5.2.1 Global structural analysis procedures (GSAPs)

The recommendations present a description of the analysis methods that could be used according to some applicability limits introduced by the guidelines. The following table, taken from the New Zealand standard, is a summary of these analysis procedures and their applicability:

Table 5.1. Analysis procedures and applicability

Analysis method	Applicability notes
1 Elastic methods	
1.1 Equivalent Static analysis (ESA)	Building, height not exceeding 30 m. No significant vertical stiffness irregularity present. No significant torsional stiffness irregularity present. Orthogonal lateral force resisting systems present. Elastic response or low ductility demand (μ<2.0) under design level earthquake, without: • in plan or out of plan discontinuities in primary lateral force resisting system • significant weak storey irregularity • significant torsional strength irregularity in any storey.
1.2 Modal response spectrum analysis (EMA)	Elastic response or low ductility demand (μ<2.0) under design level earthquake, without: • in plan or out of plan discontinuities in primary lateral force resisting system • significant weak storey irregularity • significant torsional strength irregularity in any storey.

2 Inelastic methods	
2.1 Simple lateral mechanism analysis (SlaMA)	No significant torsional stiffness irregularity. Higher modes effect not critical[5].
2.2 Lateral pushover analysis (LPA)	Higher modes effect not critical.
2.3 Inelastic time history analysis (ITHA)	May be used for any structure.

5.2.2 Materials and members strengths

The recommendations state that the use of nominal strengths specified in the original design is inappropriate for assessment analyses and suggests that the use of a probable yield strength of 280 MPa for the reinforcement steel for structures designed in the 1930-70 period. For concrete, 1.5 times the nominal specified compressive strength can be used.

However, it is recommend the performance of tests for concrete and steel when this is practicable.

5.2.3 Force based approach

The performance of the components is evaluated by examining the forces in critical elements and using rules to asses the limits of integrity of the structural members.

At the beginning the member capacities are evaluated according to standard theory, using the expressions included in the guidelines which focus on flexural and shear strength of beams and columns and shear strength of beam-column joints.

After that, the probable location of post-elastic deformations due to severe earthquakes forces is identified and hence the inelastic critical mechanism is determined. In order to do this, the sway potential index (S_i) is introduced to compare the sum of probable flexural strengths of the beams and the columns at the joint centroids:

$$S_i = \frac{\sum (M_{bl} + M_{br})}{\sum (M_{ca} + M_{cb})} \tag{5.1}$$

[5] The SRSS (square root of the sum of squares) base shear due to all modes, accounting for at least 90% of total mass, does not exceed 1.30 times that due to the first mode alone or building fundaments period exceeds approximately one second.

Where:

Where M_{bl} and M_{br} are the expected maximum flexural strengths for the beams either side of a joint and M_{ca} and M_{cb} are the minimum expected column strengths above and below the joints; all determined at the centroids of the joints. These are summed for all joints across the level being examined.

This index allows investigating whether plastic hinges occur in beams or columns considering that a value of $S_i>1.0$ implies that plastic hinges can be expected. However, the recommendations express that in order to take into account the higher modes effects, this bound should be taken as 0.85.

In this stage, the recommendations present three different methods in order to compute the lateral capacity of the frame. The first approach corresponds to a lower bound of the capacity of the frame. It is obtained as the lateral seismic force which has to be applied to cause the first plastic hinge during a static elastic structural analysis with earthquake force increasing from zero.

The second approach takes into account the mechanism of post-elastic deformation that can be found from the previously computed S_i. For example, if for a given storey it is found that the value of S_i is greater than 0.85 in a column joint, the probable lateral force capacity of the frame can be found as the sum of storey shear forces in the columns. These are obtained as the sum of the probable flexural strengths of the plastic hinges at the top and bottom of the columns of the storey divided by the storey height. This value corresponds to an upper limit of the ultimate base shear. The NZ recommendations express that this procedure must be carried out carefully because the correct mechanism may be missed and the lateral force capacity overestimated as result.

Finally, the third approach specifies the use of nonlinear lateral pushover structural analysis (LPA). This implies more effort but the results are more accurate and reliable. The problem regarding this method falls in the distribution of the lateral seismic forces. Traditionally, an inverted triangular distribution is used but the recommendations express that this does not take into account higher modes effects.

The probable basic seismic hazard acceleration coefficient (the spectral acceleration), corresponding to the ideal lateral force capacity of the frame is calculated by the following expression:

$$C_h(T,\mu) = \frac{V}{W_t S_p R Z} \tag{5.2}$$

Where

$C_h(T, \mu)$: Basic seismic hazard coefficient (g).

V: Lateral capacity of the frame found by any of the methods presented above,

W_t: The seismic weight of the structure,

S_p: The structural performance factor (an average value of 0.67 can be used),

R: The risk factor of the structure,

Z: The zone factor.

The structural period of the structure must be computed including the effect of cracking on the section properties but the standard does not indicate the appropriate method in order to compute it. With C_h and the appropriate seismic hazard acceleration spectra, the required structure ductility factor μ_{sd} can be estimated. The following figure shows the way this can be done for a structural period of 1.0 sec and basic seismic hazard coefficient equal to 0.5 g:

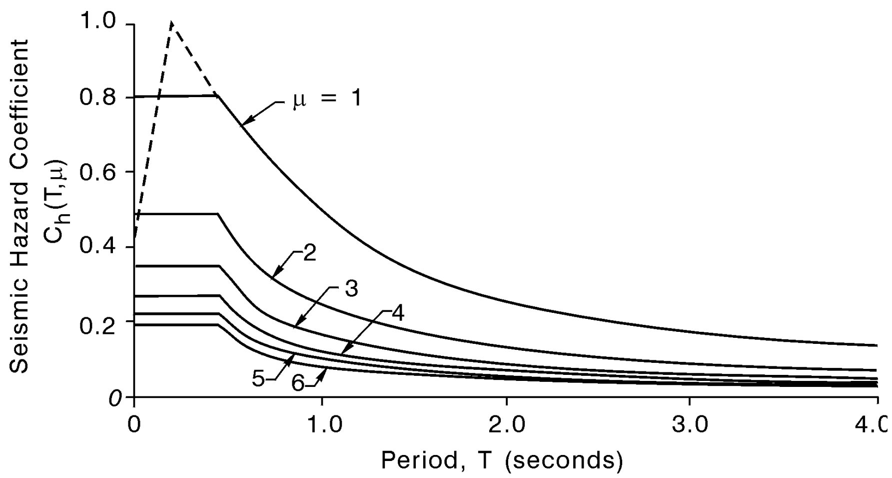


Figure 5.1. Estimation of the required structure ductility factor μ_{sd}

After this, it is required to verify if the available member ductility is able to satisfy the required ductility found before. The standard presents three methods to find the available ductility factor (μ_{sc}). The first one is based on the likely failure mechanism and the actual member detailing. The following table summarizes the recommendations included in the code:

Table 5.2. Qualitative evaluation of available structural ductility factor

Failure mechanism	As-built member detailing	Available structural ductility factor (μ_{sc})
Potential plastic hinge regions in frames where a side-sway mechanism is likely	Anchored stirrups with spacing s ≤ d/2 and s ≤ $6d_b$	6
	Not anchored stirrups with spacing s > d/2 or s > $6d_b$	2
	Intermediate values of μ can be used according to existing detailing.	

Plastic hinge regions at the base of columns or for frames of one or two storeys where a side-sway is likely	Anchored hoops with spacing s ≤ d/4 and s ≤ $6d_b$ and the ratio of volume of transverse reinforcement/volume of concrete core ≥ $0.01*(1+2.85Nf'_cA_g)$ and the confined length of column at base ≥ $h*(1+2.85Nf'_cA_g)$.	6
	Not anchored hoops or s > d/2 or s > 16 d_b	2
Potential plastic hinge regions in columns of frames of more than two storeys where a side-sway is likely		1.5

The second method, considered more accurate than the previous one, starts with the computation of the available curvature ductility factor (ϕ_u/ϕ_y) or plastic rotation [$(\phi_u-\phi_y)*L_p$], taking into account the detailing of the confining reinforcement. The standard presents different methods to compute the available ultimate curvature (ϕ_u), curvature at first yield (ϕ_y) and the equivalent plastic hinge length (L_p).

The third proposed method suggests the use of a nonlinear lateral pushover analysis, with lateral forces gradually increased. It allows the formation of a mechanism where the available ultimate curvature is reached at the critical plastic hinge, with available structural (displacement) ductility found from the ultimate displacement.

Afterwards, the comparison between the available and required ductility must be carried out. If it is found that $\mu_{sc} < \mu_{sd}$, the structure must be retrofitted. Otherwise, it is required to check if degradation of shear strength reduces the lateral force capacity of the structure. The recommendations establish that if the reduced shear strengths are found to be less than the shear forces and the flexural strengths at the plastic hinges and/or beam-column joints, the frame will need to be retrofitted.

Finally, it is required to check if the interstorey drift limit is large enough to induce important P-Δ effects or damage to non-structural elements.

5.2.4 Displacement-based approach

The response of the building structure is considered from the outset of the displacements due to the ground shaking. The procedure presented in the recommendations in order to perform a displacement based assessment followed the method proposed by Priestley (1995).

The first step consists in the evaluation of the probable strength of the members. It is expressed that a value of ϕ= 1.0 (strength reduction factor) may be used when flexural strength is computed since either the probable properties of the members as built or established default values are used.

After this, the sway potential index must be computed using the expression presented for force based procedure in order to establish the most likely failure mechanism. The probable horizontal seismic base shear capacity, V_b, of the structure can be evaluated using a simple lateral mechanism analysis, a lateral pushover analysis or an inelastic time history analysis, describe in 5.2.3.

At this stage, the shear strengths of the members are computed using the expressions presented for force based procedures. The values found must be compared against the member ductilities obtained by the moment-curvature analysis performed previously. This must be done as well for the beam-column joint shear strengths.

The plastic storey drift capacity is then computed using the plastic rotation capacities obtained from the limiting member curvature ductilities (that depend on the detailing of the transverse reinforcement in the potential hinge regions at the beam ends). The following expressions, included in the standard, can be used in order to compute the displacement ductility factor (μ_{sc}):

- For a beam sidesway mechanism:

$$if\ n \leq 4 \rightarrow \mu_s = 1 + \frac{0.64(\phi_u - \phi_y)L_p H}{\Delta_y} \tag{5.3}$$

$$if\ n \geq 20 \rightarrow \mu_s = 1 + \frac{0.44(\phi_u - \phi_y)L_p H}{\Delta_y} \tag{5.4}$$

$$if\ 4 < n < 20 \rightarrow \mu_s = 1 + \frac{(0.64 - 0.0125(n-4)(\phi_u - \phi_y)L_p H}{\Delta_y} \tag{5.5}$$

- For a column sidesway mechanism:

$$\mu_s = 1 + \frac{(\phi_u - \phi_y)L_p H}{n\Delta_y} \tag{5.6}$$

Where

n: number of storeys.

ϕ_u : Available ultimate curvature.

ϕ_y : Curvature at first yield.

L_p : Equivalente plastic hinge length.

H : Building height.

The value of Δ_{sc} (available displacement capacity at failure) is computed from the global ductility capacity of the structure found from the mechanism of plastic deformation and the critical storey drift established previously:

$$\Delta_{sc} = \Delta_y (1 + \mu_{sc}) \tag{5.7}$$

It is expressed that for structures with plan irregularities, the effect of torsion on displacement must be considered.

In this stage, the substitute structure approach proposed by Shibata and Sozen must be used in order to obtain the demand due to the ground motion. In order to do this, the effective stiffness (K_{eff}) and period of the structure are computed with the following expressions (see figure 5.2 for details):

$$K_{eff} = \frac{V_b}{\Delta_{sc}} \tag{5.8}$$

And the structural period is computed as:

$$T = 2\pi \sqrt{\frac{M}{k_{eff}}} \tag{5.10}$$

With M as the mass of the structure.

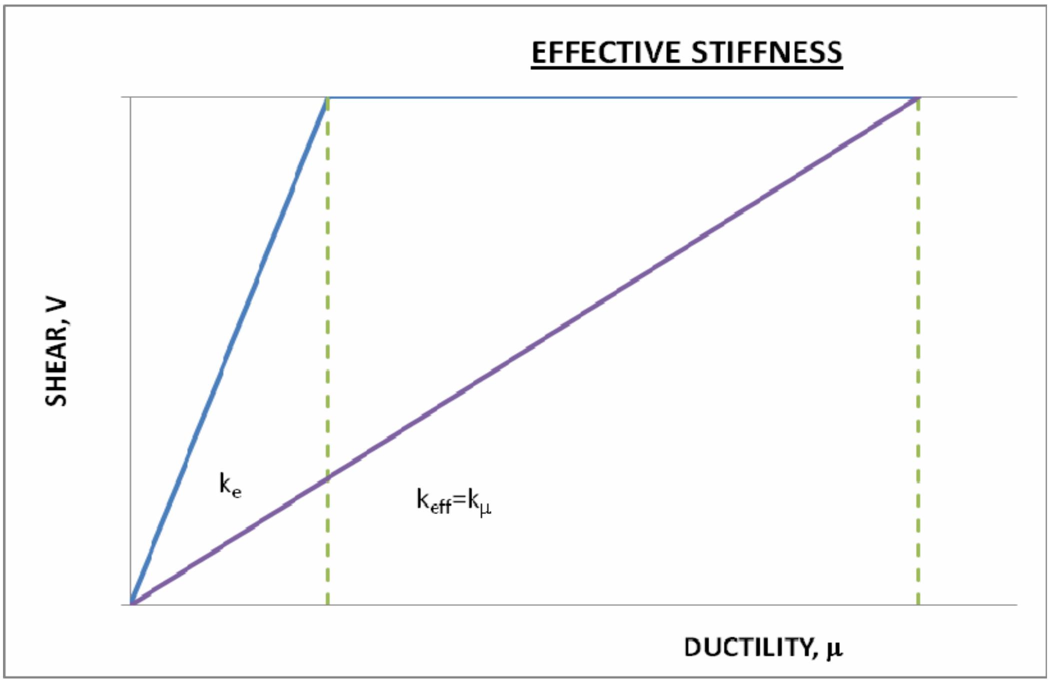


Figure 5.2. Effective stiffness

The next step is related with the evaluation of the equivalent viscous damping (ξ_{eff}). Table 5.3 is presented in the standard and may be used in order to computed the damping for concrete structures. The following definitions are required for its use:

α_s : post yield to initial stiffness ratio.

η : Efficiency factor, defined as the ratio of the actual area enclosed by the hysteresis loop to that of the assumed perfect bilinear hysteresis.

Table 5.3. Typical values of ξ_{eff} for concrete structures.

Structural Type	μ	η	α_s						
			-0.05	**-0.03**	**0**	**0.02**	**0.05**	**0.10**	**0.20**
Ductile	6	0.35	0.31	0.28	0.24	0.22	0.19	0.16	0.12
Limited ductile	3	0.30	0.19	0.20	0.18	0.17	0.16	0.15	0.12
Limited ductile	2	0.25	0.14	0.13	0.13	0.13	0.12	0.12	0.10
Nominally ductile	1.25	0.20	0.08	0.08	0.08	0.07	0.07	0.07	0.07

Given the structural period and the effective structural damping, the proper displacement spectra is used in order to obtain the maximum displacement demand (Δ_{sd}) and this is compared with the ultimate displacement capacity (Δ_{sc}). When the ratio Δ_{sc}/Δ_{sd} is greater than one, the structural performance is considered acceptable, otherwise, retrofitting is required. It must be noticed that the standard just provides displacement spectra for 5% of damping; they can be multiplied by the factor K_ξ to obtain the spectra for different dampings:

$$K_\xi = \left(\frac{7}{2+\xi} \right)^{0.5} \tag{5.2}$$

6 EUROCODE 8 – PART 3: ASSESMENT AND RETROFITTING OF BUILDINGS

The code is intended to provide a guideline for the evaluation of the seismic performance of existence building structures and the requirements for select and design the necessary corrective measures and retrofitting techniques.

6.1 Performance requirements

The Eurocode-8 describes three different limit states defined as follows, including the appropriate return period for the seismic action:

- Near Collapse (NC): The damage in the structure doesn't allow it to withstand further seismic excitation although the structural system is able to carry gravity loads. As far as non-structural elements are concerned, most of them suffered collapse. In addition, the building presents large residual drifts. This limit state is achieved with return period of 2475 years corresponding to a probability of exceedance of 2% in 50 years.

- Significant Damage (SD): The structure presents significant damage but its lateral resistance system is able to resist after-shocks of moderate intensity. In addition, it presents moderate residual drifts and important damage to non-structural elements but without collapse. According to the definition presented by the Eurocode-8, the structure is unlikely to be economically repairable. This limit state is achieved with return period of 475 years corresponding to a probability of exceedance of 10% in 50 years.

- Damage Limitation (DL): The structure is lightly damaged with negligible permanent drifts. The non-structural elements might present distributed cracking but it is economically repairable. This limit state is achieved with return period of 225 years corresponding to a probability of exceedance of 20% in 50 years.

6.2 Compliance criteria

The code differentiates the compliance criteria considering whether the q-factor approach is used or not. This approach allows avoiding inelastic structural analysis and the capacity of the structure to dissipate energy through ductile behaviour is done by reducing the elastic spectrum by a factor termed “q”. The values recommended by the code for concrete structure range from 1.5 to 2.0 but these values could be increased when a detailed assessment of the local and global ductility of the building justifies it. It must be said that the Eurocode-8 doesn’t give exact criteria for the choice of an exact number.

In addition, the Eurocode-8 specifies the compliance criteria introducing a distinction between “ductile” and “brittle” elements or mechanisms. The former are defined as beams, columns, and walls under flexure with and without axial force. Brittle elements, in the other hand, include shear mechanism of beams, columns, wall and joints.

For each limit state, demands must be computing according with the relevant seismic actions. In NC and SD, capacity for ductile elements shall be computed based on ultimate deformations and for brittle elements must be based on ultimate strengths, unless the q-factor approach is used. For DL, capacities must be based on yield strengths for all structural elements.

When the SD limit state is checked using the q-factor approach, demands shall be computed considering the reduced seismic action and the capacities shall be evaluated as for non-seismic design situations. For the DL limit state performed with q-factor approach, demands and capacities are compared in terms of the mean interstorey drift. The Eurocode establishes that the q-factor approach is not suitable for checking NC limit state.

6.3 Information for structural assessment

The main source of information for the earthquake assessment, according to the Eurocode-8, should be:

- Specific available documentation of the building.
- Codes and standards used when the structure was designed.
- Field investigation.
- In-situ and/or laboratory test and measurements.

The information collected from these sources should include:

- Identification of the structural system.
- Orientation of one-way floor slabs.
- Identification of the ground conditions.

- Information about properties of the elements of the building and their constituent materials (concrete strength, steel yield strength, ultimate strength and ultimate strain).
- Amount of longitudinal steel in beams, columns and walls.
- Amount and detailing of confining steel in critical regions.
- Identification of poor detailing and material defects.
- Information about previous and present structural damage.

6.3.1 Knowledge levels

The Eurocode-8 presents three different knowledge levels that allow the selection of the type of analysis and the confidence factor values according to the gathered information about geometry, details and materials of the structure. The following correspond to the definitions presented by the Eurocode for the different knowledge levels:

- Limited Knowledge Level (KL1): As long as geometry is concerned, it is known by survey or from the original construction drawings. If it is known from the latter, the Eurocode-8 establishes that dimensions of overall geometry and members sizes should be checked in site. The details, in the other hand, are not known and are assumed taking into account the traditional practice used when the building was constructed, but an inspection of the most critical elements should be performed. Finally, default values of the mechanical properties of the materials should be used based on the recommendations of the standards at the time but limited in-situ testing on the most critical elements should be carried out.

- Normal Knowledge (KL2): the geometry is known from an extended survey or from the original construction drawings. If it is known from the latter, the Eurocode-8 establishes that dimensions of overall geometry and members sizes should be checked in site. The details are known from an extended in-situ inspection or incomplete set of detailing drawings. Finally, the mechanical properties of the materials are obtained from an extend in-situ testing or from original design specifications.

- Full Knowledge (KL3): the geometry is known from a comprehensive survey or from the original construction drawings. If it is known from the latter, the Eurocode-8 establishes that dimensions of overall geometry and members sizes should be checked in site. The details are known from a comprehensive in-situ inspection or complete set of detailing drawings. Finally, the mechanical properties of the materials are obtained from a comprehensive in-situ testing or from original test reports.

The Table 6.1, taken from the code, summarizes the requirements given for each one of the knowledge levels prescribed by the Eurocode-8. It also included the recommend values of confidence factors and the type of analysis allowed according to the knowledge level achieved. The Table 6.2 includes the definition of the levels of inspection and testing according to the knowledge level achieved:

Table 6.1. Knowledge levels, methods of analysis and confidence factors

Knowledge Level	Geometry	Details	Materials	Analysis	Confidence factors (CF)
KL1	From original outline construction drawings with sample visual survey or from full survey	Simulated design in accordance with relevant practice and from limited in-situ inspection	Default values with accordance with standards of the time of construction and from limited in-situ testing	Lateral Force Procedure, Modal Response Spectrum	1.35
KL2		From incomplete original detailed construction drawings with limited in-situ inspection or from extended in-situ inspection	From original design specifications with limited in-situ testing or from extended in-situ testing	All	1.20
KL3		From original detailed construction drawings with limited in-situ inspection or from comprehensive in-situ inspection	From original test reports with limited in-situ testing or from comprehensive in-situ testing	All	1.00

Table 6.2. Minimum requirements for different levels of inspection and testing

	For each type of primary elements (beam, column, wall)	
	Inspection (of details)	**Testing (of materials)**
Level of inspection and testing	**Percentage of elements that are checked for details**	**Material samples per floor**
Limited	20	1
Extended	50	2
Comprehensive	80	3

6.4 Structural analysis and modelling

In addition to the recommendation about representation of stiffness and masses, the Eurocode-8 establishes that the model shall consider the contribution of joint regions to the deformability of the building. The non-structural elements which could influence the response of the structural system must be also included.

For concrete buildings, the model should take into account the effect of cracking. This can be done, unless a more detailed analysis is performed, taking the elastic flexural and shear stiffness of concrete elements equal to the half of the stiffness of uncracked elements.

For non-linear methods, the models should include the strength of structural elements and their post-elastic behaviour. Bilinear force-deformation envelopes should be used at the element level as a minimum, but more elaborated models that take into account pre-crack and post-crack stiffnesses are allowed.

6.5 Methods of analysis

6.5.1 Lateral force method

This method is recommended for buildings whose are not affected by higher mode effects. In addition, the buildings should meet the elevation regularity criteria prescribed by the Eurocode and they should have values of structural period (T) lower than $4.0T_c$ or 2.0 s, where T_c corresponds to the higher limit of the constant acceleration branch of the spectrum. Other condition presented in the code established that the higher and lower ratio between the demand (D_i) and the corresponding capacity (C_i) for all the ductile elements of the building shouldn't exceed a value that ranges from 2.0 to 3.0.

According to the code, the base shear force, F_b, must be computed as follows:

$$F_b = S_d T_1 m \lambda \tag{6.1}$$

Where:

T_1 : Fundamental structural period of vibration for each horizontal direction.

S_d : Acceleration in the design spectrum for a given period T_1.

m : Total mass of the building.

λ : Factor that takes into account that the effective mass of the first mode for building with two or more storeys is, in average, 15% smaller than the total mass of the building. The Eurocode proposes a value of λ = 0.85 for buildings with two or more storeys, or λ=1.0 otherwise.

For the determination of the structural period, the Eurocode allows the use of expression based on methods of structural dynamics. In addition, for buildings with heights up to 40 m, the following equation might be used:

$$T_1 = C_t H^{0.75} \tag{6.2}$$

For moment resistant space concrete frames, the appropriate value of C_t is equal to 0.075.

The distribution of the horizontal seismic forces can be done using the displacement shape response in each direction or assuming response shape where displacements increase linearly along the height.

The accidental torsional effects can be considered multiplying the effects that the horizontal seismic forces produced in the individual resisting elements by a factor δ given by:

$$\delta = 1 + 0.6 \frac{x}{L_e} \tag{6.3}$$

Where

x : Distance from the element considered to the centre of the building, measured perpendicularly to the seismic direction.

L_e : Distance between the two outermost lateral load resisting elements.

When two planar models are used, the factor 0.6 must be increased to 1.2.

6.5.2 Modal response spectrum analysis

This method must be followed for the buildings that don't fulfil the requirements prescribed for the application of the previous method.

In order to assure that the modes that have significantly contribution to the response are considered, it must be proved that the 90% of the effective mass in each direction participates. In addition, modes with effective masses greater than 5% shall be taken into account.

The combination of modal responses can be done using the complete quadratic combination (CCQ) or the square root of sum of squares (SRSS).

6.5.3 Non-linear methods

- ***Non-linear static (pushover) analysis:*** This analysis should be performed under two different load patterns. The first one corresponds to uniform lateral accelerations. The second pattern is similar to the lateral forces used for linear static analysis or derived from a modal response spectrum (fib, 2003).

  The capacity curve (relation between base shear and control displacement) should be for values of control displacement ranging from zero to 150% of the target displacement. The target displacement is defined as the seismic displacement reached by an equivalent single-degree-of-freedom system when this is subjected to the elastic response spectrum.

- ***Non-linear time-history analysis:*** In addition to the requirements prescribe for non-linear methods modelling (see 6.4), the model should incorporate rules that describe the element behaviour under post-elastic unloading-reloading cycles. At least 7 non-linear time-history analysis should be performed in order to obtain the average response used for design, unless the most unfavourable value is used.

6.6 Acceptance criteria

6.6.1 Ductile components/mechanisms

The ductile mechanisms are verified by comparing chord rotations from the linear analysis to limiting values corresponding to the limit state in consideration (Mihaylov, 2006). The chord rotation can be defined “as the element drift ratio, i.e., the deflection at the end of the shear span with respect to the tangent to the axis at the yielding end, divided by the shear span”.

The Eurocode presents expressions to compute the value of the total chord rotation capacity (elastic plus inelastic part) at ultimate, θ_u, for concrete members under cyclic loading that must be used for NC limit state. It also establishes that the chord rotation capacity for SD limit state, θ_{sd}, may be assumed as ¾ of the ultimate chord rotation.

The capacity used for DL limit state corresponds to the yielding bending moment under the design axial load, unless the verification is performed in terms of deformation. When the latter is done, the capacity corresponds to the chord rotation at yielding, θ_y.

6.6.2 Brittle components/mechanisms

The Eurocode presents an expression in order to compute the shear capacity of beams and columns. This expression accounts for degradation of the shear-transfer mechanism after flexural yielding by reducing the contribution of concrete mechanisms and stirrups for increasing ductility demands.

7 COMPARISON OF THE REVIEWED CODES

In this chapter, a comparison between the codes revisited is provided in order to highlight their main differences with the NSR-98. It also presents a discussion about the more important topics that should be modified in the Colombian standard in order to obtain more accurate results when a seismic vulnerability assessment of existing structures is performed.

It must be said that the New Zealand recommendations for displacement-based design will not be included in the comparisons carried out because the current version of the Colombian standard is based on entirely in a force-based methodology and the aim of this work consists in the improvement of the procedure used nowadays in the country by the structural design community.

7.1 Definition of a rehabilitation objective

According to the NSR-98, its aim is focused in life protection although its use can diminish the probability of total or partial loss of the structure. It states that a building retrofitted according to the requirements presented in the NSR-98 will be able to withstand low intensity earthquakes without damage, moderate intensity earthquakes without structural damage but minor non-structural damage and strong earthquakes without collapse. This is achieved using the design spectrum presented by the standard that is defined for an earthquake with return period of 475 years corresponding to a probability of exceedance of 10% in 50 years (see figure 3.3.2).

Taking this into account, the rehabilitation objective proposed by the NSR-98 can be related with a limited rehabilitation objective (see Table 4.5) from the FEMA-356 or with the significant damage (SD) limit state included in the Eurocode-8. The NZ recommendations, in the other hand, accept a higher probability of damage, considering a building attaining the ultimate limit state (ULS) when subject to no less than 67% of the design earthquake as a low risk structure: it implies a less conservative approach. In this case, the ULS is considered when the building suffers collapse or partial collapse which is likely to lead to loss life.

It is seen that even though the rehabilitation objective presented in the NSR-98 can be related with the ones included in the other codes, it is visible the lack of more detailed definition about the performance the structure will have if retrofitted according to the standard. This would allow to the owner to select a more specific retrofitting technique according to his needs.

7.2 Knowledge levels

The comparison made in this matter is mainly based on the procedures presented by the FEMA-356 and the Eurocode-8 because the requirements included in the New Zealand Recommendations are not clear about the information needed in order to perform the seismic assessment of existing buildings.

As it was expressed before, the NSR-98 proposed the factors ϕ_c and ϕ_e in order to take into account the design and construction quality and the current condition of the structure (see 3.4.7 and Table 3.10). These factors imply reductions of the original capacity of the elements from 81% up to 49%. Unfortunately, the standard does not provide a qualitative description of the way these factors should be selected. Furthermore, it is not clear the minimum required as-built information (drawings, testing, material properties, etc.) in order to qualify the design, construction and condition of the structure as good, regular or bad. According to this, the selection of these factors is a decision that the designer must take based entirely on his judgment.

The FEMA-356 and the Eurocode-8, in the other hand, proposed instead the knowledge factor, κ, and the confidence factor, CF, respectively. The κ factor can be either 1.0 or 0.75 depending on the selected rehabilitation objective, the analysis procedure and the collected data. In order to do so, it proposes three different levels of knowledge named as minimum, usual and comprehensive and presents a detailed description of each one of them.

For the Eurocode-8, the quality of the gathered information defines the admissible type of analysis and the CF factor. The recommended values for CF are 1.35, 1.20 and 1.00 (the higher the CF factor, the lower the knowledge level). It must be noticed that the factors ϕ_c and ϕ_e and κ are supposed to multiply the obtained capacity of the members but the CF is intended to divide it. Taking this into account, it is noticed that the FEMA-356 and the Eurocode-8 propose a reduction of 25% of the capacity which is lower than the maximum reduction proposed by the NSR-98.

When the methodology proposed by the FEMA-356 and the Eurocode-8 is compared with the NSR-98, the following aspects can be highlighted:

- It is clear that the definition of levels of knowledge gives more importance to the information collected and the test and surveys performed rather than the designer judgment.

  According to this, the NSR-98 should present the definition of levels of knowledge which allows to the designer to select the ϕ_c and ϕ_e factors according to the classification proposed by the standard (good, regular or bad).

- The values of ϕ_c and ϕ_e factors are more conservative than the proposed by the FEMA356 and the Eurocde-8 for κ and CF. This can be explained when the quality of the construction and design procedures used in Colombia in the past are considered and compared with the ones used in the United States or in the European countries. But it also must be said that even though Colombia just have a seismic code after 1984, some designers knew and used international codes before that. This last consideration could allow the use of factors ϕ_c and ϕ_e equal to one when it is justified by the collected information.

  As far as the lower bound is concerned (bad quality of design, construction and condition), a diminishing of 50% of the members capacity seems to be extremely conservative. First of all, if the values are proposed based on the lack of seismic detailing (ductility of the structure), this factor is already taking into account with the $\mathbf{R_0}$ factor according with the methodology proposed by the NSR-98. In addition, old structures were usually designed according to the elastic method (allowable stresses method) which provides higher safety factors than the ultimate limit state method used nowadays, at least when non-seismic loads are considered. These considerations allow using factors higher than the proposed by the NSR-98 due to the fact that the structure has been able to withstand serviceability loads without damage and the poor seismic detailing (bad quality of design) is also taken into account by the $\mathbf{R_0}$ factor.

7.3 Structural ductility

The NSR-98 takes into account the structural ductility with the use of the Factor **R** (see 3.3.1 and Table 3.2), which is selected by the designer, in the assessment of existing buildings, taking into account factors such as materials, structural system irregularities and his own judgment. This corresponds to one of the main weakness that the Colombian standard presents. According to the analysis presented in the ATC-34 about the global modifications factors used in some of the United States codes, these problems can be associated with:

- The current values of Factor **R** were obtained by a consensus of engineers made in the late 50's.

- The use of a unique factor to describe the structural ductility of an entire building is not appropriate because the ductility assessment of existing structures must be done taking into account local deficiencies instead of global ones.

- In general, the codes establish a group of seismic requirements that must be fulfilled in order to guarantee the performance defined by the Factor **R**. For existing structures, the accomplishment of these requirements can be reached by the structural elements (beams, columns, foundations) in different levels according to their detailing. This implies that the use of a unique factor **R** can underestimate or overestimate the ductility of the elements of the analyzed building.

- Recent investigations carried out to determine values of **R** using non-linear analysis (including time-history analysis) have shown that there is a big dispersion in the values found. This implies that this kind of methods should be used during the design stage to find a more reliable value of **R**. When the NSR-98 methodology is considered, it must be noticed that the selection of the factor **R** is made by the designer using entirely qualitative tools instead of performing a more exact procedure.

7.3.1 FEMA-356

The FEMA-356 considers the ductility, for deformed-controlled actions, with the inclusion of the *m*-factors. These are supposed to modify the individual expected strength of the elements of the building (see equation 4.8). Their selection is based on the detailing characteristics of the members, the performance level selected, the component type (primary or secondary), the failure mechanism controlled by shear or flexure and the analysis method chosen. Among the factors considered for the *m*-factor evaluation, the standard proposes the reinforcement ratio, the transversal reinforcement, the shear stress of the element and the axial load (for columns).

This makes that the selection of the *m*-factor for an elements becomes a more rational procedure. This is not the engineer´s decision because the designer has to find out the expected performance of the elements and the collected information starts to play a more important role. It must be noticed that the definition of the *m*-factor will make the seismic assessment more complicated than the use of the methodology proposed by the NSR-98, but it will also produce a more accurate outcome.

In a practical case, the FEMA-356 presents values of the *m*-factors ranging from 6.0 to 1.5 depending on the characteristics mentioned above for the analysis of beams. For columns, in the other hand, an upper and lower bound of 3.0 and 1.0 are defined. The NSR-98, proposes values that range from 7.0 (DES) to 1.0 but for $\mathbf{R_0}$, for a building constructed in Bogotá an average value of 3.75 could be used when there is not information about the original design for the assessment of the entire building (see 3.4.4). It must be said that the higher values proposed by the FEMA-356 corresponds to elements that have an almost total fulfilment of the seismic design requirements established by the standard.

Olarte (2005) shows that for typical elements designed according to old Colombian practices, the value of *m*-factor could vary from 1.5 to 2.0 for beams and for 1.0 to 1.5 for columns, which are significantly lower than the average value of the **R** factor defined above.

7.3.2 New Zealand recommendations

For force-based procedure, this standard requires the computation of two different structure global ductility factors: the required and the available. The first one is found based on the structural period of the building and the proper design spectra. For the second one, the standard proposes three different methods. The first method consists in a qualitative evaluation of the detailing of the structures according to the definition presented in the code (see Table 5.2). The second one, determinates it from the mechanism and the third one consist in non-linear lateral pushover structural analysis.

It is noticed that in none of these methods, the selection of the ductility factor does depend entirely on the engineer criteria because the standard gives him tools to evaluate the ductility. Furthermore, the code includes a description of advanced methods that will allow assessing this factor in a more reliable way, which is not found in the NSR-98. The maximum ductility the recommendations allow using is 6.0 but for DES structures, according to the NSR-98 this value can go as high as 7.0.

7.3.3 Eurocode-8

The Eurocode-8 allows taking into account the structural ductility by the use of the q-factors that are intend to reduce the seismic forces. According to the Eurocode-8, the q-factors consider the capacity of the structure to dissipate energy. It allows performing the design through an elastic analysis that involves the use of smaller forces due to the capacity of the structure to resist forces in the non-linear range.

This definitions agrees with the one contained by the NSR-98. In spite of this statement, the value of q recommended by the Eurocode-8 (1.5 for concrete frames unless larger values justified according to the code requirements) is considerable lower than the ones recommended by the NSR-98. It also must be noticed that the q-factor values, although consider ductility in a global manner, are closer to the *m*-factors proposed by the FEMA-356 (see 7.3.1).

In addition, the Eurocode-8 also allows performing a local evaluation of the deformation capacities of the ductile elements in terms of the chord rotation, with the use of unreduced seismic forces. (see 6.6.1).

7.4 Prescribed analysis procedures

The NSR-98 includes descriptions for the linear static and for the linear dynamic procedures. It only requires performing non-linear analysis for structures of the group of use IV and for those structures that, according to the engineer judgment, could present a nonlinear response only discovered with this method. Unfortunately, it does not give further information about the way these analyses should be performed or the minimum requirements they should comply. This can be understood when it is taken into account that the common design engineer is not familiarized with these kind of methods and in most of the assessment, they would perform, at most, a dynamic analysis.

7.4.1 Analysis procedures selection

In this section, some of the consideration proposed by the different codes in order to select the appropriate analysis procedure are presented and compared with the limitations included in the NSR-98. It must be said some of these considerations could be specific of a given code and in that case, the particular requirements are included:

- Period Limitation: The NSR-98 does not include prescriptions about this issue. The FEMA-356 proposes that the structural period can be larger than $3.5T_s$[6] (see 4.5.1.). The limits proposed by Eurocode-8 are $4.0T_c$[6] or 2.0 s (see 6.5.1). The NZ code establishes that simple lateral mechanisms analysis and lateral pushover analysis must

[6] T_s and T_c define the beginning of the constant displacement response range of the spectrum for FEMA-356 and Eurocode-8, respectively.

not be used for structures with structural period larger than 1.0 s (limit that defines higher mode effects as critical, see Table 5.1).

- Irregularities: These are taken into account by the NSR-98 with the use of the factors ϕ_a and ϕ_p (see Tables 3.4 and 3.5), intended to diminish the value of **R**, but no additional requirements are set in order to consider possible nonlinearities of the structure and its element response due to those irregularities.

  The other codes incorporate restrictions that have to do with the structural irregularity in order to use the linear static procedure. This is done because the use of these methods for irregular structures could miss the effect of non-linear response. The irregularities considered in the three codes are related with in-plan or out of plane discontinuities, weak storey and torsional strength irregularity.

- Collected information: The analysis procedure requirements for a given structure are based on the data collection. FEMA 356 only allows the use of linear static and dynamic procedures for the minimum knowledge level (Table 6.1). The Eurocode-8, in the other hand, prescribes the use of these methods for the knowledge level 1 (KL1, see Table 6.1). The NSR-98 as well as the New Zealand recommendations do not include requirements about this issue.

- DCR[7] (FEMA-356): This factor, described in section 4.5, intends to indicate the magnitude and distribution of inelastic demand for each action (such axial force, moment, shear), and whether linear procedures must be used or not. The limitations proposed for the applications of linear procedures according to FEMA-356, taking into account the DCR factor can be found in section 4.5 as well.

- Ductility demand (NZ recommendations): The linear procedures are not supposed to be used when the structures present high ductility demand (μ>2.0) under design earthquake. It is even expressed that the modal response spectrum analysis should be used for timber and concrete structures that are expected to perform elastically.

It is seen that the FEMA-356 and the New Zealand recommendations include some limitations about the use of linear procedures. It is even noticed that non-linear analysis are compulsory for some structures according to the parameters presented above. In the other hand, the Eurocode-8 and the NSR-98 do not present limitations for the use of linear procedures but the Eurocode-8 is stricter about the applicability of the static linear procedure.

7.4.2 Linear static procedure

The definitions given for this procedure in the NSR-98 and the Eurocode-8 are similar, with the exception of the factor λ proposed by the European standard (see 6.5.1) that takes into account the effective mass of the first mode for buildings with two or more storeys. In

[7] Demand-capacity ratio

addition, the NSR-98 includes the factor k that modifies the seismic force distribution according to the structural period (equation 3.8), likewise FEMA-356 does.

The equivalent static method proposed by the New Zealand code is based on the development of the first plastic hinge of the weaker element of the building, when lateral forces, increasing from zero, are applied to the model. The way this procedure must be carried is completely different when it is compared with the ones included in the other codes.

The FEMA-356 method, although is similar to the Eurocode-8 and the NSR-98 methodology, presents some factors, described in detail in 4.5.1. These factors pursuit to describe in a more accurate way the non-linear behavior of the structure, even though the analysis performed is elastic.

7.4.3 Linear dynamic procedure

The methodologies presented are similar with some differences regarding their applicability. In addition, the FEMA-356 states that the forces found by this method should be modified by the factors described in 4.5.1, in order to obtain a more accurate response, as it was done for linear static procedure.

The Eurocode-8 as well as the NSR-98, requires that the 90% of the effective modal masses participate but it also requires that all modes with effective modal masses greater than 5% of the total mass are considered, which is not demanded by the Colombian standard.

7.4.4 Non-Linear procedures

As it has been insisted on, the NSR-98 does not present the requirements to perform this kind of analysis. Furthermore, it only states that its use is compulsory for structures of group of use IV (indispensable edifications) and its use for other type structures depends entirely on the engineer judgment.

8 CONCLUSIONS

- Taking into account the Colombian seismicity and the large number of vulnerable edifications, it is required an updated of the NSR-98 considered the advances the seismic vulnerability assessment has obtained and that are now included in the codes used worldwide.

- The national government has made big efforts in order to carry out the development of the Colombian standards but, as it was noticed before, there is still more to do. It is understood that a gradual change from the old methods used by Colombian engineers to more modern techniques was required in the first two editions of the national codes (CCCSR-84 and NSR-98).

  Unfortunately, the procedures established by the NSR-98 can produce unrealistic data of the structural behavior. That is the reason why it is also required that the designers begin to familiarize with more accurate techniques as non-linear analysis, which are more proper to model the performance of irregular structures.

- The structural performance of a structure retrofitted according to the NSR-98 can be related to the limited rehabilitation objective proposed by the FEMA-356 or to the limit state of significant damage of the Eurocode-8 when factors such as expected damage and return period of the seismic action are taken into account.

  Even though this rehabilitation objective could be considered enough, the inclusion of a range of limit states should be done in the NSR-98. This would allow the designer to decide which limit states should be checked considering the importance of the structure and the owner's needs.

- The NSR-98 should be more demanding about the quality of collected information instead of decreasing the capacity of the elements with the use of low ϕ_c and ϕ_e factors. This could be done with the definitions of levels of knowledge as the ones proposed by FEMA-356 and Eurocode-8. In addition, the requirements to classify the quality of design, construction and conditions should be described instead of letting their definition entirely to the engineer judgment.

- The main weakness of the NSR-98 is linked with the use of the factor **R**. It is considered that the use of a global factor in order to estimate the structural ductility of a building could end up in a mistaken assessment. The use of local ductility factor that modifies the internal element forces or a change in the computation of the required element strength (considering available local ductility) will produce more reliable results and hence a better design.

- The presence of irregularities in a building will generate a non-linear structural response. The NSR-98 takes this into account with the use of the factors ϕ_a and ϕ_p, intended to diminish the value of the factor **R**, but it is clear that the use of the factors named above is not enough to take into consideration a non-linear response.

  With the study of the other codes, it was observed that the structural irregularity is a factor that determines the allowable analysis procedure. Keeping this in mind, the NSR-98 should consider a change in the way irregularities are included in the design using more strict requirements about the prescribed methods when the designer deals with non-regular structures.

- The linear dynamic procedure, although is more precise than a linear static procedure, is not recommend for every type of structures but the NSR-98 allows its use for the design or assessment of all the buildings covered by the code. As it was expressed before, this method is not the more precise for the determination of a non-linear response and its failures become more evident when the structural ductility is considered with the factor **R**. This allows concluding, as it was done before, that a special effort should be made by the NSR-98 in order to determine whether a linear procedure is permitted or not, taking into account factors such as structural irregularities.

- A lack of information about non-linear analysis in the NSR-98 is evident. In addition, it is used depends entirely on the engineer criteria. The Colombian standard should incorporate the minimum requirements for non-linear procedures (static and dynamic) and the cases its use should be compulsory according to features described in this dissertation.

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