Seismic Vulnerability of Existing R.C. Structures with Special Focus on High-Priority Buildings in Central America

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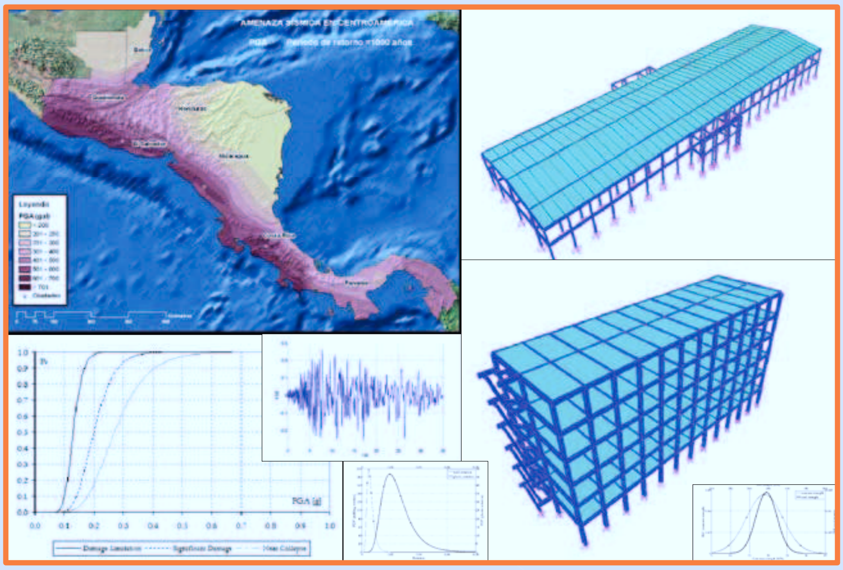

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Seismic Vulnerability of Existing R.C. Structures with Special Focus on High-Priority Buildings in Central America

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History teaches everything,
including the future.
- *Lamartine –*

Table of contents

List of figures

List of tables

Chapter 1. Introduction

As part of a regional cooperation project on the reduction of earthquake risk in the Central American countries Guatemala, El Salvador, and Nicaragua (RESIS-II), one of the major work tasks consists in the identification of the structural and non-structural seismic vulnerability of schools, hospitals and health centres.

The regional cooperation project RESIS II (*Reduccion de Riesgo Sismico*) is focused on earthquake risk reduction for the Central American countries Guatemala, El Salvador and Nicaragua, project funded by the Norwegian Embassy in Managua (Nicaragua) and headed by NORSAR (Norway). Beside a number of project tasks dealing with seismic hazard and risk assessment, a main part of the project is allocated to earthquake vulnerability studies of those buildings that are of major importance to the society: schools and hospitals. The integrity of schools and hospital buildings during an earthquake disaster is of utmost importance.

As given by PAHO (2004) the safety of a health facility (hospital) is determined by four different modules:

1. Geographic location (natural and man-made hazards or dangers, geotechnical properties of soils at the site);
2. Structural safety (structural vulnerability determined by the building's design and primary structural system);
3. Non-structural safety (non-structural elements such like infill walls, equipment, installations or furniture may not influence the building's stability but it may put people and the contents of the building at risk and increases the follow-up losses during evacuation);
4. Functional capacity (how hospital personnel is trained and organized in disaster situations is crucial in order to assess the hospitals functionality after the event).

As for any building, the assessment of the structural vulnerability is of utmost importance in order to get an idea about the building's exposure to suffer structural damage as a direct effect of earthquake shaking. However, especially for high-priority structures like hospitals and schools non-structural and functional vulnerability can lead to severe follow-up losses in the direct aftermath of an event and in the weeks or months to follow.[1]

In this work a study on the seismic vulnerability of existing reinforced concrete structures is presented, with special focus on schools and hospitals in Central America.

The evaluation of seismic risk is an actual problem, many earthquakes all over the word occur each day, so it is important the assessment and the reduction of seismic risk; it can be evaluated as the product with hazard and vulnerability.

This work has two goals:

- do vulnerability analysis for representative buildings of analysed schools and hospitals with a mechanical model;
- check a survey card formulated ad hoc and the correspondent method for vulnerability evaluation based on questionnaires.

For each category (schools and hospitals), representatives buildings are chosen following to surveys done ad hoc in Central America, which have allowed to chose really representative buildings, and to have all necessary information about geometry, structural peculiarities and materials.

Detailed analysis are used to compute structural vulnerability of reinforced concrete existing structures; suitable structural models and the corresponding non-linear lumped plasticity models are generated. The seismic capacity is determined via pushover analysis and by the transformation of the equivalent SDOF capacity curve into bilinear form. The resulting lateral strength and displacement capacity are considered for selected limit states; also the effective period is retrieved. Combining structural capacity with seismic demand through Capacity Spectrum Method, fragility functions are derived for each representative building of schools or hospitals.

1.1. Seismic risk and vulnerability

Worldwide each year, many earthquake occur. According to USGS' study on Historic Earthquakes and Earthquake Statistics, the average of events annually, depending on magnitude, is reported in Table 1.

Magnitude	***Average annually***
8 and higher	1
7 – 7.9	17
6 – 6.9	134
5 – 5.9	1319

Magnitude	***Average annually***
4 – 4.9	13,000 (estimated)
3 – 3.9	130,000 (estimated)
2 – 2.9	1,300,000 (estimated)

Table 1. Earthquake average annually in function of magnitude

The largest recorded earthquake in the world was a magnitude 9.5 in Chile on May 22, 1960. Following the highest earthquakes, with magnitude stronger then 8.5, in the world in the last 100 years, are reported:

Date	***Time (UTC[1])***	***Location***	***Magnitude***
January 31, 1906	15:36	Off the Ecuadorean coast	8.8
November 11, 1922	04:32	Chile – Argentina	8.5
February 03, 1923	16:01	Kamchatka	8.5
February 01, 1938	19:04	Banda Sea, Indonesia	8.5
August 15, 1950	14:09	Assam – Tibet	8.6
November 04, 1952	16:58	Kamchatka	9.0
March 09, 1957	14:22	Andreanof Islands, Alaska	8.6
May 22, 1960	19:11	Chile	9.5
October 13, 1963	05:17	Kuril Islands	8.5
May 28, 1964	03:36	Prince William Sound, Alaska	9.2
February 04, 1965	05:01	Rat Islands, Alaska	8.7
December 26, 2004	00:58	West shore Sumatra	9.1
March 28, 2005	16:09	North Sumatra, Indonesia	8.6

Table 2. Highest earthquakes in the world in the last 100 years

[1] UTC is the acronym for Coordinated Universal Time (National Institute of Standards and Technology)

It is estimated that there are 500,000 detectable earthquakes in the world each year. 100,000 of those can be felt, and 100 of them cause damage.[1]

Two major earthquakes struck beneath the Pacific and Indian Oceans on September 29 and 30, respectively. The first caused a tsunami affecting islands in the Samoan archipelago, including American Samoa, and the subsequent quake hit Sumatra, with both causing major damage; the second one around 1,000 of died. USGS analysts at the National Earthquake Information Centre quickly responded to these events and their many aftershocks, issuing a range of rapid earthquake information products to support emergency response and relief operations. The USGS is the lead federal government agency for earthquake monitoring in the United States and around the globe.

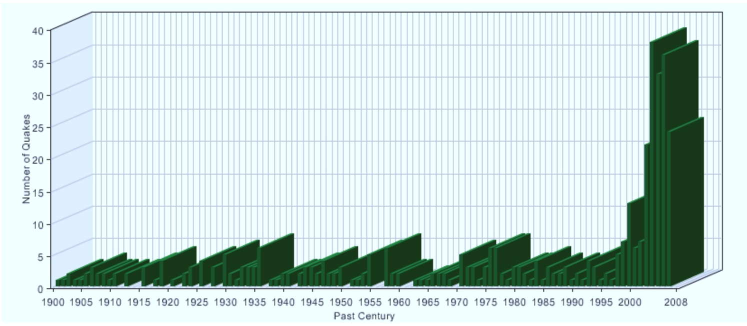


Figure 1. USGS Worldwide Deadly & Destructive Earthquakes between Magnitudes 6 and 8 [3]

In this contest, the present work takes place; in fact, starting from the actual need to reduce the damage caused by earthquakes, moving on various approaches, apply the study to a real case: Central American Countries.

In the last few decades, a dramatic increase in the losses caused by natural catastrophes has been observed worldwide. Reasons for the increased losses are the increase in world population, the development of cities with a population greater than 2 million, located in zones of high seismic hazard, and the high vulnerability of modern society and technologies. The 1994 Northridge (California, US) earthquake produced the highest ever insured earthquake loss, and the 1995 Kobe (Japan) earthquake was the highest ever absolute earthquake loss.[1]

The evaluation of seismic risk to buildings involves many disciplines from data collection to vulnerability assessment to seismic hazard assessment to social and economic sciences. In simple terms, the seismic risk can be described as the probability of loss at a given site and is obtained through the convolution of three parameters: exposure, vulnerability and seismic hazard [4]. A fourth parameter may then be added through which the seismic risk can be related to a social or economic loss; for example, the damage of buildings may be related to the direct economic loss for their repair or replacement, or the collapse of the buildings may be related to the number of casualties. When carrying out seismic risk assessment for a large region, or even a whole country, the exposure is generally obtained from a building census whilst the seismic hazard is described in terms of a ground-motion parameter which should be correlated to the damage of different classes of buildings or other exposed elements through a vulnerability function.[5]

1.2. Vulnerability assessment methods

A good state of art on vulnerability assessment methods, during the past 30 years, is done in Calvi et al. [6]. They report evolution of vulnerability assessment procedures for buildings.

The seismic vulnerability assessment of buildings at large scales has been first carried out in the early 70's, through the employment of empirical methods, based on observed damage after earthquakes, initially developed and calibrated as a function of macroseismic intensities.

In 1973, Whitman et al. proposed the use of damage probability matrices, based on the concept that a given structural typology will have the same probability of being in a given damage state for a fixed earthquake intensity, for the probabilistic prediction of damage to buildings from earthquakes. Braga et al. in 1982, did the first European versions of a damage probability matrix, based on the damage data of Italian buildings after the 1980 Irpinia earthquake.

The use of observed damage data to predict the future effects of earthquakes has the advantage to have a realistic indication of the expected damage when the damage probability matrices are applied to regions with similar characteristics, even if there are various disadvantages associates with empirical methods, as the high number of information needed, uncertainty related to intensity parameters' measure.

An other method based on empirical data is the Vulnerability Index Method (Benedetti and Petrini, 1984; GNDT, 1993), used extensively in Italy , in the past few decades; a vulnerability index give the relationship between the seismic action and the response. The method uses a field survey form to collect information on the important parameters of the building which could influence its vulnerability (configuration in plan and in elevation, type of foundation, structural and non-structural elements, state of conservation and type and quality of materials). Indirect vulnerability index methods allow to determine the vulnerability characteristics of the building. Nevertheless, the methodology still requires expert judgement to be applied in assessing the buildings, and the coefficients and weights applied in the calculation of the index have a degree of uncertainty not generally accounted for.

The use of rapid screening methods has an important role to play in the definition of prioritisation of buildings for seismic retrofit, but the use of such methods in large-scale seismic risk models is limited due to the need to consider buildings individually in a deterministic fashion, so not economically feasible.

The empirical methods are disadvantages; in fact they do not only allow detailed sensitivity studies to be undertaken, but also they need calibration to various characteristics of building stock and hazard. In this context the analytical methods of loss assessment take place.

Although vulnerability curves and damage probability matrices have traditionally been derived using observed damage data.

Singhal and Kiremidjian (1996) developed fragility curves and damage probability matrices for three categories of reinforced concrete frame structures using Monte Carlo simulation. The probabilities of structural damage were determined using nonlinear dynamic analysis with an ensemble of ground motions.

Masi (2003) employed a similar procedure to characterise the seismic vulnerability of different types of reinforced concrete frames designed for vertical loads alone, constructed in Italy over the past 30 years. A simulated design of the structures was carried out with reference to design codes, available handbooks and known practice at the time of construction. The seismic response was estimated through nonlinear dynamic analyses with artificial and natural accelerograms.

However, the derivation of analytical vulnerability curves is a procedure extremely computationally intensive and the curves cannot be easily developed for different areas or countries with diverse construction characteristics; in this context hybrid methods take place.

Hybrid damage probability matrices and vulnerability functions combine post-earthquake damage statistics with simulated, analytical damage statistics from a mathematical model of the building typology under consideration. Hybrid models can be particularly advantageous when there is a lack of damage data at certain intensity levels for the geographical area under consideration and they also allow calibration of the analytical model to be carried out.

Kappos et al. (1995, 1998) have derived damage probability matrices using a hybrid procedure.

The main difficulty in the use of hybrid methods is probably related to the calibration of the analytical results, considering that the two vulnerability curves include different sources of uncertainty and are thus not directly comparable.

Many recent proposals for analytical vulnerability assessment methods use collapse multipliers calculated from mechanical concepts to ascertain whether a mechanism will form and damage will occur.

Cosenza et al. (2005) presented a mechanics-based approach for the assessment of reinforced concrete buildings, which is also based on the formation of collapse mechanisms. First, the seismic capacity of a generic building model is defined. The assumed pre-defined mechanisms are established and the corresponding base shear is calculated assuming a linear distribution of horizontal seismic forces. The ultimate roof displacement is determined as a function of the ultimate rotation of the structural elements. The global seismic behaviour is represented by the base shear coefficient, computed as the ratio between the base shear and the seismic weight, and the corresponding lateral drift, determined as the ratio between roof displacement and building height.

The procedure allows the main parameters, as morphologic and geometric configuration, mechanical properties, to be chosen and their relative influence on the capacity of RC buildings to be evaluated. In fact, a number of models are generated based on the probabilistic distribution of the structural parameters, then a Monte Carlo simulation technique is applied to the calculate probability capacity curves which represent the probability of having a capacity lower than a given threshold value.[7]

A procedure adopted all over the world for loss assessment of urban areas is HAZUS. The methodology in itself has not been adapted in any way, but the capacity curves and fragility functions have been calibrated to the building stock under consideration; in fact it has been used for the loss assessment of Turkey by Bommer et al. (2002), the seismic risk assessment of Oslo by Molina and Lindholm (2005), the loss estimation of Taiwan by Yel et al. (2000) and in the RISK_UE project.

In fact, HAZUS defines the conditional probability of being in, or exceeding a particular damage state d_s given by the spectral displacement S_d (or other seismic demand parameter) with the following equation:

$$P\left[ds \middle| S_d\right] = \Phi\left[\frac{1}{\beta_{ds}} \cdot \ln\left(\frac{S_d}{\overline{S}_{d,ds}}\right)\right] \tag{1}$$

in which:

$\overline{S}_{d,ds}$ is the median value of spectral displacement at which the building reaches the threshold of damage state d_s;

β_{ds} is the standard deviation of natural logarithm of spectral displacement for damage state d_s;

Φ is the standard normal cumulative distribution function.

The standard deviation β_{ds} is computed which the following equation:

$$\beta_{ds} = \sqrt{CONV\left(\beta_C, \beta_D\right)^2, \beta_{tds}{}^2} \qquad (2)$$

where β_C is the standard deviation of natural logarithm of the capacity of the structure for the limit state d_s;

β_D is the standard deviation of natural logarithm of the inelastic demand obtained with the reduction of elastic spectra with a reduction factor (C_R defined by Miranda),

β_{tds} is the standard deviation of natural logarithm of the thresholds' variability.

Giovinazzi in 2005 presented a mechanical procedure for the risk assessment of both masonry and reinforced concrete frames. This uses simplified bilinear capacity spectra, derived using the equations and parameters available in seismic design codes.

The first steps towards the development of a fully displacement-based vulnerability assessment framework can be found in Calvi (1999); the displacements is used as the fundamental indicator of damage and a spectral representation of the earthquake demand is employed. This procedure utilised the principles of the Direct Displacement-Based Design method, wherein a multi-degree-of-freedom (MDOF) structure is modelled as a single DOF System and different displacement profiles are accounted for according to the failure mechanism or displacement profile at a given limit state, while using the geometric and material properties of the structures within a building class. Calvi considered the inherent variability in the structural properties within an urban environment by assigning maxima and minima to the variables and assuming a uniform probability distribution function. The period of vibration was calculated using the empirical formula in EC8 (CEN, 2003) which directly relates the height of a building to its period.

The methodology proposed by Calvi (1999) has subsequently been developed for reinforced concrete buildings by Pinho et al. (2002) and Crowley et al. (2004, 2006), leading to the Displacement-Based Earthquake Loss Assessment (DBELA) procedure.

In Iervolino et al. (2007) a mechanical based procedure is used to obtain capacity parameters C_d, C_s and T for a building class, that allows, from the comparison with demand, the construction of the fragility curves for varying damage levels. [4]

In Polese et al. (2008), the derivation of class representative capacity curves and the relative fragility curves for *slight, moderate, extensive* and *complete* damage states as defined by the well known HAZUS methodology is presented. Starting from an extensive building survey of Arenella district in Naples, statistics on main model input parameters are obtained for selected building classes of existing and/or pre-code RC buildings. Accordingly, a number of building models is simulated designed and analysed in order to determine building class capacity. Fragility curves are computed simulating the fraction of "failures" within a capacity spectrum method framework. The capacity and fragility curves have been used by Lang et al. (2008) for the computation of damage scenarios in Arenella. [8]

Models capable of estimating losses in future earthquakes are of fundamental importance for emergency planners.

Many analytical/mechanical models require a large amount of detailed data, but the benefit of collecting such data is often not proven through validation of the methodology with empirical methods based on the observed damage data. On the other hand, the derivation of vulnerability curves from the observed data does not always consider the frequency characteristics of the buildings stock, and the influence of incorrectly modelling the seismic demand experienced by the buildings in the damaged region is normally unaccounted for. So, the ideal approach for the future needs to be a combination of the positive aspects of different vulnerability assessment methodologies.

1.3. Why focus on high-priority buildings

Among other infrastructure systems, the integrity of schools and hospital buildings during an earthquake disaster is of utmost importance. For hospitals and health centres this holds especially true since these facilities have to remain fully operational in order to protect the lives of patients and health workers as well as to provide emergency care and medical treatment in the aftermath of the disaster. In addition to other particularities, the importance of a hospital to suffer as little damage as possible is increased by the 24/7 occupancy, a high percentage of immobile and highly vulnerable occupants, and the presence of highly sensitive and expensive installations and medical instruments. Damage to these equipments leads to high direct economic losses. The total amount of indirect economic losses e.g. caused by the interruption of hospital network services, in most cases cannot be estimated but may be higher than the direct costs of replacement.

Even though possible direct economic losses caused by earthquake damage to school buildings are comparably low, every effort should be made to increase the seismic safety of schools in order to prevent damage and to protect pupils from harm. To ensure the seismic safety of schools, whose function is to foster and patronize our children, should be among the common responsibilities of any society. Irrespective of the ethical and psychological reasons to care about the seismic vulnerability of schools, there are numerous technical reasons which are regularly corroborated by damage and casualty statistics of strong earthquakes.

Tangshin (China), 1976	>2,000
Spitak (Armenia), 1988	> 1,000
Ardakul (Iran), 1997	110
Cariaco (Venezuela), 2001	46
Molise (Italy), 2002	26
Bingol (Turkey), 2003	84
Ahmedabad (India), 2003	>25
Bachu (China), 2003	>20

Table 3. Number of children died for a school collapse during an earthquake

During the years, a lot of school collapse. According to GeoHazards International, in 1988 Spitak (Armenia) earthquake, death toll among children over 1,000; more children died than adults. As in Lopez, 2004 and GeoHazards International, the 2001 Cariaco (Venezuela) earthquake, five reinforced concrete buildings collapsed, two were schools; most fatalities were in children. In 2005 October earthquake, 18,000 children in Pakistan died while attending school. Following (Table 3), the number of children died for a school collapse during an earthquake, is reported:

Ben Wisner writes that the question is why, again and again, even in developed nations, with a wealth of engineering expertise, schools would collapse in earthquakes. Every school should be inspected and where necessary reinforced. This is so basic to risk mitigation in a seismically active area, it seems foolish to have to write it down.

Children are number one on the public safety agenda; we don't need equations or calculations of cost effectiveness to tell us what our guts already know and millennia of evolution have wired us to feel, there is no greater treasure to a society than its children. Risk mitigation and primitive brainstem response.[9]

In October 2006, UN Secretary General on School Seismic Safety Kofi Annan launches 2 years global campaign to make schools a focal point for disaster reduction:

- "Children are especially vulnerable to the threats posed by natural hazards."
- "Strengthening school buildings and educating students about how to prepare for disasters will save lives."

- "Governments must act now to reduce the devastating impact of disasters on their citizens, especially their children."

In Indonesia, after the big earthquake occurred September 30, 2009, replied in October 01, 2009, the race against time to save thousands of people still buried alive under tonnes of rubbles continues. So far, the quake has killed 1,100 people, injuring an additional 2,400. In Padang, the "natural" epicentre of the quake because of its location on the ring of fire, the situation is desperate. Fernando Abis, a Xaverian missionary of Italian origin, told *AsiaNews* that a "school collapsed" with 50 pupils inside. The hospital built by the Catholic Church was severely damaged, but continues to function, working overtime.[10]

1.4. Work organization

When it comes to disaster mitigation, high-priority buildings require special attention due to the vital functions they performs, their high level of occupancy, and the role they play during a disaster situation.

A vulnerability assessment needs to be made for a particular characterization of the ground motion, which will represent the seismic demand of the earthquake on the building. The selected parameter should be able to correlate the ground motion with the damage to the buildings. Traditionally, macroseismic intensity and peak ground acceleration (PGA) have been used, whilst more recent proposals have linked the seismic vulnerability of the buildings to response spectra obtained from the ground motions.

A study on the seismic vulnerability of existing reinforced concrete structures is presented, with special focus on schools and hospitals in Central America.

An overview of seismic risk problem is done. A survey on seismic hazard in Central America is reported; the vulnerability of existing structures and the principal structural types used in Central American Countries are related.

Vulnerability assessment is presented, with the attention on types of vulnerability and its evaluation with mechanical approach and with a new qualitative methodology based on questionnaires. A vulnerability analysis begins with a visual inspection of the facilities and the preparation of a preliminary report. So high-priority buildings in Central America are selected, and analytical mechanical vulnerability assessment is done for a school in Guatemala City and a hospital in San Salvador. Capacity and fragility functions are described, comparing analytical with qualitative approach reported in Appendix A.

To evaluate vulnerability, a variability in materials strength and in thresholds is considered, to taken into account toughness given by a not simple characterization of materials for existing buildings and the dispersion in rotation values computed with regression formulas.

A lognormal distribution in characterization of elements' mechanical behaviour is assumed, and for the school a different model of roof is considered to take into account the real structural behaviour under loads (in fact, schools have always metal sheet roof). P-delta effects are computed in the approach in case of slender elements.

For what concerns materials, a normal distribution of concrete and steel strength is considered to taken into account problems and mistakes in the evaluation of materials in situ. In fact, as above mentioned, only El Salvador has design codes from 1989, so it is very difficult to found structural plans for buildings previous and for structures in other countries, and it is not easy to define materials' strength. During visual inspection, pacometric tests are done to know steel position and steel percentage in structural elements; but, it was impossible to do the relief of all structural elements, problem embanked with simulated design applied to not investigated elements; rebound hammer tests were made to evaluate the concrete strength value, but it was not possible to do some sample of concrete and steel bars to evaluate.

Fragility curves, in terms of PGA (peak ground acceleration) are derived. The analysis considered three limit states: slight damage, severe damage and near collapse state. Uncertainties in the fragilities account for are those of the surveyed parameters; moreover variability of the limit state thresholds and of inelastic demand are also included.

Chapter 2. Seismic Risk in Central American Countries

Earthquakes have been a constant scourge of mankind. Central America has not escaped this phenomenon, indeed the territory has been most affected by them precisely because of its condition as an isthmus that serves as a fragile union between the continental land masses of North and South America, and in consequence being subject to disturbances by the displacement of the continental plates.

The lands abound with beautiful volcanoes, which have also contributed to local seismic activity. Whatever their origin, the earthquakes that have struck the country have left their share of destruction of lives and property.

In Table 5 a history of seismic activity and volcanic eruptions in the countries on the Caribbean Platform is presented, based on [11] and [12], with the detailed attention on catastrophes that occurred from the XVI Century until 2009. In five centuries the following number of catastrophes originated by earthquake and volcanic eruptions were recorded in Central America:

Century	***No. of catastrophes***
XVI	1
XVII	3
XVIII	2
XIX	9
XX	16
XXI	2
Total	***33***

Table 4. Number of catastrophes originated by earthquake and volcanic eruptions recorded in Central America, in five centuries

XVI CENTURY	
1530 – Venezuela	Partial destructions of New Cadiz by tidal waves.
1541 – Guatemala	Destruction of the old part of the capital by flooding mud from “Agua” volcano.
1543 – Venezuela	Cumaná destroyed in the greater part by earthquake.
XVII CENTURY	
1609 - Nicaragua	Destruction of the capital, León, after the eruption of Momotombo volcano.
1641 - Venezuela	Earthquake near Caracas, causing great damage.
1648 – Nicaragua	Serious earthquake damage in reconstructed León.
1663 – Nicaragua	Total destruction of Le6n. Earthquake damages also in the surrounding area. Numerous landslides.
XVIII CENTURY	
1766 – Venezuela	The most severe earthquake damages in Caracas and Cumaná.
1772 – Nicaragua	Severe earthquake, and an eruption lasting 10 days, of Masaya volcano.
1773 – Guatemala	Severe earthquake damage to the capital Antigua, Guatemala. Move to present location.
XIX CENTURY	
1805 – Venezuela	Cumaná is affected again by an earthquake.
1812 – Venezuela	Seismic catastrophe in Caracas and other cities; more than 10,000 dead.
1822 – Costa Rica	Earthquake almost destroys Cartago completely.
1825 – Colombia	Serious earthquakes north of Barranquilla; destructive tidal waves.
1841 – Costa Rica	Cartago and surrounding area affected by quake. Damages also in Nicaragua.
1844 – Nicaragua	Serious earthquake damage in Rivas and San Juan del Norte.
1859 – Guatemala	Probably one of the strongest quakes felt in Central America. Tidal waves in Pacific coast. Eruption of Izalco volcano.
1867 – Costa Rica	Considerable damages caused by eruption of two volcanos.
1875 – Venezuela	Destructive earthquakes causes more than 15,000 deaths in Cucuta and Tachira.
1881 – Nicaragua	Considerable earthquake damage in Nicaragua.
1882 - Panamá	Panamá City seriously affected by earthquake. Eruption of

	Atrato volcano.
1885 – Nicaragua	Catastrophic earthquake damages in León, Chinandega and Managua.
1898 - Nicaragua	Considerable earthquake damages in León, and also in E1 Salvador.
XX CENTURY	
1900 – Venezuela	Serious earthquake damages in Caracas and its surrounding area.
1902 – Guatemala	Quezaltenango totally destroyed by earthquake. Serious damages throughout the province.
1904 – Costa Rica	Considerable earthquake damages in ample parts of the country.
1904 - Panamá	Strong quake in the Gulf of Panamá.
1906 – El Salvador	A strong earthquake destroys the towns of El Salvador (June 19)
1917/18 – Guatemala	Capital of Guatemala widely destroyed. Enormous material damages, many deaths.
1919 – Costa Rica	An earthquake in Costa Rica destroys the city of Cartago, 20 km east of San Jose, causing around 700 deaths (May 4).
1926 – Nicaragua	A very serious earthquake causes millions in damages in Managua.
1929 – Venezuela	Seismic catastrophe in Cumaná.
1931 – Nicaragua	Again a great part of Managua is destroyed. Damages valued at 15 million dollars. More than 1,000 dead (March 31).
1942 - Guatemala	The major lower crustal earthquake, magnitude 7.9, occurred in western Guatemala. The shock, which was of long duration, caused widespread damage and some destruction along the west-central highlands in Guatemala, where 38 people were killed (August 6).
1950 – Venezuela	Earthquake damage in E1 Tocuyo, Guárico, Anzoategui, Humocaro Alto and Cuaitó.
1951 – El Salvador	Serious earthquake damages in Jucuapa. More than 400 deaths (May 6).
1956 – Nicaragua	Widespread damages in Managua caused by a very strong quake.
1963 – Costa Rica	Eruption of Irazfi volcano. Damages of some US $ 150 Million.
1965 – El Salvador	A magnitude 6.3 earthquake occurred in La Libertad, El

	Salvador. The violent earthquake left 125 people dead, about 500 injured and an estimated 48,000 homeless. Many of the victims were from the town of Ilopango which was almost completely destroyed. Several buildings were wrecked in San Marcos, San Salvador and Santo Tomas (May 3).
1967 – Venezuela	One of the quakes with the most consequences for the country, US $ 100 million in damages and 25o deaths.
1968 – Costa Rica	Eruption of Arenal volcano, 76 victims.
1972 – Nicaragua	10,000 deaths and 20,000 injuries are the budget of an earthquake in Managua, again reduced to rubble (December 23).
1973 – Costa Rica	Considerable damages by earthquake in Tilarán, Rio Chiquito, and Arenal.
1976 – Guatemala	The biggest catastrophe in Central America: more than 23,000 deaths. Material damages of approx. US $ 500 million (February 4).
1986 – El Salvador	More than 1,400 died in San Salvador hit by a violent earthquake (October 10).
1991 – Costa Rica	A magnitude 7.6 earthquake occurred in the coast area between Costa Rica and Panama. Forty-seven people killed, 109 injured, 7,439 homeless and severe damage in the Limon-Pandora area. Some damage also occurred in the San Jose-Alajuela area and landslides blocked roads between Limon and central Costa Rica. Twenty-eight people killed, 454 injured, 2,400 homeless and 866 buildings destroyed in the Guabito-Almirante-Bocas del Toro area, Panama. Slight damage also occurred at David and Puerto Armuelles, Panama. Felt at Colon and at Panama City. Felt in eastern El Salvador and at San Salvador. Also felt in Nicaragua and Honduras and on San Andres Island, Colombia. Maximum uplift of 1.4 meters was observed near Limon and sandblows and liquefaction caused subsidence of soils in the Bocas del Toro area. Ground cracks also occurred in the epicentral area. A 2-meter tsunami with maximum runup of 300 meters was observed in the Cahuita-Puerto Viejo area, Costa Rica. Tsunamis were also reported on Bastimentos, Carenero and Colon Islands and at Portobelo, Panama. The maximum amplitude of the tsunami in Panama was about 0.6 m. A 7-cm tsunami (peak-to-trough) was recorded on the tide gauge at Cristobal, Panama. Damage in Costa Rica estimated to be about 43 million U.S. dollars (April 22).
1992 – Nicaragua	At least 116 people killed, more than 68 missing and over 13,500 left homeless in Nicaragua from a magnitude 7.6

	earthquake. At least 1,300 houses and 185 fishing boats were destroyed along the west coast of Nicaragua. Total damage in Nicaragua is estimated at between 20 and 30 million U.S. dollars. Some damage was also reported in Costa Rica. Most of the casualties and damage were caused by a tsunami affecting the west coasts of Nicaragua and Costa Rica, reaching heights of up to 8 meters (September 2).
1999 – Costa Rica	A magnitude 6.9 earthquake occurred in Costa Rica (August 20).
XXI CENTURY	
2001 – El Salvador	A magnitude 7.7 earthquake occurred. At least 844 people killed, 4,723 injured, 108,226 houses destroyed and more than 150,000 buildings damaged in El Salvador. About 585 of the deaths were caused by large landslides in Nueva San Salvador and Comasagua. Utilities and roads damaged by more than 16,000 landslides. Damage and injuries occurred in every department of El Salvador. Eight people killed in Guatemala. Felt from Mexico City to Colombia. The main shock appears to be located within the Carribean plate above the subducting Cocos plate, and is a normal faulting event. In contrast, the large aftershocks on January 14 and 16 were strike-slip events (January 13).
2001 – El Salvador	At least 315 people killed, 3,399 injured and extensive damage from a magnitude 6.6 earthquake, within the Cocos-Caribbean subduction zone (February, 13).
2004 – Nicaragua	A magnitude 7.0 earthquake occurred near the coast of Nicaragua (October 9).
2004 – Costa Rica	A magnitude 6.4 earthquake occurred in Costa Rica (November 20).
2005 – Nicaragua	A strong earthquake occurred at 02:16:44 (UTC). The magnitude 6.6 event has been located in near the coast of Nicaragua (July 2).
2007 – Guatemala	A 6.7 magnitude earthquake occurred in offshore Guatemala (June 13).
2009 – Costa Rica	A magnitude 6.1 earthquake occurred in Costa Rica. At least 20 people killed in the Cinchona-Dulce Nombre area. Many of casualties were caused by landslides. Many people were injured, several buildings were damaged and landslides blocked roads in the area. Electricity was disrupted in parts of San Jose (January 8).

Table 5. Serious earthquakes and volcanic eruptions during five centuries in Central America and the Caribbean zone: Central America, Northern Columbia, Venezuela

Some of these countries are taking advantage of studies on seismic risk that have been made on behalf of underwriting institutions and government authorities. In the VI Congress of Underwriters of Central America and Panama, that took place in San José, Costa Rica, in November of 1976, aspects related to catastrophic events were discussed, as could well be expected, and among the recommendations was to increase the efforts of the Underwriters Association in each of the countries of the isthmus, before their respective governments, to see that seismological and other studies relevant to risks of a catastrophic nature be made and placed at the disposal of the underwriting companies for their effective use.

Towards this end it would be of great help for these Countries to have the collaboration from developed nations that dispose of greater elements for investigation. The inclusion of this subject in this Colloquium leads them to await valuable contributions that will permit them to get closer to the solution that, for a part of mankind submitted with greater frequency to these catastrophic events, is of vital importance.

2.1. Seismic hazard of Central America

As above-mentioned, Central America is already been upset on several occasions by strong earthquakes. In Table 5 a list of the most devastating shocks that have caused death and destruction in the region.

Figure 2. Seismic Hazard Map, Central America [13]

Connecting North and South America is a curving strip of land or isthmus that makes up the region of Central America. A bridge between Mexico and Colombia, Central America separates the Atlantic and Pacific Oceans. Its countries are Belize, Costa Rica, El Salvador, Guatemala, Honduras, Nicaragua, and Panama. More than 40 million people live in the seven countries. [1]

The ring marks the edges of several of Earth's tectonic plates. The plates' movements against one another create volcanoes and earthquakes. Central America's volcanoes grew out of the pressure and heat created when the Cocos Plate pushed under its neighboring Caribbean Plate—the plate most of Central America sits on. Only some events: Nicaragua had serious earthquakes in 1931 and 1972; two damaging earthquakes hit El Salvador in 2001, and in 2005 the Santa Ana volcano erupted.

All parts of Central American Countries are affected by earthquakes and volcanic activity. Guatemala, El Salvador, Nicaragua and Costa Rica, in fact, are located on the western edge of the Caribbean Plate as shown in Figure 3. The Caribbean plate is a piece of the earth's crust that resembles a small continent, although much of it is covered by the Caribbean Sea. The eastern edge is formed by the Lesser Antilles. The western edge borders the Cocos Plate and forms a portion of Ring of Fire, shown in Figure 4. Ring of fire [16], which dominates the tectonics of the Region. Sea floor spreading of the Cocos Plate to the west and the Caribbean plate to the east apply compressive pressure normal to the Pacific coastline. The Cocos Plate is being forced under the Caribbean Plate (subduction) at a rate of 6-8 cm per year. [14]

Figure 3. Cocos Plate [15]

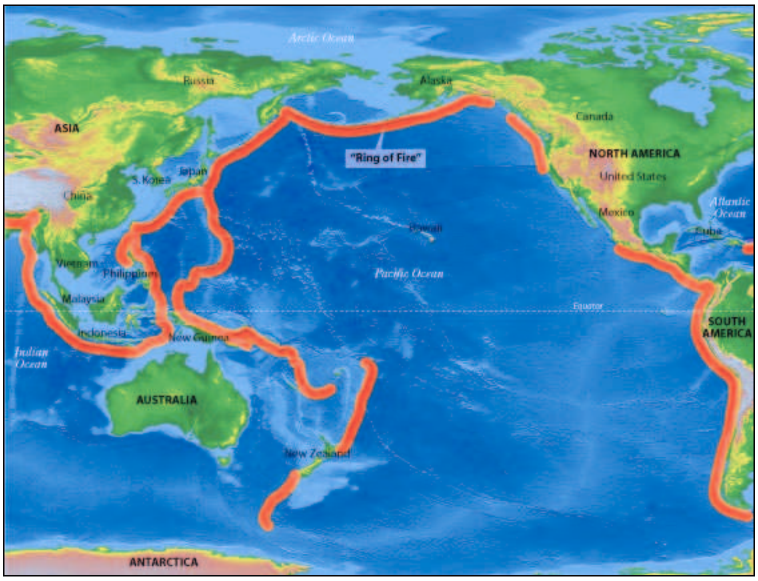


Figure 4. Ring of fire [16]

2.1.1. Guatemala

Guatemala is one of the Central American countries that for some years now have been participating in a regional program for natural hazard assessment and disaster reduction, funded by the Nordic countries and coordinated by a regional institution (CEPREDENAC*). Recent work related to seismic hazard has included the standardization, reporting and processing of seismicity data across the borders, followed by regional hazard modelling. The site specific hazard calculations indicate that expected values of peak ground acceleration are ranging from less than 2 to more than 6 m s^{-2}, corresponding to annual exceedence probabilities ranging from 0.1 to 0.001, respectively.[17]

Guatemala's highlands lie along the Motagua Fault, part of the boundary between the Caribbean and North American tectonic plates. This fault has been responsible for several major earthquakes in historic times, including a 7.5 magnitude tremor on February 4, 1976 which killed more than 25,000 people. In addition, the Middle America Trench, a major subduction zone lies off the Pacific coast. Here, the Cocos Plate is sinking beneath the Caribbean Plate, producing volcanic activity inland of the coast. Guatemala has 37 volcanoes, four of them active: Pacaya, Santiaguito, Fuego and Tacaná. Natural disasters have a long history in this geologically active part of the world. For example, two of the three moves of the capital of Guatemala have been due to volcanic mudflows in 1541 and earthquakes in 1773.[18]

2.1.2. El Salvador

The republic of El Salvador in Central America is an area of high seismic hazard where at least twelve destructive earthquakes have occurred this century alone. The principal sources of seismic hazard are earthquakes associated with the subduction of the Cocos plate in the Middle America Trench and upper-crustal earthquakes in the chain of Quaternary volcanoes that runs across the country parallel to the subduction trench. Hazard assessments for Central America have suggested almost uniform distribution of hazard throughout El Salvador. Seismic zonations for three successive building codes in El Salvador simply divide the country into two regions, with the higher hazard zone containing the volcanoes and the coastal areas. Historical records suggest that the greatest hazard is posed by the upper-crustal earthquakes concentrated on the volcanic centres which, although of smaller magnitude than the subduction events, are generally of shallow focus and coincide with the main population centres. These earthquakes have repeatedly caused intense damage over small areas in the vicinity of some of the main volcanoes. [19]

El Salvador lies along the Pacific Ring of Fire, and is thus subject to significant tectonic activity, including frequent earthquakes and volcanic activity. Recent examples include the earthquake on January 13, 2001, that measured 7.7 on the Richter scale and caused a landslide that killed more than eight hundred people; and another earthquake only a month after the first one, February 13, 2001, killing 255 people and damaging about 20% of the nation's housing. Luckily, many families were able to find safety from the landslides caused by the earthquake. The San Salvador area has been hit by earthquakes in 1576, 1659, 1798, 1839, 1854, 1873, 1880, 1917, 1919, 1965, 1986, 2001 and 2005. The 5.7 M-earthquake of 1986 resulted in 1,500 deaths, 10,000 injuries, and 100,000 people left homeless.

Even so, El Salvador is subject to other natural disasters. In fact, El Salvador's most recent destructive event took place on October 1, 2005, when the Santa Ana Volcano, currently dormant, spewed up a cloud of ash, hot mud and rocks, which fell on nearby villages and caused two deaths. The most severe volcanic eruption in this area occurred in the 5th century A.D. when the Ilopango erupted, producing widespread pyroclastic flows and devastating Mayan cities. El Salvador's position on the Pacific Ocean also makes it subject to severe weather conditions, including heavy rainstorms and severe droughts, both of which may be made more extreme by the El Niño and La Niña effects. El Salvador's location in Central America also makes it vulnerable to hurricanes coming off the Caribbean, however this risk is much less than for other Central American countries.[20]

2.1.3. Nicaragua

Nicaragua can be divided into two distinctive geographies. The country's eastern portion is an irregular upland composed of tertiary volcanoes and pyroclastics. The average altitude in the highlands is about 500 meters with peaks reaching 1,000 meters and the tallest peaks reaching 1,500 - 2,000 meters. The western pacific coast region contains a long valley called the Nicaraguan Graben. The Graben extends from the Pacific Ocean at the Gulf of Fonseca into Costa Rica where it joins with the Costa Rican Coastal Plain. The Graben contains a boundary fault nearly parallel with a string of volcanoes called "cordillera de Marrabios"[14]

2.2. Vulnerability of existing structures

In recent times, many parts of the world have seen a trend of increased construction with reinforced concrete and masonry block systems. These systems can provide excellent seismic resistance when they are designed by an engineer, built by well-trained workers, constructed of quality materials and all in conformance with building codes. Unfortunately, many structures are constructed without one or more of these requirements.[14]

In this work, attention will be focus on reinforced concrete structures.

Many features affect structural vulnerability. First of all, irregularity in plan and in height: irregularities in the buildings layout will result in eccentricities of mass and stiffness center, causing torsional effects and large displacements, often reduced by the presence of seismic joint, moreover interruption of lateral stiffness over the buildings height will cause weak locations in the structural system, as soft storey, short columns.

Inadequate constructive details can cause a bad performance of structural system, so an increase of structural vulnerability. Some critical factors are: geometrical and mechanical percentage of steel in elements' cross-section; beam-column joints in which the lacking confinement can causes shear crises.

The absence of structural capacity design is a cause of fragile collapse mechanism as shear brittle failure of structural elements, failure in beam-column joints, foundations' plasticity. So, it is important to follow, during the design, fundamental criteria of capacity design: strength hierarchy and ductility rules, to avoid brittle failures in favour of global structural failure.

From a historical perspective, a code by itself cannot guarantee safety from excessive damage, since codes are rules that establish minimum requirements, which are continually updated in accordance with technological advances and lessons learned through research and study of the effects of earthquakes. Ductility (i.e., energy absorption capacity) and structural redundancy have proven to be the most effective means of providing safety against collapse, especially if the movements are more severe than those anticipated by the original design. Severe damage or collapse of many structures during major earthquakes is, in general, a direct consequence of the failure of a single element or series of elements with insufficient ductility or strength.

Structural damages as a result of strong earthquakes are frequently found in columns, including diagonal cracks caused by shearing or twisting, vertical cracks, detachment of column sheathing, failure of concrete, and warping of longitudinal reinforcement bars by excessive compression. In beams, diagonal cracks and breakage of supports due to shearing or twisting are often seen, as are vertical cracks, breakage of longitudinal reinforcements, and failure of concrete caused by the earthquake flexing the section up and down as a result of alternating stresses.

As above mentioned, connections or unions between structural elements are, in general, the most critical points. In beam-column connections (ends), shearing produces diagonal cracks, and it is common to see failure in the adherence and anchorage of the longitudinal reinforcements of the beams because of their poor design or as a consequence of excessive flexural stress.

In the slabs, cracks may result from punctures around the columns, and longitudinal cracks along the plate due to the excessive flexure that earthquakes can cause in certain circumstances. This type of damage has been seen repeatedly in hospital facilities submitted to moderate to strong seismic movements.

Irregularities in height, translated into sudden changes in stiffness between adjacent floors, concentrate the absorption and dissipation of energy during an earthquake on the flexible floors where the structural elements are overburdened. Irregularities in mass, stiffness, and strength of floors can cause torsional vibrations, concentrating forces that are difficult to evaluate. For this reason, a higher standard for these elements must guide the designers entrusted with the design of these buildings.

Few buildings are designed to withstand severe earthquakes in the elastic range, so it is necessary to provide the structure with the ability to dissipate energy through stiffness and ductility, in the places where it is expected that elastic strength may be exceeded. This is applied to structural elements and connections between these elements, which are usually the weakest points.[21]

2.3. Relevant seismic code

Seismic engineering is one of the most rapidly evolving disciplines in the civil/structural engineering profession. Recent seismic events around the world have provided new insight into the way structures perform when subjected to earthquake related ground motion. These events have focused the attention of government agencies, code bodies, insurance companies, the scientific community and the general public on safety hazards and potential losses associated with structures that perform poorly during earthquakes. As a result, there is growing national emphasis on seismic risk assessment, seismic design requirements for new structures, and seismic retrofit of existing structures. Seismic provisions of model building codes have been extensively revised in recent years; many communities have adopted certain mandatory seismic upgrade requirements, and at least one state has instituted specific earthquake related licensing requirements for professional engineers.

Many structural engineers have limited experience concerning the behaviour of structures subjected to strong ground motion. In addition, most building code seismic design provisions are prescriptive in nature and provide little or no insight into actual structural performance.

Following, some reference codes used in the work are briefly reported: the first dedicated code for hospitals in Central America, introduced in El Salvador; the International Building Code used by Central American engineers often to remedy the absence of national code and the Eurocode 8 which is one of the more relevant code in the world, used in the following for assessment.

2.3.1. El Salvador Hospital Code 2004

The Department of Health in El Salvador introduced in 2004 a code provision for hospitals and health centres. It is clearly inspired of American Codes (e.g. American Concrete Institute, American National Standard Institute, American Society of Mechanical Engineers, American Standard of Testing and Materials, Federal Emergency Management Agency).

Indications on architectural aspects are reported with special focus on evacuation system. To evaluate geotechnical aspects, how do the site conditions study is reported. Structural aspects are considered in the section on structural design; the indications are not so exhaustive even if it is appreciable the first approach with a code provision only for a building typology. Loads' analysis is very clear and complete.

This code show an important consciousness of Central American engineers have about seismic joint; even if they design elongated structures, they never make irregular plan shape, they interrupt the shape with seismic joint, often under proportioned.

Finally, the large part of code is dedicated to non-structural components and medical equipments.

It is appreciable the attempt of El Salvador to have a code only for hospitals and health centres; it is the first in all Central American Countries.

2.3.2. IBC 2006 – ACI

Internationally, code officials recognize the need for a modern building code addressing the design and installation of building systems through requirements emphasizing performance.

The International Building Code 2006 established minimum regulations for building systems using prescriptive and performance-related provisions. It is founded on broad-based principles that make possible the use of new materials and new building designs.

This code is founded on principles intended to establish provisions consistent with the scope of a building code that adequately protects public health, safety and welfare; provisions that do not unnecessarily increase construction costs; provisions that do not restrict the use of new materials, products or methods of construction; and provisions that do not give preferential treatment to particular types or classes of materials, products or methods of construction.

The International Building Code is available for adoption and use by jurisdictions internationally.[22]

2.3.3. EC8

Eurocode 8 explains how to make building and civil engineering structures resistant to earthquakes. It examines seismic actions and defines rules for buildings and bridges; it investigates strengthening and repair of buildings; it analyzes silos, tanks and pipelines, foundations, retaining structures and geotechnical aspects, towers, masts and chimneys.

It contains:

1. additional provisions for the structural design of buildings and civil engineering works to be constructed seismic regions where risk to life and risk of structural damage are required to be reduced;

2. general requirements and rules for assessment of seismic actions and combinations with other actions;
3. general rules for earthquake-resistant design of buildings and specific rules for buildings and elements constructed with each of the various structural materials;
4. design rules for earthquake-resistant design of steel, concrete and composite bridges;
5. guidelines for the evaluation of the seismic performance of existing structures, the selection of corrective measures and the design of repair and strengthening measures with additional considerations for monuments and historic buildings;
6. design rules for the earthquake-resistant design of groups of silos, storage tanks including single water towers and pipeline systems;
7. additional rules for the design of various foundation systems, earth-retaining structures and soil-structure under seismic actions in conjunction with the structural design of buildings, bridges, towers, masts, chimneys, silos, tanks and pipelines;
8. design rules for the earthquake-resistant design of tall, slender structures: tower, masts and chimney and lighthouses constructed in reinforced concrete or steel.[23]

Chapter 3. Vulnerability Assessment

Seismic risk assessment is an essential first step to seismic hazard reduction for a large structural inventory. Components of seismic risk assessment are hazard analysis; local site effects; exposure information (structural inventory); vulnerability analysis; estimation of risk.

The standard definition of risk is the probability or likelihood of damage and consequent loss to a given element at risk, over a specified period of time. It is important to note the distinction between risk and vulnerability. Risk combines the expected losses from all levels of hazard severity, also taking their occurrence probability into account, while vulnerability of an element is usually expressed for a given hazard severity level. Loss is defined as the human and financial consequences of damage, including injuries or deaths, the costs of repair, or loss of revenue. The distinction between risk and loss is often very loose and, based on their definition, these terms are sometimes used interchangeably. Since the standard definition of risk is a probability or likelihood of loss, between zero and one, it may be more appropriate to express risk as

$$\mathit{Risk} = \mathit{Hazard} \times \mathit{Vulnerability} \qquad (3)$$

So, vulnerability plays an important role into the risk assessment. Vulnerability can simply be defined as the sensitivity of the exposure to seismic hazard. In fact, vulnerability analysis reveals the damageability of the structure under varying intensity or magnitudes of ground motion. Multiple damage states are typically considered in the analysis.[24]

3.1. Types of vulnerability

In the vulnerability assessment, three kinds of vulnerability can be considered: structural, non-structural and operational.

3.1.1. Structural

Structural vulnerability refers to the susceptibility of those parts of a building that are required for physical support when subjected to an intense earthquake or other hazard. This includes foundations, columns, supporting walls, beams, and floor slabs.

Strategies for implementing disaster mitigation measures in hospital facilities will depend on whether the facilities already exist or are yet to be constructed. The structural components are considered during the design and construction phase when dealing with a new building, or during the repair, remodelling, or maintenance phase of an existing structure.

Unfortunately, in many Latin American countries, earthquake-resistant construction standards have not been effectively applied, and special guidelines have not been considered for hospital facilities. For this reason, it is not surprising that each time an earthquake occurs in the region, hospitals figure among the buildings most affected, when they should be the last to suffer damage. The structural vulnerability of hospitals is high, a situation that must be totally or partially corrected in order to avoid enormous economic and social losses, especially in developing countries.

Since many hospital facilities are old, and others have neither been designed or built to seismic resistant standards, there are doubts as to the likelihood of these buildings continuing to function after an earthquake. It is imperative to use vulnerability assessments to examine the ability of these structures to withstand moderate to strong earthquakes.

Experience of seismic activity in the past shows that in countries where design meets good seismic-resistant standards, where construction is strictly supervised, and where the design earthquake is representative of the real seismic risk to the area, damage to infrastructure is marginal in comparison to that observed in locations where such conditions are not met.

3.1.2. Non-structural

Non-structural elements such like infill walls, equipment, installations or furniture may not influence the building's stability but it may put people and the contents of the building at risk and increases the follow-up losses during evacuation. A building may remain standing after a disaster, but be incapacitated due to non-structural damages. Assessment of non-structural vulnerability seeks to determine the damage that these elements may suffer when affected by moderate earthquakes, which are more frequent during the life of a hospital. Due to the high probability of earthquakes that could affect the non-structural components, necessary steps must be taken to protect these elements.

The cost of non-structural elements in most buildings is appreciably higher than that of structural elements. This is particularly true in hospitals, where between 85% and 90% of the facility's value resides in architectural finishes, mechanical and electrical systems and the equipment and supplies contained in the building. A low-magnitude seismic event can affect or destroy vital aspects of a hospital, those directly related to its function, without significantly affecting the structural components. It is easier and less costly to apply damage mitigation measures to non-structural elements.

It is not enough for a hospital to simply remain standing after an earthquake; it must continue to function. The external appearance of a hospital might be unaffected, but if the internal facilities are damaged, it will not be able to care for its patients. This section focuses on preventing loss of function due to non-structural failure, which may also affect the integrity of the structure itself.

The design of any structure subjected to seismic movements should consider that non-structural elements such as ceilings, panels, partition walls, windows, and doors, as well as equipment, mechanical and sanitation installations, must withstand the movements of the structure. Moreover, it should be noted that the excitation of the non-structural elements, caused by movements of the structure, is in general greater than the excitation at the foundation of a building, which means, in many cases, that the safety of the non-structural elements is more compromised than that of the structure itself.

Notwithstanding the above, little attention is generally paid to these elements in the seismic design of structures, to the extent that many design codes do not include standards for non-structural components. This is evident in the experience of recent earthquakes where structures designed in accordance to modern seismic-resistance criteria performed well, but unfortunately there was a deficient response of the non-structural elements. If the safety of the occupants of a building, replacement costs, and the losses involved in interrupting the operations of the building itself are taken into account, the importance of seismic design of the non-structural elements can be understood.

In the case of hospitals, the problem is of major importance for the following reasons:

1. Hospital facilities must remain as intact as possible after an earthquake due to their role in providing routine medical services as well as attending to the possible increase in demand for medical treatment following an earthquake.

2. In contrast to other types of buildings, hospitals accommodate a large number of patients who, due to their disabilities, are unable to evacuate a building in the event of an earthquake.
3. Hospitals have a complex network of electrical, mechanical and sanitary facilities, as well as a significant amount of costly equipment, all of which are essential both for the routine operation of the hospital and for emergency care. Failure of these installations due to a seismic event cannot be tolerated in hospitals, as this could result in the functional collapse of the facility.
4. The ratio of the cost of non-structural elements to the total cost of the building is much higher in hospitals than in other buildings. In fact, while non-structural elements represent approximately 60% of value in housing and office buildings, in hospitals these values range between 85% and 90%, mainly due to the cost of medical equipment and specialized facilities.

Experience shows that the secondary effects caused by damage to non-structural elements can significantly worsen the situation. For example, ceilings and wall finishes can fall into corridors and stairways and block the movement of occupants; fires, explosions and leaks of chemical substances can be life-threatening. The functions of a hospital are dependent on such basic services as water, power and communications. Damage or interruption of these services can render a modern hospital virtually useless.

Nagasawa[2] describes that, as a result of the Kobe, Japan, earthquake in 1995, a significant number of hospitals reported damage due to falling shelves, movement of equipment with wheels without brakes or that were not in use, and falling office, medical and laboratory equipment that was not anchored down. In some cases, even heavy equipment such as magnetic resonance, computerized axial tomography and X-ray equipment moved between 30 cm and 1 m, and equipment hanging from ceilings, such as an angiograph, broke away from its supports and fell, in turn damaging other important equipment.

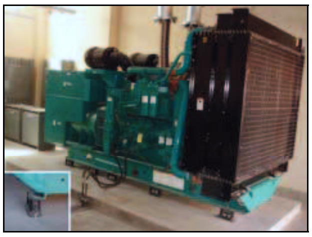
Non-structural elements can be classified in the following three categories: architectural elements, equipment and furnishings and basic installations.

- The architectural elements include components such as non–load-bearing exterior walls, partition walls, inner partition systems, windows, ceilings, and lighting systems.

[2] Nagasawa, Y., Damages caused in hospitals and clinics by the Kobe earthquake, Japan. *Japan Hospital* No. 15.

- The equipment and furnishings include medical and laboratory equipment, mechanical equipment, office furnishings, medicine containers, etc..
- The basic installations include supply systems such as those for power and water, networks for medical gases and vacuum, and internal and external communications systems.[21]

Following (Figure 5) examples of non-structural features that affect the non-structural vulnerability are reported.


(a) Emergency generator mounted with undersized bolts (H)


(b) Flexible connection of pipes (H)


(c) Non-maintained hose-reel cabinet (H&S)


(d) Insufficient securing of suspended ceilings (H&S)


(e) Loosened façade claddings (H&S)


(f) Inadequately secured gas cylinders (H)

3.1.3. Operational

Of all the elements that interact in the day-to-day operations of a hospital, the administrative and organizational aspects are among the most important in ensuring that disaster prevention and mitigation measures are adopted before a disaster strikes, so that the building (especially hospital) can continue to function after an earthquake or other catastrophic event.

Functional and operational vulnerability to emergencies and disasters can be analyzed at two different levels. The macro level involves studying the resolution capacity of health facilities, which is based on currently popular concepts of health services modernization and decentralization. This type of analysis is ambitious: its final objective is the implementation of a total quality management policy for health services. Continually improving the quality of a health facility's services automatically brings about improvements in the structural, non-structural, and functional conditions of day-to-day operations, leading to a hospital that performs more effectively, as a whole, in the event of an emergency or disaster. However, such an analysis lies beyond the scope of this book.

Instead, the micro-level is normally focuses only on those aspects relevant to a particular health establishment. However, it is possible to draw on the information available from several health facilities, to carry out a micro-level analysis of the administrative and organizational vulnerability of a fairly typical hospital. This includes those operational aspects that might have a negative impact on its ability to provide its services both in normal and in external or internal emergency conditions, as we will see in greater detail below. In order to do this, it is necessary to examine the activities carried out in the different departments of a hospital, their interactions, the availability of basic public services, and the modifications required in the event of an emergency.

Similarly, we will perform a critical review of a typical hospital emergency plan, seen as another administrative and organizational tool, in order to identify its possible weaknesses and underscore the useful components related to guaranteeing the functionality of existing services. It is important to stress that a hospital emergency plan, no matter how well crafted, will be useless if the building suffers serious damage to its physical infrastructure. Accordingly, this analysis is based on the assumption that structural and non-structural deficiencies have been corrected or, if this has not yet been accomplished, that they have at least been identified and the emergency plan has taken them into account.

In the event of a disaster, a hospital must be able to continue caring for its inpatients while treating victims of the event, safeguarding all the while the lives and health of its personnel. For this to happen, the staff must be deployed effectively and know exactly how to respond to such a situation. The building and its equipment, supplies and lifelines must remain operational. Most hospital authorities recognize this fact, which is why they have established formal disaster mitigation plans.

However, most of these plans fail to provide administrative and organizational alternatives in the event of severe damage to the facilities. The issue has received little attention. This is worrisome, particularly in the many locations throughout the Americas where the population only has ready access to one hospital that, if rendered inoperative, could lead to a severe health crisis.

A systematic approach, which takes into account the fluid movement of staff, equipment and supplies in a safe environment during normal operations, is vital if an effective response to disasters is to be in place. This underscores the critical nature and interdependence of the various processes, buildings, and equipment. Deficiencies in any of these areas can plunge a hospital into a crisis.

1. Processes: They mostly have to do with the movements of people, equipment and supplies. They also include routine administrative processes such as hiring, acquisitions, human resource management, and the flow of patients through the various clinical and support service areas of the hospital.
2. Buildings: Experience has shown that the design and construction of hospital buildings, as well as their future expansion and remodeling, their everyday operations and maintenance, must be safety-oriented to protect certain critical hospital operations such as emergency care, diagnosis and treatment, surgery, pharmaceutical supplies and food storage, sterilization, patient registration, reservations, or any other areas the institution considers a high priority.

In hospital design, emphasis must be placed on the optimal use of space and the configuration of the services provided, so that the different departments and activities can mesh together with the greatest possible efficiency and the lowest vulnerability. Many facilities have suffered a functional collapse as a result of simple omissions during their design, which could have been easily corrected or addressed at a marginal cost during construction or retrofitting.

To preserve equipment, regular inspections and the proper maintenance can ensure that key and often costly hospital equipment can remain in good working order.

As discussed earlier, it is the duty of the authorities to assess the hospital's vulnerability to natural phenomena and obtain precise estimates of existing risk levels. Once the analysis is complete, the information gathered should be used to determine what level of risk is acceptable. In the case of administrative and organizational vulnerability, the analysis can start with a visual inspection of the facilities and the drafting of a preliminary assessment report identifying key areas that demand attention, alongside a study of administrative procedures, their critical points, and their flexibility in emergency situations.[21]

3.2. Vulnerability assessment based on questionnaires

Compared with buildings of residential or commercial occupancy, buildings like hospitals or schools are of high priority in case of a natural disaster and thus should be given special attention.

The detailed structural analysis of any building requires a multitude of input information on the buildings layout, its detailing (e.g. reinforcement) or construction material properties. These data can only be obtained if sufficient constructional drawings are available and material testings are conducted which derive reliable estimates of material properties. Since these investigations cannot be performed for a larger number of buildings, it is proposed an alternative procedure which allows a quick assessment of the structure's actual vulnerability. Through the application of standardized questionnaires, structural and non-structural vulnerability indexes are derived which allow a priority ranking and an identification of the most vulnerable structures so that responsible authorities are able to conduct a more targeted investigation using more advanced investigation methods.

In contrast to other approaches (e.g. the 'Hospital Safety Index' initiative by the Pan American Health Organization, 2008) the structural and non-structural vulnerability are treated separately. While the structural vulnerability index is generated taking into account main design failures as well as the age of the building and its general state of maintenance, the non-structural vulnerability index covers all types of installations, secondary structural elements as well as their impact on the functionality of the building. To optimize a realistic selection of survey questions, the questionnaires have been tested to numerous hospitals and school buildings in the Central American countries Guatemala, Nicaragua, El Salvador and Panama. Based on the results of these pre-studies and the experiences gained during these case studies, a calibration of the questionnaires was done through the definition of reliable weighting factors for the different vulnerability-affecting aspects.[1]

Name (ID):		Occupancy:	☐ Hospital ☐ Health Center ☐ other: _______________
Address: Contact:		No. of:	☐ inhabitants/occupants: ________ ☐ beds: ________ ☐ patients: ________ ☐ medical staff/employees: ________
Coordinates:	Latitude ___________ Longitude ___________	Occupancy period:	☐ 24 h ☐ 12 h ☐ 8 h ☐ morning ☐ afternoon from: ______ to: _______
Structural characteristics:	Typology of the primary structure: ____________________________________ ____________________________________ no. of individual buildings: _____ no of stories (basements): ____ (___) interstory height: _____ *m* no. of cores: _____ plan shape: ☐ □ ☐ L ☐ U ☐ T max. length L: _____ *m* max. width W: _____ *m*	Age:	☐ < 10 *years* ☐ 10 – 20 *years* ☐ 20 – 40 *years* ☐ > 40 *years* ☐ year of construction: _______
		Actual state:	☐ good (new) ☐ recently renovated ☐ in need of renovation ☐ bad (decayed)
		Maintenance program:	☐ exists ☐ does not exist if yes, in which period: _______________
Photo ID's: Screener/date:		Topography:	☐ plane (flat) ☐ sediment basin (valley) ☐ close to river ☐ foothill (base of slope) ☐ slope situation ☐ ridge (top of slope; hilltop)

Table 6. "General information" part of the hospital questionnaire

No.	FEATURES AFFECTING THE STRUCTURAL SEISMIC VULNERABILITY	RC			URM		
		YES	NO	NA	YES	NO	NA
01	Is the building irregular in plan?	8	0		10	0	
02	Are the columns regularly distributed?	0	4				
03	Are both building directions adequately braced (RC frames or shear walls, URM walls)?	0	16		0	20	
04	Does the ratio between the building's length and width is > 2.5 ?	4	0		10	0	
05	Does the building possess eccentric cores (staircases or elevators)?	8	0		10	0	
06	Does the building have a soft story?	16	0	0			
07	Is the building irregular in elevation caused by setbacks of upper stories?	8	0	0	20	0	0
08	Does the building have cantilevering upper stories?	8	0	0	10	0	0
09	Does the building possess a heavy mass at the top or at roof level?	4	0		5	0	
10	Are pounding effects possible?	4	0		5	0	
11	Does the building have short columns?	8	0				
12	Are strong beams–weak columns available?	16	0				
13	Does the building possess shear walls ?	0	4				
14	Did the building suffer any significant structural damage in the past?	4	0		5	0	
15	Does the building possess seismic retrofitting or strengthening measures?	0	8		0	5	
	SUM			max 120			max 100
	NO. OF ANSWERED QUESTIONS			12 or 15			8 or 10
	STRUCTURAL VULNERABILITY INDEX *SVI* (= Sum ÷ No. of questions)						

Table 7. "Structural seismic vulnerability" part of the hospital questionnaire

No.	FEATURES AFFECTING THE NON-STRUCTURAL SEISMIC VULNERABILITY	YES	NO	NA
I. Electrical Facilities				
01	Is there an **emergency generator** and **fuel tank** available?	0	16	
02	If yes, are both located outside the building? (if Q01 = NO → NA)	0	16	0
03	If outside, in a certain distance such that e.g. parts of the building can not fall on them? (if Q01 = NO → NA)	0	8	0
04	Are they adequately secured? (if Q01 = NO → NA)	0	8	0
05	Are **service lines** and **other pipes** attached with flexible connections?	0	16	
06	Are they able to accommodate relative movement across joints?	0	16	0
07	Are **bus ducts** and **cables** able to distort at their connections to equipment without rupture?	0	8	
08	Are they able to accommodate relative movement across joints?	0	8	
II. Fire Fighting				
09	Are there **smoke detectors** and **alarms** available?	0	4	
10	Are there enough **fire extinguishers** and **hose-reel cabinets** available?	0	16	
11	Are they easily accessible? (if Q10 = NO → NA)	0	16	0
12	Is the **emergency water tank** located outside the building?	0	16	
13	If located outside, can it collapse or be damaged during an earthquake by falling parts? (if Q12 = NO → NA)	8	0	0
III. Propane pipes or any other gas pipes (e.g., oxygen)				
14	Does the system have an automatic, earthquake-triggered **shut-off valve**?	0	16	
15	If not, can it be easily closed manually e.g. by a wrench tool stored close by? (if Q14 = YES → NA)	0	16	0
16	Are **supply pipes** able to accommodate relative movement across joints and at the tank?	0	16	
17	Are **supply pipes** able to distort at their connections to equipment without rupture?	0	16	
IV. Elevators				
18	Are elevators available?	4	0	
19	Are elevators maintained and are they regularly (every 2 months) controlled? (if Q18 = NO → NA)	0	4	0
20	Are **motors** and **control cabinets** anchored to the floor? (if Q18 = NO → NA)	0	4	0
V. Non-structural Infill Walls and Partitions				
21	Are (infill) brick walls protected against out-of-plane failure by e.g. internal reinforcement or surface meshes?	0	8	
22	Do movement joints between brick infill walls and RC frames exist to allow damage-free movement? (for masonry buildings → NA)	0	8	0
VI. Ceilings				
23	Are suspended ceilings available?	4	0	
24	Are the suspended ceilings adequately secured against failure? (if Q23 = NO → NA)	0	4	0
VII. Emergency Exits and Escape Routes				
25	If **exit fire doors** jam in an earthquake, is there a crowbar or sledge hammer readily available to facilitate emergency opening?	0	16	
26	Do all **exit doors** open outwards?	0	16	
27	Are all doors unlocked from the inside and also unblocked?	0	16	
28	Are **automatic doors** available?	8	0	
29	Do automatic doors have manual overrides? (if Q28 = NO → NA)	0	8	0
30	Has the **glazing of windows** been designed to accommodate lateral movement?	0	4	
31	Do large windows, door transoms and skylights have safety glass?	0	4	
32	Are **emergency exits** and **escape routes** adequately designated, e.g. by fluorescent signs?	0	8	
33	Are emergency exits and escape routes adequately illuminated?	0	8	

No.	FEATURES AFFECTING THE NON-STRUCTURAL SEISMIC VULNERABILITY (cont'd)	YES	NO	NA
VIII. Appendages				
34	Can nonstructural elements (e.g. parapets, facade cladding, roof tiles, chimneys, external AC machines) fall from the building and harm people running outside?	8	0	
IX. Movable Equipment				
35	Are **gas cylinders** tightly secured with chains at top and bottom (or otherwise)?	0	8	
36	Are **chemicals** stored in accordance with manufacturers recommendations?	0	4	
37	Are **cabinets for hazardous materials** given special attention with respect to anchoring?	0	8	
X. Appurtenant structures				
38	Are there enough **open spaces** around the building which can be used as escape routes and where people are safe from falling objects?	0	16	
39	Can **neighboring structures** (e.g. buildings, walls, electricity lines) block escape routes or harm people running/gathering outside?	8	0	
40	Can **road access** to and from the hospital be blocked due to collapse of buildings or geotechnical effects (slope failure, landslide etc.)?	8	0	
	SUM			max 404
	NO. OF ANSWERED QUESTIONS			max 40
	NON-STRUCTURAL VULNERABILITY INDEX *NVI* (= Sum ÷ No. of questions)			

Table 8. "Non-structural seismic vulnerability" part of the hospital questionnaire

Based on a number of available manuals, guidelines and provisions which were mainly developed for the assessment of health facilities (PAHO 2000a, 2000b, 2006; FEMA 2003, 2004, 2005; WHO 2002, 2007; NRCC, 1993), a simplified procedure was derived in order to quickly assess the seismic vulnerability of hospitals and school buildings.

The assessment procedure is based on standardized questionnaires which consist of three different parts:

Part 1: General Information (partly different for hospital and school assessment);

Part 2: Structural Vulnerability;

Part 3: Non-Structural Vulnerability.

While Part 1 and Part 3 are customized to the particular differences of schools and hospitals and thus address different issues and questions, Part 2 is equal for both building types.

It should be regarded, that especially in Part 1 of the questionnaires (general information) not all information required will be used for the calculation of the vulnerability factors. Most of this information will more have statistical purpose.

The first part of the questionnaire (Table 6 and Table 9) covers general information on the facility, such as:

- name, address and geographic coordinates,
- basic structural characteristics,
- occupancy characteristics,
- age, actual state and maintenance status of the building,

- topography.

The parameters “age” and “actual state” of the building are linked with factors which will be added to the structural vulnerability indexes.

The second part of the questionnaire includes a number of questions which target the structural vulnerability of the respective building.

The selection of the 15 different questions was done with regard to existing screening procedures and provisions (FEMA 154; PAHO, 2006).

The questionnaire are explicitly designed for masonry and reinforced-concrete buildings, because the majority of the buildings to be observed cover these construction materials. A general subdivision is done for masonry and reinforced-concrete structures since some of the questions may not be applicable to masonry buildings and are accordingly highlighted. Further, some questions may not be applicable to single-story structures and need thus be answered by checking *NA* (not applicable).

Table 7 and Table 10 reproduce the 15 questions which are used to assess the structural seismic vulnerability and gives their levels of importance which are dependent on the construction type. Each level of importance is connected to a certain importance factor which is summed up if the respective answer increases the building’s vulnerability. The structural vulnerability index *SVI* is then derived by dividing the sum of importance factors by the number of answered questions. The number of answered questions is reduced if one or more questions cannot be answered and are thus not applicable (*NA*).

Once the structural vulnerability index *SVI* is derived it is multiplied with both age factor *AF* and actual state factor *ASF* to (*SVI*). Both indexes are provided in order to allow a more transparent estimation of the indexes and how age and/or actual state of the building influence its vulnerability index.

Table 11 list the decided values for both factors and the percentile increase of structural vulnerability factor SVI.

Assigning the level of importance for the different questions was done considering comparable weighting factors as given by FEMA 154 and by expert judgement. The values for the different weighting factors were chosen such that:

1. The maximum vulnerability of a reinforced-concrete building is ~20 % lower than for a comparable masonry building.
2. The maximum vulnerability of a multi-story building is ~10 % lower than for a single-story building.

Primarily, structural damages and mode of failure in buildings due to earthquake depends mainly on factors like age of the building, structural type, number of storeys, configuration, design and maintenance. Hence in the present study, the statistics of these factors are evaluated from the collected data. These statistical data helps to visualize broadly the scenario of risk of the city for the earthquake.

For the non-structural seismic vulnerability assessment of hospital and health facilities 40 questions for hospital and 25 for school of 10 (8 in school case) different areas of expertise were collected (Table 8 and Table 12). Many of the questions of one area are interdependent so that some become not applicable (*NA*) if a certain question is answered in a certain way. Those questions are highlighted accordingly.

As it was done for the structural vulnerability part, a level of importance and an importance factor is assigned to each of the 40 questions. After having answered the 40 questions, the factors are summed up and the non-structural vulnerability factor *NVI* is derived by dividing the sum by the number of answered questions. Again, question which are not applicable (*NA*) are excluded and subtracted from the number of answered questions. The weighting factors were chosen such that the highest non-structural vulnerability factor *NVI* has a value of 10.

In case of hospitals, the safety of non-structural elements (equipment and services) is equally important as that of structural elements. Due to damage of non-structural elements like surgical operating light, high pressure steam sterilizer, oxygen cylinders or oxygen supply conduits, etc. in operations theatres, and crash cart mechanical ventilation systems, life supportive equipment, monitoring equipment, etc. in Intensive Care Unit, a hospital may become functionless. Also, the cost of non-structural elements may be even higher than structural cost. It was observed by the survey team, that in most of the surveyed hospitals, the seismic safety guidelines of non-structural elements are not followed especially in anchoring of oxygen cylinders and Monitors in ICU's, and provision of flexible joints to oxygen supply conduits.

Name (ID):		Occupancy:	☐ School ☐ Kindergarten ☐ University ☐ other: ______________
Address: Contact:		No. of:	☐ pupils/students: ________ among disabled: ________ ☐ teachers/employees: ________ ☐ classrooms: ________ ☐ total classroom area: ________ m^2
Coordinates:	Latitude ___________ Longitude ___________	Occupancy period:	☐ 24 h ☐ 12 h ☐ 8 h from: ______ to: _______
Structural characteristics:	Typology of the primary structure: ___________________________________ ___________________________________	Age:	☐ < 10 *years* ☐ 10 – 20 *years* ☐ 20 – 40 *years* ☐ > 40 *years* ☐ year of construction: _______
	no. of individual buildings: _____ no. of stories (basements): ____ (___) interstory height: _____ *m* no. of cores: _____	Actual state:	☐ good (new) ☐ recently renovated ☐ in need of renovation ☐ bad (decayed)
	plan shape: ☐ □ ☐ L ☐ U ☐ T max. length L: _____ *m* max. width W: _____ *m*	Maintenance program:	☐ exists ☐ does not exist if yes, in which period: _______________
Photo ID's:		Topography:	☐ plane (flat) ☐ sediment basin (valley) ☐ close to river ☐ foothill (base of slope) ☐ slope situation ☐ ridge (top of slope; hilltop)
Screener/date:			

Table 9. "General information" part of the school questionnaire

No.	FEATURES AFFECTING THE STRUCTURAL SEISMIC VULNERABILITY	RC YES	RC NO	RC NA	URM YES	URM NO	URM NA
01	Is the building irregular in plan?	8	0		10	0	
02	Are the columns regularly distributed?	0	4				
03	Are both building directions adequately braced (RC frames or shear walls, URM walls)?	0	16		0	20	
04	Does the ratio between the building's length and width is > 2.5 ?	4	0		10	0	
05	Does the building possess eccentric cores (staircases or elevators)?	8	0		10	0	
06	Does the building have a soft story?	16	0	0			
07	Is the building irregular in elevation caused by setbacks of upper stories?	8	0	0	20	0	0
08	Does the building have cantilevering upper stories?	8	0	0	10	0	0
09	Does the building possess a heavy mass at the top or at roof level?	4	0		5	0	
10	Are pounding effects possible?	4	0		5	0	
11	Does the building have short columns?	8	0				
12	Are strong beams–weak columns available?	16	0				
13	Does the building possess shear walls ?	0	4				
14	Did the building suffer any significant structural damage in the past?	4	0		5	0	
15	Does the building possess seismic retrofitting or strengthening measures?	0	8		0	5	
	SUM			max 120			max 100
	NO. OF ANSWERED QUESTIONS			12 or 15			8 or 10
	STRUCTURAL VULNERABILITY INDEX *SVI* (= Sum ÷ No. of questions)						

Table 10. "Structural seismic vulnerability" part of the school questionnaire

			Actual state			
			good (new)	***recently renovated***	***in need of renovation***	***bad (decayed)***
			ASF = 1.00	***ASF = 1.05***	***ASF = 1.10***	***ASF = 1.20***
Age	***< 10 years***	***AF = 1.00***	0 %	5 %	10 %	20 %
	10–20 years	***AF = 1.025***	2.5 %	7.6 %	12.8 %	23 %
	20–40 years	***AF = 1.05***	5 %	10.3 %	15.5 %	26 %
	> 40 years	***AF = 1.10***	10 %	15.5 %	21 %	32 %

Table 11. Suggested values for age factor AF and actual state factor ASF and the percentage increase of structural vulnerability index SVI

With regard to the development of the questionnaires and the decision on the weighting scheme for each individual question, the following aspects have considered:

1. Single-hazard approach: Solely seismic vulnerability is covered assessing the vulnerability of the building under earthquake loading. Other hazards such as volcanic activity, flooding, are not considered (no multi-hazard approach);
2. Only structural and non-structural vulnerability is addressed disregarding operational (functional) vulnerability. This because an objective assessment of functional features is very difficult and requires sophisticated interviews with different personnel groups of a hospital or school;
3. Separate SVI and NVI: Structural and non-structural vulnerability are treated separately since they are not automatically connected. This means, that separate vulnerability indexes are derived which should be considered in parallel but not be merged;
4. Geographic features (i.e. topographic situation of the building) are addressed in the questionnaires, however, these facts have no direct influence on the derived vulnerability indexes;
5. Simplicity: Each question has been formulated such that neither personal (subjective) opinions nor judgments of the screener can influence the answer. A question can only be answered by either stating 'YES' or 'NO'. Some questions, which may not be applicable in the specific case can be answered with "NA" (Not Applicable). These questions are explicitly marked

and will not excluded from the calculation of the vulnerability index. (The final decision on the questions has been done on a learning-by-doing basis and is considered to be an ongoing process);

6. Realizability: Especially with regard to non-structural features of the building the screener may not be able to reliably answer certain questions since these require detailed technical knowledge or since certain installations of the building are not easily accessible, e.g. elevator;

No.	FEATURES AFFECTING THE NON-STRUCTURAL SEISMIC VULNERABILITY	YES	NO	NA
I. Fire Fighting				
01	Are there **smoke detectors** and **alarms** available?	0	4	
02	Are there enough **fire extinguishers** and **hose-reel cabinets** available?	0	8	
03	Are they easily accessible? (if Q02 = NO → NA)	0	8	0
II. Elevators				
04	Are elevators available?	4	0	
05	Are elevators maintained and are they regularly (every 2 months) controlled? (if Q04 = NO → NA)	0	4	0
06	Are **motors** and **control cabinets** anchored to the floor? (if Q04 = NO → NA)	0	4	0
III. Non-structural Infill Walls and Partitions				
07	Are (infill) brick walls protected against out-of-plane failure by e.g. internal reinforcement or surface meshes?	0	8	
08	Do movement joints between brick infill walls and RC frames exist to allow damage-free movement? (for masonry buildings → NA)	0	8	0
IV. Ceilings				
09	Are suspended ceilings available?	8	0	
10	Are the suspended ceilings adequately secured against failure? (if Q09 = NO → NA)	0	8	0
V. Emergency Exits and Escape Routes				
11	If **exit fire doors** jam in an earthquake, is there a crowbar or sledge hammer readily available to facilitate emergency opening?	0	16	
12	Do all **exit doors** open outwards?	0	16	
13	Are all doors unlocked from the inside and also unblocked?	0	16	
14	Are the **windows of ground floor** barred/trellised?	8	0	
15	Are glazed windows available?	8	0	
16	Has the **glazing of windows** been designed to accommodate lateral movement? (if Q15 = NO → NA)	0	8	0
17	Do large windows, door transoms and skylights have safety glass? (if Q15 = NO → NA)	0	8	0
18	Are **emergency exits** and **escape routes** adequately designated, e.g. by fluorescent signs?	0	4	
19	Are emergency exits and escape routes adequately illuminated?	0	4	
VI. Appendages				
20	Can nonstructural elements (e.g. parapets, facade cladding, roof tiles, chimneys) fall from the building and harm children or teachers running outside?	8	0	
VII. Movable Equipment				
21	Are **wardrobes/lockers/bookshelves/blackboards** adequately anchored to the walls?	0	8	
22	Are **tables** stable enough to protect children from falling objects (e.g. suspended ceilings)?	0	8	
VIII. Appurtenant structures				
23	Are there enough **open spaces** around the building which can be used as escape routes and where people are safe from falling objects?	0	16	
24	Can **neighboring structures** (e.g. buildings, walls, electricity lines) block escape routes or harm people running/gathering outside?	8	0	
25	Can **road access** to and from the school be blocked due to collapse of buildings or geotechnical effects (slope failure, landslide etc.)?	8	0	
	SUM			max 208
	NO. OF ANSWERED QUESTIONS			max 25
	NON-STRUCTURAL VULNERABILITY INDEX ***NVI*** **(= Sum ÷ No. of questions)**			

Table 12. "Non-structural seismic vulnerability" part of the school questionnaire

7. Time limitation: The number of questions has been limited to a certain number so that interviews with hospital and school personnel lasting over several hours are avoided. Experience has proven that the thorough fill out of a questionnaire for a hospital including interviewing the maintenance personnel and a walk down through the facilities takes approximately 1 hour per individual building or block. Schools can be handled in 30 minutes as the number of questions and the building sizes generally are smaller;
8. Detailedness and comparability: The questionnaires try to avoid to be too detailed since this will lead to a multitude of building groups which can only be compared within one group. The weighting factors have been chosen such that structural vulnerability indexes SVI can be compared between different building typologies (RC, masonry) irrespective of the occupancy type (school, hospital). Non-structural vulnerability indexes NVI can only be compared for the respective occupancy type (schools or hospitals). [1]

Both, the questionnaire for schools and hospitals are given in Appendix A, with results by visual inspections done on a school and a hospital.

The goal of vulnerability questionnaires is a definition of a priority list of buildings to attend. This list is based on structural and non-structural vulnerabilities evaluated and it is designed for the authorities to solve the higher vulnerabilities in function of kind of vulnerability and the building's typology.

Governments have the ultimate responsibility for the safety of their citizens. At the national level and in cities, municipalities, and communities, governments have much at stake when it comes to ensuring their health services are available should disaster strike. Strong political commitment can make a tremendous difference to whether or not hospitals are safe.[25]

With respect to non-structural vulnerability, by first applications of questionnaires in Central America and India a direct comparison of single features is difficult so that this is limited to a comparison of the respective non-structural vulnerability indexes *NVI*. Similarities in the non-structural vulnerabilities between schools and hospitals in above mentioned countries could be observed. On average, higher NVI were derived for hospitals where even new or well-maintained buildings revealed severe deficits of non-structural issues. In terms of schools it needs to be stated that many of the observed buildings are of smaller size.

3.3. Analytical vulnerability assessment

Several methods have been proposed for rapid evaluation of building vulnerability at the territorial scale. Broadly speaking, there are two categories of assessment methods.

Observational-statistical "empirical" methods, adopted worldwide, rely on inspection and statistical treatment of a large number of observations from post-earthquake damage. However, unavailability of a homogeneous database of damage observations, especially for reinforced concrete (RC) buildings, and the use of macro-seismic scales to formulate damage probability, discourage using this method in the framework of quantitative methods for seismic risk assessment.

On the other hand, "mechanical" methods rely on structural modelling and analytical evaluation of the aptitude of buildings to be damaged by earthquakes of a given intensity; building classification is based on selected parameters that are assumed to have a clear influence on seismic behaviour and are properly accounted for in the building modelling process. The various analytical methods for deriving vulnerability relationships for RC buildings are generally dependent on model parameter availability (quality of the building inventory). A first approach adopts simple models, such as mechanism-based analyses, acknowledging the fact that the available data from building inventory are usually very poor. More refined methods (using pushover analyses or dynamic analyses) require a large number of input data for the generic building (geometric, structural and material properties). For this reason these methods are generally applied to single buildings considered as representative of an entire population.

The preparation of an inventory of the built environment is usually the most time-consuming and costly step in loss assessment. Therefore, although classical methods for compiling a building inventory could be supported by more innovative techniques, knowledge of the built environment at the regional scale is still generally limited to poor data, such as morphologic shape, base plant dimensions and building height.

The whole process is implemented with a specific procedure that allows: a) automatic generation of the correspondent lumped plasticity models for non-linear analyses; b) execution of pushover analyses to determine the global capacity parameters. In order to consider different performance levels, three limit states are introduced.[26]

3.3.1. Mechanical modelling and capacity analysis

The non-linear behaviour is investigated via pushover analysis; a lumped plasticity model is adopted. Capacity model is based on the computation of global capacity force-displacement curve through a non-linear lumped plasticity model.

Global seismic capacity parameters for the generic structure identified in the previous phase are determined via nonlinear static pushover analysis. Adopting the lumped plasticity model, it is first necessary to define the characteristic curve representing the nonlinear behaviour for each single element (beam and column); next, global building analyses for both directions X and Y is performed. The local element model allows to track it in the global building response.

The capacity curve, in terms of lateral strength V_b and displacement at the roof level Δ, is determined up to maximum lateral strength (near collapse), consistently with the adopted mechanical models.

Three limit states are evidenced along the pushover curve: damage limitation, significant damage and ultimate state. Damage limitation corresponds to the first attainment of $\theta = \theta_y$ for an element; significant damage corresponds to the first attainment of $\theta = 0.75 \cdot {}^{1}\!/_{\gamma_{el}} \cdot \theta_u$ for an element. The ultimate state corresponds to the first attainment between element failure, ultimate chord rotation $\theta = {}^{1}\!/_{\gamma_{el}} \cdot \theta_u$ (flexural failure), and global failure intended as near collapse condition of the structure.[26]

Reinforcement design for elements are performed by considering real steel percentage in elements for known elements and with a simulated design according to [27] for the other ones.

For each considered limit state, the capacity parameters for the SDOF equivalent to the real MDOF system: the capacity strength C_s, the capacity displacement C_d and the effective period T are computed automatically with a procedure done ad hoc.

3.3.1.1. Materials

With regard to material properties, the concrete compressive strength and the yield strength are assumed in collusion with used code ([27], [28]) and constructive practice in Central American countries. In fact, all materials used in constructions, as in the Code Hospitales 2004 in El Salvador, have to be of a good quality. The concrete have to be consistent with ASTM C150, and the steel consistent with ASTM

C-31 and ASTM C-39, with a minimum strength of 420 MPa for diameter greater than ½” and of 280 MPa for diameter equal to 3/8”.

According to [29], the modulus of elasticity is equal to:

$$E_{cm} = 22000 \cdot \left(f_{cm} / 10 \right)^{0.3} \tag{4}$$

Where $f_{cm} = f_{ck} + 8\,MPa$ is the mean compressive strength.

3.3.1.2. Strength capacity

Element flexural behaviour is characterized by the definition of a moment-rotation constitutive law (plastic hinge); Figure 6 shows the monotonic curve of the flexural model, which is described by seven parameters (M_{cr}, M_y, $M_{\max}$, M_u, θ_y, $\theta_{\max}$, θ_u). By performing nonlinear analyses of the extreme sections of each element it can be determined: cracking moment M_{cr}, yielding moment M_y, maximum moment $M_{\max}$, while M_u is evaluated as a fraction of $M_{\max}$ ($M_u = 0.8 \cdot M_{\max}$). The corresponding chord rotations (θ_y, $\theta_{\max}$, θ_u) are determined as a function of a number of mechanical/geometric factors.

The flexural behaviour is modelled with a quadri-linear moment-rotation relationship that describes the cracking, yielding, maximum and ultimate state of the element (Figure 6). The mean value of yield rotation is computed according to Eurocode 8 [29]:

$$\theta_y = \phi_y \cdot \frac{L_V + a_V z}{3} + 0{,}0013 \cdot \left(1 + 1{,}5 \cdot \frac{h}{L_V} \right) + 0{,}13 \cdot \phi_y \cdot \frac{d_b \cdot f_y}{\sqrt{f_c}} \tag{5}$$

where ϕ_y is the yield curvature of the end section, $a_V \cdot z$ is the tension shift of the bending moment (if $M_y < L_V \cdot V_{R,c}$ then $a_V = 0$), h is the section height, d_b is the mean diameter of the tension reinforcement, L_V is the shear span at member end, f_c and f_y are concrete compression strength and longitudinal steel yield strength in [*MPa*], respectively.

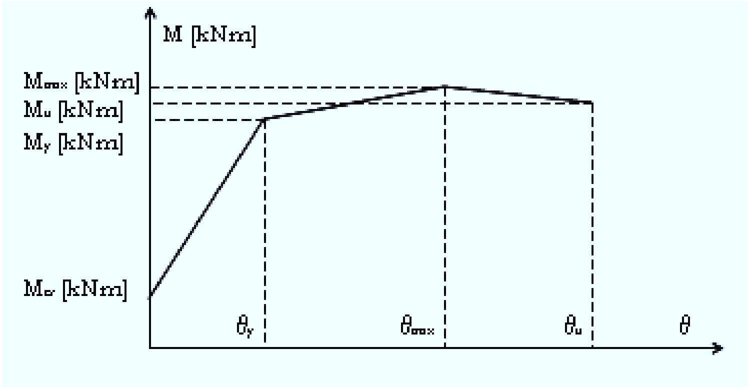


Figure 6. Moment-rotation relationship

It is assumed that the maximum moment is attained at a rotation equal to 1.5 times the yield rotation.

The ultimate rotation can be computed as the sum of the yield rotation and the plastic part of the chord rotation. Its mean value has the following expression[29]:

$$\theta_{um}^{pl} = \frac{1}{\gamma_{el}} \cdot 0.0145 \cdot \left(0.25^{\nu}\right) \cdot \left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)}\right]^{0.3} \cdot f_c^{0.2} \cdot \left(\frac{L_V}{h}\right)^{0.35} \cdot 25^{\left(\alpha \cdot \rho_{sx} \cdot \frac{f_{yw}}{f_c}\right)} \cdot \left(1.275^{100 \cdot \rho_d}\right) \tag{6}$$

where γ_{el} is an element factor (in this study it is assumed $\gamma_{el} = 1.0$), h is the depth of the cross-section, $\nu = N / (A_c \cdot f_c)$ is the normalized axial load, $\omega = A_s \cdot f_y / (b \cdot h \cdot f_c)$ and $\omega' = A_s' \cdot f_y / (b \cdot h \cdot f_c)$ are mechanic percentages of compression or tensile longitudinal steel respectively (b and h are the width and height of the cross-section, respectively), f_c, f_y and f_{yw} are concrete compression strength, longitudinal and transversal steel yield strength in [*MPa*], respectively, ρ_{sd} is transversal steel percentage, ρ_d is crosswise steel percentage, α is a confinement efficiency factor. In members without detailing for earthquake resistance, as in this case, the value given by Equation 6 is multiplied by a factor of 0.85.

The shear model is this one defined by Fardis in the [29]. The shear strength, as controlled by the stirrups and accounting for the reduction with the plastic part of the ductility demand, is computed by the following expression:

$$V_R = \frac{1}{\gamma_{el}} \left[\begin{array}{l} \frac{h-x}{2L_v} \cdot \min(N;0.55A_c \cdot f_c) + (1-0.05\min(5;\mu_\Delta^{pl}))\cdot \\ 0.16\max(0.5;100\rho_{tot})\cdot\left(1-0.16\min\left(5;\frac{L_V}{h}\right)\right)\cdot\sqrt{f_c}\cdot A_c + V_w \end{array} \right] \quad (7)$$

where: γ_{el} is assumed equal to 1,

h is the depth of cross section,

x is the compression zone depth,

N is the compressive axial force (positive, taken as being zero for tension),

$L_V=M/V$ is ratio moment/shear at the end section,

A_c is the cross-section area, taken as being equal to $b_w d$ for a cross-section with a rectangular web of width b_w and structural depth d,

f_c is the concrete compressive strength,

ρ_{tot} is the total longitudinal reinforcement ratio,

V_w is the contribution of transverse reinforcement to shear resistance, taken, for cross-sections with rectangular web of width b_w, as being equal to

$$V_w = \rho_w \cdot b_w \cdot z \cdot f_{yw} \quad (8)$$

where: ρ_w is the transverse reinforcement ratio,

z is the length of the internal lever arm, and

f_{yw} is the yield stress of the transverse reinforcement.

The cyclic shear resistance, V_R, decreases with the plastic part of the ductility demand, expressed in terms of ductility ratio of the transverse deflection of the shear span or of the chord rotation at the member end: $\mu_\Delta^{pl} = \mu_\Delta - 1$. In the generated model, μ_Δ^{pl} is calculated as the ratio of plastic part of the chord rotation, θ, normalized to the chord rotation at the state of yielding, θ_y, while V_R is considered to be constant.

The adopted modelling considers only the flexional contribution.

3.3.1.3. Variability in mechanical model

The definition of limit states at the structural level is a problematic issue, since they should roughly represent a certain damage level for the entire building and different approaches are proposed in the literature (SEAOC Vision 2000, 1995; Kircher et al., 1997; Eurocode 8, 2004, Dolsek and Fajfar, 2007).

Assuming that the most critical element controls the state of the structure (Eurocode 8, 2004; Fajfar, 1999) the global limit states can be detected as a function of limit states defined at the level of single elements. In particular, three different limit states are considered, i.e. limit states for Damage Limitation DL, corresponding to the first attainment of yield rotation for a column, Significant Damage (SD) and Near Collapse (NC) when the element rotation exceeds 75 or 100% of the ultimate rotation. It has to be considered that yielding and ultimate rotations are uncertain quantities, as evidenced in [30]. Equations (5) and (6) represent mean values of yielding and plastic rotations, respectively, and in order to account for their inherent uncertainty a suitable distribution should be assigned.

The variability of characteristic points of the moment-rotation relation at the element level has direct influence on the capacity parameters detected at the structural level. In order to assess such an influence, a variation of yielding and ultimate rotations in moment-rotation constitutive law is considered.

Adopting the relations as proposed in Panagiotakos and Fardis (2001), a lognormal variability with a coefficient of variation equal to 0.42 for yield and plastic rotations is adopted.

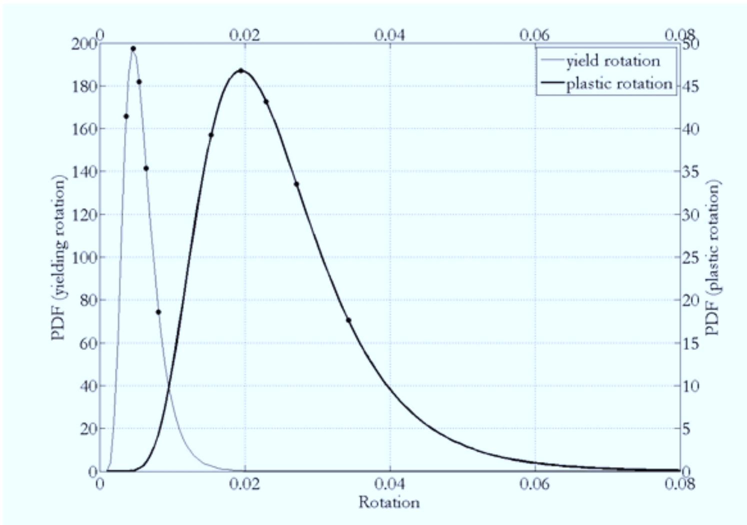


Figure 7. Rotation lognormal distribution

Figure 7 shows the lognormal distributions for both yield rotation and for the plastic part of the ultimate rotation of a concrete member; the five points plotted on each curve represent the 16, 34, 50, 66 and 84 percentiles.

In order to quantify the effect of variability of θ_y-θ_{um}^{pl} on the displacement capacity C_d, a series of analyses are performed for the representative building, considering rotation variability according to the distribution proposed by Panagiotakos and Fardis (2001). In the chapter 5, analysis results are presented.

Given that materials' strength values are investigated by non destructive tests and adopted in function of code hint and of used planning practice, a variability in material values is adopted. So, concrete and steel strengths are assumed to be normal distributed with a Coefficient of Variation equal to 25% and 8%, respectively ([31], [32]).

3.3.1.4. Variability in materials

While analysing existing buildings and computing their structural capacity, many uncertainties are involved in the model. In the conventional code provisions, in fact, the various knowledge degrees are considered by different Confidential Factors which are adopted for the materials and the model itself.

In the following, a parametric evaluation of materials is conducted. In order to consider the uncertainty of the material strength, a combination of five concrete types with five different types of steel is investigated; the considered points on normal distribution are representative of mean value, mean value plus and minus standard deviation and mean value plus and minus double standard deviation ($\mu - 2\cdot\sigma$, $\mu - 1\cdot\sigma$, μ, $\mu + 1\cdot\sigma$, $\mu + 2\cdot\sigma$).

Both, concrete and steel strengths f_c and f_y are considered through a normal distribution with a mean value and CoV 25% and CoV 8%, respectively; the Coefficients of Variation adopted are taken from [31] and [32].

Figure 8 shows the normal distributions of materials with the ten points (five for each material) taken into account in the study.

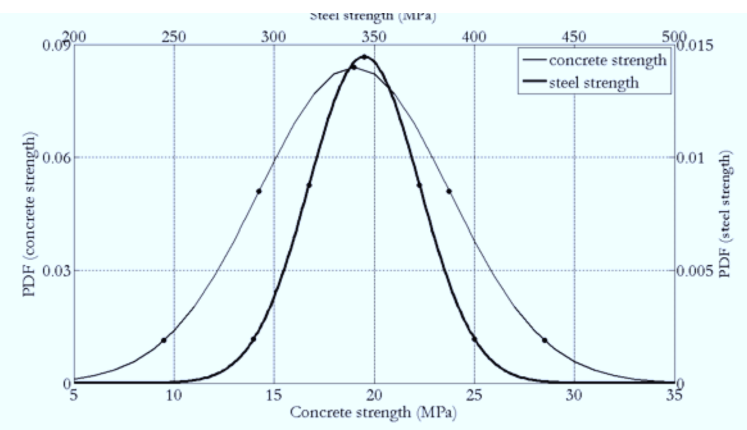


Figure 8. Normal distribution adopted for materials

3.3.1.5. Displacement rotation capacity

Global seismic capacity parameters, for the single limit state, are determined with reference to an equivalent Single Degree Of Freedom system, defined starting from the capacity curve of the Multiple Degree Of Freedom model (real structure). Transformation of the capacity curve into bilinear form allows us to estimate: nonlinear strength C_s, capacity displacement C_d and the period T of the SDOF structure.

From each Capacity Curve for MDOF, the Bilinear Curve is extracted by geometric equation.

The area under Capacity Curve is:

$$A_{CC} = \frac{(V_{b+1} + V_{bi}) \cdot (\Delta_{i+1} - \Delta_i)}{2} \tag{9}$$

the ultimate displacement admitted for structure is the maximum displacement of the Pushover Curve

$$\delta_u = \max(\delta) \tag{10}$$

the resistance strength for building is taken equal to the maximum value of strength

$$V_y^* = \max(V_b) \qquad (11).$$

For Bilinear system, the area under curve is

$$A_{BC} = \frac{V_y^* \cdot \delta_y}{2} + V_y^* \cdot (\delta_m - \delta_y) = V_y^* \cdot \delta_m - \frac{V_y^* \cdot \delta_y}{2} \qquad (12),$$

so, for equivalence of the two system, it is

$$A_{CC} = A_{BC} \qquad (13).$$

Therefore, for eq. (13), yield displacement for the Bilinear System is:

$$\delta_y = 2 \cdot \delta_m - \frac{2 \cdot A_{CC}}{V_y^*} \qquad (14).$$

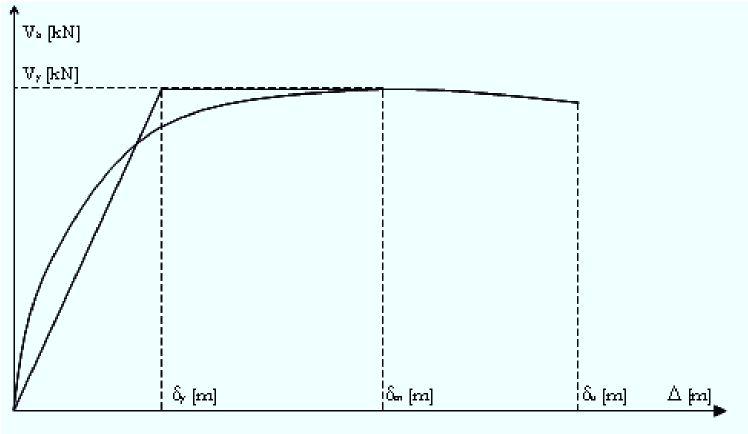


Figure 10: Pushover Curve and Bilinear System

From Bilinear Curve, it is possible to evaluate Nonlinear parameters for a Single Degree Of Freedom system (SDOF system).

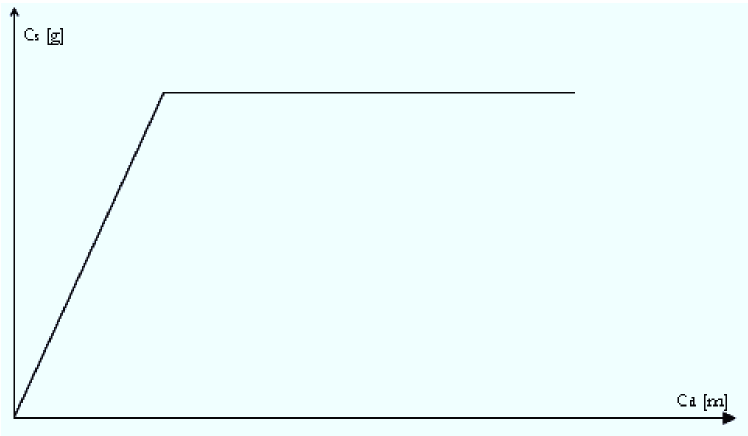


Figure 11: Single Degree Of Freedom system

Nonlinear stiffness is the ratio between Bilinear System strength and its yield displacement:

$$k^* = \frac{V_y^*}{\delta_y} \quad (15);$$

The participation factor is equal to:

$$\Gamma = \frac{\sum_i m_i \cdot \phi_i}{\sum_i m_i \cdot \phi_i^2} \quad (16)$$

where m_i is the mass at the level i, and ϕ_i is the ratio of displacement at the level i over the roof displacement.

The effective period T^* of the idealized equivalent SDOF system is determined by:

$$T^* = 2 \cdot \pi \cdot \sqrt{\frac{\sum_i m_i \cdot \phi_i}{k^*}} \quad (17),$$

the nonlinear strength is

$$C_s = \frac{V_y^{\ *}}{\left(\sum_i m_i \cdot \phi_i\right)^2} \cdot \sum_i m_i \cdot \phi_i^2 \quad (18)$$

and the nonlinear displacement of the SDOF system is

$$C_d = \frac{\delta_u}{\Gamma} \quad (19).$$

3.3.2. RC infilled frames influence

A good study on behaviour and analysis of masonry-infilled frames was done by P. Benson Shing and Armin B. Mehrabi [33].

In their study it is shown the frequently use of masonry infills as interior partitions and exterior walls in buildings. Their strengths are not negligible, and they will interact with the bounding frame when the structure is subjected to strong lateral loads induced by earthquakes. This interaction may or may not be beneficial to the performance of the structure, and it has been a topic of much debate in the last few decades.

Frame–infill interaction can induce brittle shear failures of reinforced concrete columns and short-column phenomena. Furthermore, infills can over-strengthen the upper stories of a structure and induce a soft first storey, which is highly undesirable from the earthquake resistance standpoint.

In spite of the aforementioned shortcomings that have sometimes been observed, masonry infills have been used to strengthen existing structures, in fact they can improve the earthquake resistance of a frame structure if they are properly designed. The main difficulty in evaluating the performance of an infilled structure is to determine the type of interaction between the infill and the frame.

All studies have shown that the behaviour of an infilled frame is heavily influenced by the interaction of the infill with its bounding frame. In most instances, the lateral resistance of an infilled frame is not equal to a simple sum of those of the infill and the bounding frame because frame–infill interaction can alter the load-resisting mechanisms of the individual components.

Studies have shown that infill panels can significantly enhance the performance of a bare frame under earthquake loads, provided the short-column phenomenon and the brittle shear behaviour of the columns can be prohibited.

The behaviour of masonry-infilled frames is complicated, and this type of structure can exhibit a number of possible failure mechanisms. The load resistance of an infilled frame depends to a large extent on the frame–infill interaction, and cannot be considered as a simple sum of the resistances of a bare frame and a stand-lone wall panel. Frame–infill interaction can change the resistance mechanism of a reinforced concrete frame from ductile flexure to brittle shear.

Most recent studies have indicated that infills can enhance the earthquake resistance of frame structures, provided they are properly designed. [33]

3.3.3. Spectral analysis and CSM

Seismic inelastic demand is evaluated according to capacity spectrum method (CSM) by Fajfar 1999[34]. In collusion with this approach, elastic demand is evaluated in function of effective period of structure on ADRS spectrum (Acceleration Demand Response Spectrum) where displacement is on x-axis and acceleration is on y-axis. Y-axis values of structural capacity curve (SDOF) are divided by generalized mass to obtain SDOF capacity acceleration (Figure 9). To evaluate inelastic displacement demand, it need to modify elastic spectral demand $S_{d,e}(T)$ through a factor C_R by Miranda et al. 2003 [35], [36] which represents the ratio between inelastic displacement and elastic one of SDOF.

$$S_{d,i}(T) = S_{d,e}(T) \cdot C_R(R,T) \tag{20}$$

In the equation 20 T is the effective period and R is the reduction factor defined as:

$$R = \frac{m \cdot S_{a,e}(T)}{V_y} = \frac{S_{a,e}(T)}{S_{ay}} \tag{21}$$

where m is the generalized mass, $S_{ae}(T)$ is the elastic acceleration, V_y is the inelastic strength and S_{ay} is the structural inelastic acceleration.

The factor C_R is computed according to following equation:

$$C_R = 1 + \left[\frac{1}{a \cdot \left(T \middle/ T_s \right)^b} - \frac{1}{c} \right] \cdot (R - 1) \tag{22}$$

where the mathematical constant a, b, c and the period T_s assume the values reported in Table 13 depending on site class B, C, D by NEHRP classification:

Site Class	***a***	***B***	***c***	***Ts [sec]***
B	42	1.60	45	0.75
C	48	1.80	50	0.85
D	57	1.85	60	1.05

Table 13. Mathematical values to determine coefficient CR

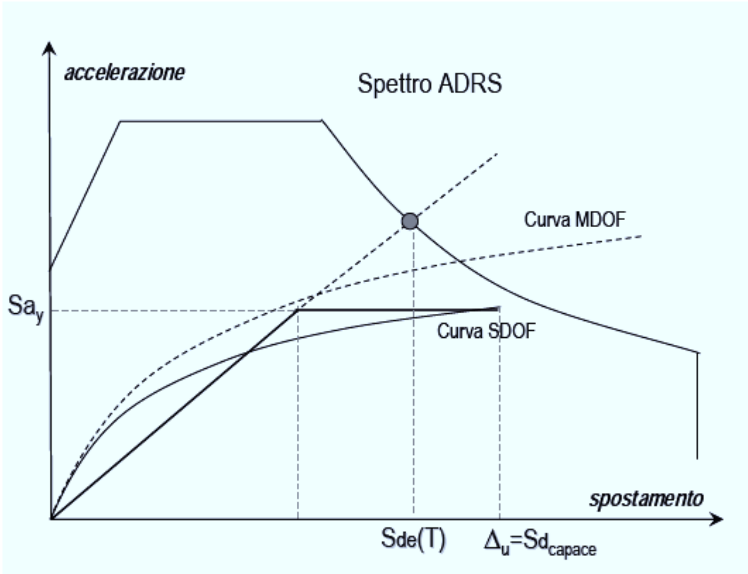


Figure 9. ADRS spectrum [37]

Actually CR represent a mean value linked with a variation index shown in Figure 10. Depending on period and reduction coefficient for each soil class, it is possible to express variability in seismic input which on average assumes the value of 40% in case of period higher than one second while it can reach the 100 % per period lower than 0.5 seconds.

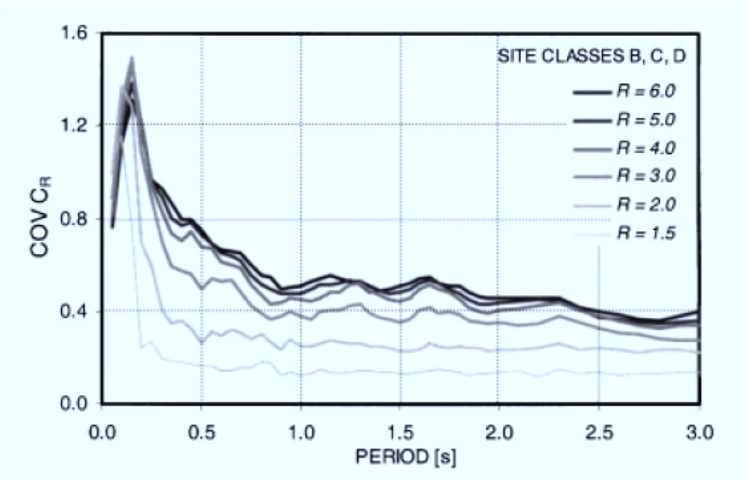


Figure 10. Coefficient of variation of inelastic displacement ratios for all 216 ground motions recorded in NEHRP site classes B, C and D [35]

To consider this variability, for each pseudo random shot of probabilistic simulation the coefficient $C_R(R,T)$ is evaluated, drawing from a lognormal distribution characterized by mean value according to equation 22 and standard deviation equal to CoV of C_R.

3.3.4. Ground condition and seismic action

To define seismic action, the IBC-2006 is used.

The standardized response spectrum shape, as given in IBC 2006, consists in four parts: a region from peak ground acceleration (PGA) to T_A, a region of constant spectral acceleration at periods from zero seconds to T_{AV}, a region of constant spectral acceleration between periods from T_{AV} to T_{VD}, and a region of constant spectral displacement for periods of T_{VD} and beyond.

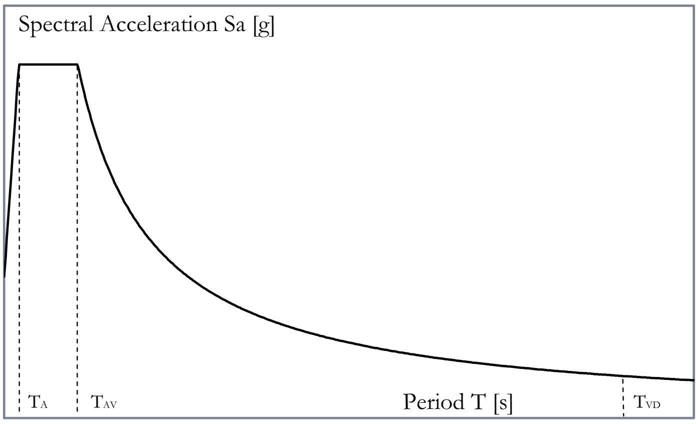


Figure 11. IBC-2006 Spectra (T - Sa)

The elastic design spectrum $S_a(T)$ is defined by the following equations:

$$S_a(T) = S_a@0.3 \cdot \left(0.4 + 0.6 \cdot T/T_A\right) \qquad \text{if } 0 < T < T_A \qquad (23)$$

$$S_a(T) = S_a@0.3 \qquad \text{if } T_A < T < T_{AV} \qquad (24)$$

$$S_a(T) = S_a@1.0/T \qquad \text{if } T_{AV} < T < T_{Vd} \qquad (25)$$

$$S_a(T) = S_a@1.0 \cdot T_{VD}/T^2 \qquad \text{if } T_{VD} < T < 10\,s \qquad (26)$$

where: $S_a@0.3$ is Sa at 0.3 s;

$S_a@1.0$ is Sa at 1.0 s;

T_{AV} is the period based on the intersection of the region of constant spectral acceleration and constant spectral velocity, its value is $T_{AV} = S_a@1.0/S_a@0.3$;

T_A is the left corner period of the spectral plateau, its value is

$$T_A = 0.2 \cdot T_{AV} = 0.2 \cdot \left(S_a@1.0/S_a@0.3\right) \qquad (27)$$

T_{VD} is the reciprocal of the corn frequency f_c, which is proportional to dress drop and seismic moment, its value is a function of moment magnitude M $T_{VD} = \dfrac{1}{f_c} = 10^{\left[(M-5)/2\right]}$; when the moment magnitude is not known, the period TVD is assumed to be 10 seconds (M 7.0).

In order to be able to describe the elastic design spectra (for rock: site class B) in case that only Peak Ground Acceleration (PGA) is given, the following expressions have to be regarded:

$$S_a @ 0.3 = S_{AS} = 2.5 \cdot PGA \tag{28}$$

$$S_a @ 1.0 = S_{Al} = PGA \tag{29}$$

Amplification of ground shaking to account for local site conditions is based on the site classes and soil amplification factors as given by the IBC-2006 provisions. The methodology amplifies rock (B) PGA by the same factor as that specified in Table 15 for short period spectral acceleration, as

$$PGA_i = PGA \cdot F_{Ai} \tag{30}$$

in which PGA_i is the peak ground acceleration for site class i (expressed in [*g*]); *PGA* is that for site class B (expressed in [*g*]) and F_{Ai} is the short period amplification factor for site class i, for spectral acceleration S_{AS}.

Site class	***Site class description***	***Shear-wave velocity vs,30 [m/s]***
A	Hard rock, eastern U.S. sites only	> 1500
B	Rock	760 – 1500
C	Very dense soil and soft rock	360 – 760
D	Stiff soil	180 – 360
E	Soft soil, profile with >3m of soft clay defined as soil with plasticity index PI>20, moisture content w>40%	< 180
F	Soils requiring site-specific evaluations	-

Table 14. "NEHRP" site classification (FEMA, 1997a) as applied by IBC-2006 (ICC, 2006)

The construction of demand spectra including soil effects is done using the following equation for short periods:

$$S_{ASi} = S_{AS} \cdot F_{Ai} \tag{31}$$

and for long periods:

$$S_{Ali} = S_{Al} \cdot F_{Vi} \tag{32}$$

while the period T_{AV}, which defines the transition period from constant spectral acceleration to constant spectral velocity is a function of the site class. It can be determined by the following equation:

$$T_{AVi} = \left(\frac{S_{Al}}{S_{AS}}\right) \cdot \left(\frac{F_{Vi}}{F_{Ai}}\right) \tag{33}$$

where:

S_{ASi} – short-period spectral acceleration for Site Class i (in [g]);

S_{AS} – short-period spectral acceleration for Site Class B (in [g]);

F_{Ai} – short-period amplification factor for site class i and for spectral acceleration SAS;

S_{Ali} – 1-second period spectral acceleration for Site Class i (in [g]);

S_{Al} – 1-second period spectral acceleration for Site Class B (in [g]);

F_{Vi} – short-period amplification factor for site class *i* and for spectral acceleration S_{Al};

T_{AVi} – transition period between constant spectral acceleration and constant spectral velocity for Site Class *i* (in [*sec*]).

Site Class B	**Site Class**				
Spectral Acceleration	**A**	**B**	**C**	**D**	**E**
Short-Period, S_{AS} [g]	***Short-Period Amplification Factor, F_A***				
0.25	0.8	1.0	1.2	1.6	2.5
0.50	0.8	1.0	1.2	1.4	1.7
0.75	0.8	1.0	1.1	1.2	1.2
1.00	0.8	1.0	1.0	1.1	0.9
> 1.00	0.8	1.0	1.0	1.0	0.9
1-Second Period, S_{A1} [g]	***1-Second Period Amplification Factor, F_v***				
0.1	0.8	1.0	1.7	2.4	3.5
0.2	0.8	1.0	1.6	2.0	3.2
0.3	0.8	1.0	1.5	1.8	2.8
0.4	0.8	1.0	1.4	1.6	2.4
> 0.4	0.8	1.0	1.3	1.5	2.4

Table 15. Site amplification factors as given in IBC-2006

For the evaluation of structural damage it is more convenient to plot the acceleration response spectrum as a function of spectral displacement. This could be due to the relation between the different spectral parameters:

$$\frac{S_a}{\omega} = S_v = S_d \cdot \omega \tag{34}$$

in which ω is the circular natural frequency of the oscillator

$$\left(\omega = 2 \cdot \pi \cdot f = 2 \cdot \pi \cdot \frac{1}{T}\right) \tag{35}$$

The final result of this process is the computation of a 5% damped response spectrum.

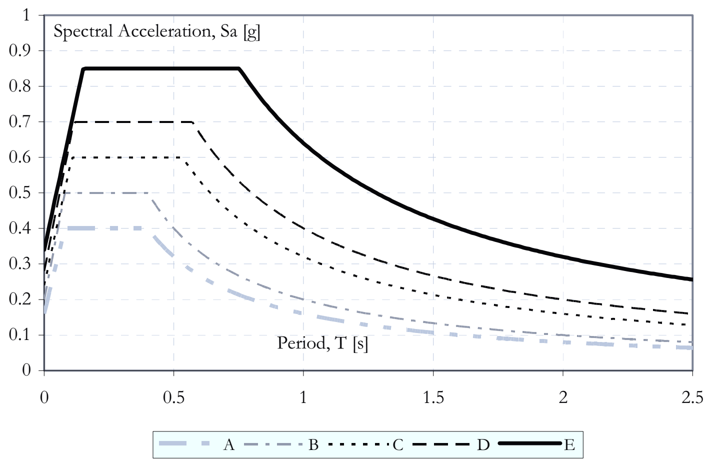


Figure 12. IBC-2006 Spectrum (T – Sa)

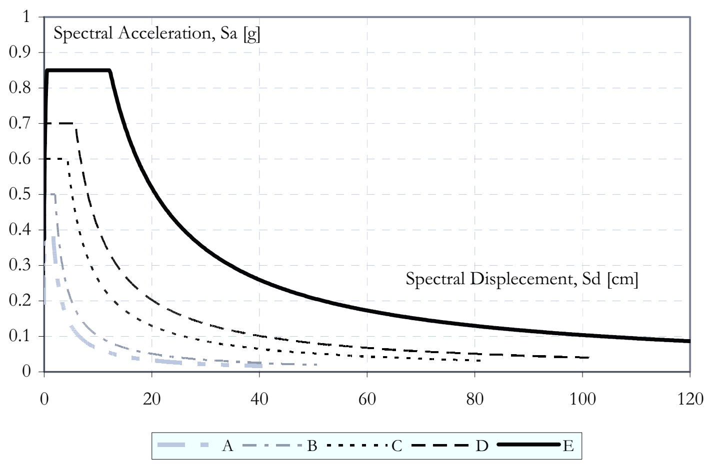


Figure 13. IBC-2006 Spectrum (Sd – Sa)

3.3.5. Fragility functions

Evaluated seismic capacity, it is possible to compute fragility curve considering variation in seismic intensity. Fragility curve represents the probability for considered structures to reach a fixed limit state, for a given spectral intensity measure parameter. Variability in capacity and demand is generated by both structural system variability and spectral demand. While variability in capacity given by variability in structural system follows an assigned probability low, all considered spectrums have the same probability to occur. The variability in input is given by uncertainty in the evaluation of expected inelastic displacement which is function of CoV C_R [4].

When investigating on the seismic vulnerability of a building class there are a number of factors that determine variability of the fragility curves, i.e. affecting the slope of the curve representing the probability of attaining a certain damage level for a building class. According to [38] those factors can be related to the variability of the building class capacity β_c within the class, the variability of the seismic demand β_d and the variability of the limit state thresholds β_{tds} that are adopted to represent relevant damage in the analysis process. Besides, if the calculations of damage and loss are based on the integration of the fragility with hazard functions that have already incorporated ground shaking variability in the hazard calculations or when a response spectrum is reasonably well known the explicit accounting of β_d is not requested and the variability of the fragility curves depends only on β_c, β_{tds}. β_{tds} is related to the reliability of the mechanical models adopted in the analysis and is independent from β_c, β_d.

The force-displacement curve for SDOF, suitably idealized, allows to identify the capacity parameters, C_d, C_s and T, that are necessary to evaluate the seismic demand with a spectral approach. In fact, from the displacement spectrum, the elastic displacement demand for the equivalent SDOF is straightforwardly determined in correspondence of the period T.

The inelastic demand, that is evaluated multiplying the elastic displacement demand by a modification factor (C_R) depending on effective period T and on the spectral reduction factor R [36], is confronted to the displacement capacity C_d of the SDOF (for the corresponding limit state) in order to check for failure at the given limit state. The reduction factor R is defined as the ratio of the elastic acceleration demand versus C_s, the base shear coefficient.

Fragility curves are obtained using the mechanical based procedure as described in [4] and further implemented in [8] that allows the construction of the fragility curves for varying damage levels. In such a method, the capacity of a reinforced concrete building class is analysed starting from push-over analyses performed for virtually all the buildings.

The method consists of a series of subsequent steps: (a) perform building inventory and determine statistics of the building model input parameters; (b) generate a sample of building models, perform push-over analyses and determine global capacity parameters, such as non-linear strength C_s, displacement capacity C_d and the effective period T of the equivalent SDOF, for each one of them (c) run Montecarlo analysis extracting random model input parameters, corresponding to generic building, from the relative statistics and determine, by the confrontation with demand compute as above, the fragility for different damage levels.

The process to derive fragility function can be synthesized with the following equation:

$$fragility = \sum_{a} P[C \leq D | IM = a] \tag{36}$$

Fragility is the sum of probability that capacity is lower that demand for a fixed seismic intensity measure, considering all seismic intensity, as shown by equation 36. Fragility curve is a cumulative density function of probability reported as a function of a spectral parameter, generally the peak ground acceleration (from now on PGA).

Fixed PGA, the point on fragility curve represent the percentage of buildings belonging to analysed class which overtake the considered limit state (capacity lower than demand).

Because it need to compute exceeding probability for various PGA, when more spectrums are considered, it is necessary to make change in PGA to obtain overtaking probability.

When geo-seismic information are not available, the only way to determine demand is to use a code spectrum, zone consistent. In this way the determination of fragility function is obtained changing PGA in spectrum constant shaped.

Instead, when zone compatible spectrums are available, the computation is more refined because various spectral shape are considered. The best solution, but not always possible, is to have the hazard curves. In this way all spectrum corresponding to PGA may be considered, with their probability to happen and their various spectral shape. This eventuality allows to determine risk. For each pseudo random shot, the capacity and the demand are evaluated for a fixed PGA, obtaining in case of failure ($C \leq D$) the increase of control variable k. At the end of all pseudo random shot, probability of failure is given by ratio between k and all shots, obtaining for a PGA a point on the fragility function [37].

For example, the fragility curve shown in Figure 14, expresses that the probability of failure to reach near collapse limit state for a PGA equal to $0.4\,g$ is about 60 %.

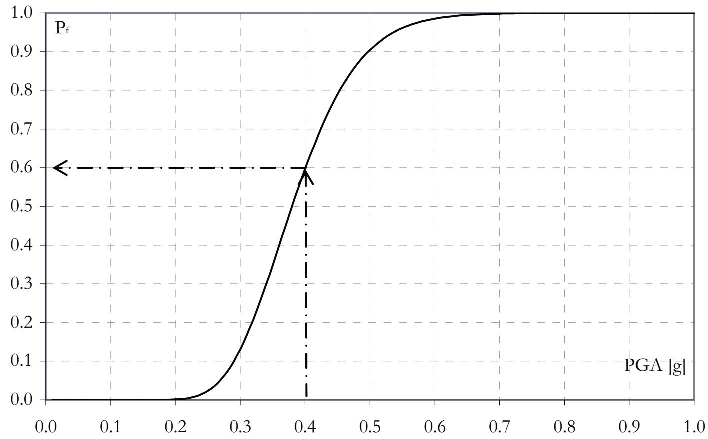


Figure 14. Example of fragility curve

Chapter 4. High-Priority Buildings in Central America Selection

Since slight modifications in the construction techniques and building code provisions exist between the different Central American countries, a number of representative school and hospital buildings were identified in each of the three countries. A visual inspection in Guatemala, El Salvador and Nicaragua was done. In addition to visual inspections and questionnaire surveys, chosen reinforced concrete buildings, geometric and structural surveys were conducted on schools and hospitals and non-destructive material testings were done in order to get an idea about the general concrete quality and reinforcement detailing.

No destructive test was done for the difficulty in make screen test; only one piece of bar used in a new construction in El Salvador was taken and tested in official laboratory of University of Naples. In Figure 15, σ-ε relationship for steel is reported, showing typical American characters.

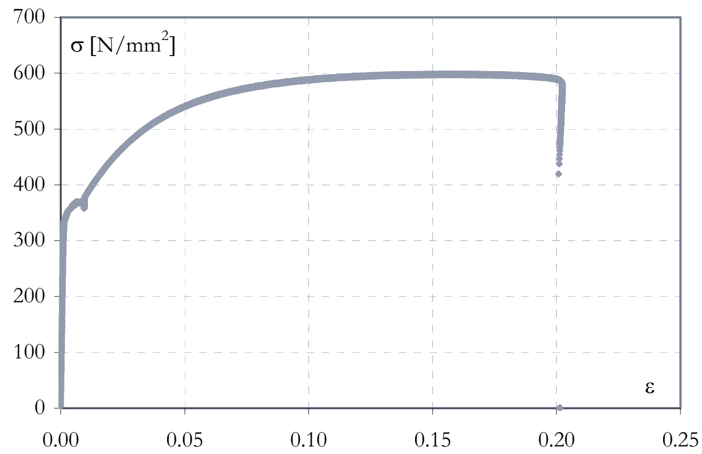


Figure 15. Experimental test on new C.A. steel bar in University of Naples' Official Laboratory

4.1. Selected study areas

As part of a regional cooperation project on the reduction of earthquake risk in the Central American countries Guatemala, El Salvador, and Nicaragua (RESIS-II), one of the major work tasks consists in the identification of the structural and non-structural seismic vulnerability of schools, hospitals and health centers. The project proposal is:

- Establish building categories for each city and preferably with regional applicability. Some selection criteria will be height, building material, load frame type, bracing type, degree of reinforcement etc. Additionally the function (residential, commercial etc.) will be used for the classification.
- Building categories as function of materials, age and maintenance. (structural system and number of stories are critical parameters, exposure to previous earthquakes may also be very important).
- Establish pushover parameters for each building type and associated (physical) vulnerability curves.
- Cost effective strengthening recommendations for the building categories.
- Typical number of occupancies for each building category.
- Special buildings of particular importance.

In total building stock inventories by walk-down surveys were conducted in four different study areas (Table 16). The location of the study areas as well as their subdivision into geographical units are depicted in Figure 16 - Figure 19.

No.	***Study area***	***Country***	***No. of geounits***	***No. of buildings***	***Total population***
1	San Salvador – Distrito 2	El Salvador	16	3,377	16,870
2	Guatemala City – Zona 11	Guatemala	3	2,499	22,047
3	Managua – Distrito 4 (Racachaca)	Nicaragua	1	385	3,412
4	Masaya – inner-city area	Nicaragua	1	834	6,474

Table 16. characterization and state of elaboration of the different study areas

The study area in Masaya, a small city located southeast of Managua (Nicaragua), will be at the current state excluded from the risk and loss computations. However, the building stock survey in this more rural settlement provided useful information on the available Nicaraguan building typologies which goes into the defined classification scheme.

Figure 16 - Figure 19 illustrate satellite images of the entire metropolitan areas of the four target cities as well as zoomed cutouts of the study areas with their single geographical units (census tracts):

- Figure 16: San Salvador (El Salvador) – Part of Distrito 2;
- Figure 17: Guatemala City (Guatemala) – Part of Zona 11;
- Figure 18: Managua (Nicaragua) – Part of Distrito 4 (Racachaca);
- Figure 19: Masaya (Nicaragua) – inner-city area.

Figure 16. Location and clustering of the study area in San Salvador – Part of Distrito 2 [1]

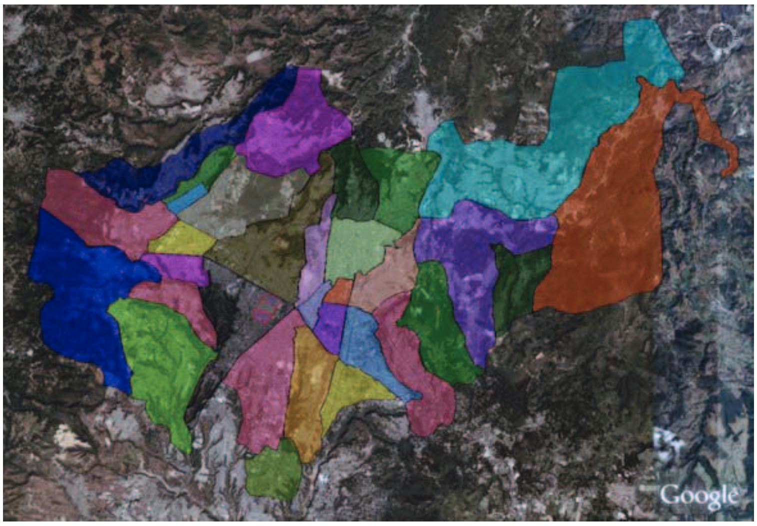


Figure 17. Location and clustering of the study area in Guatemala City – Part of Zona 11 [1]

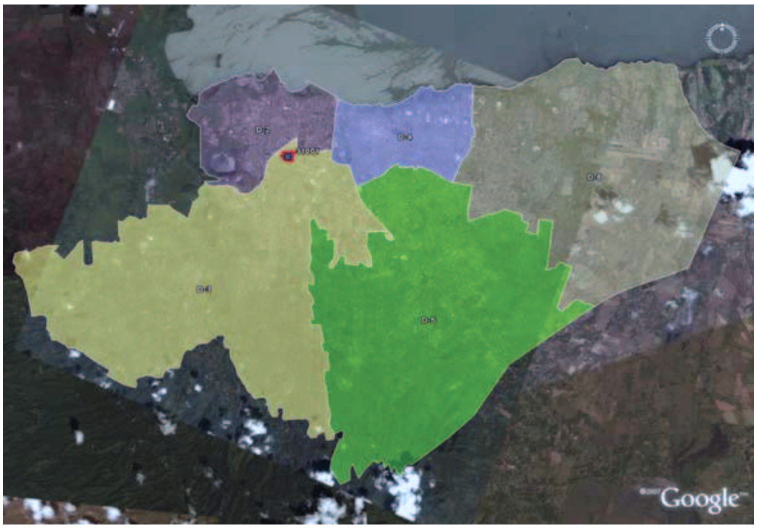


Figure 18. Location and clustering of the study area in Managua – Part of Distrito 4 (Racachaca) [1]

Figure 19. Location and clustering of the study area in Masaya – inner-city area [1]

In order to get a first impression on the type of study areas Figure 20 illustrates the distribution of the general housing types and occupancy classes. Before a detailed classification of the different building types is given, a coarse classification of the building's wall material is used.

As it can be taken from Figure 20, the different study areas are quite comparable in terms of main occupancy (residential and commercial use) as well as prevalent building wall materials.

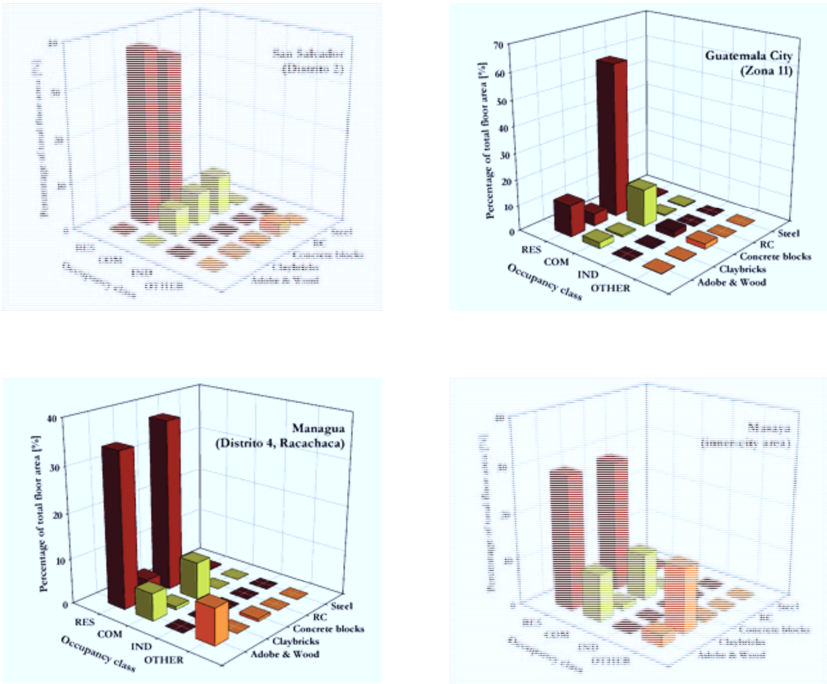
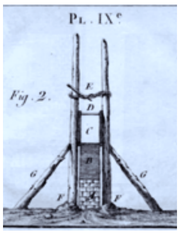


Figure 20. Composition of the building inventory in the different study areas illustrating the distribution of occupancy class and building types [1]

4.2. Building typologies

In Central American Countries, structural typologies are various: apart from reinforced concrete and masonry buildings, characteristic systems it can be found. The most common systems are Vivienda de Adobe and Vivienda de Bahareque (Timber Building) in El Salvador; Adobe with sawn timber roof framing and corrugated iron sheeting and Vivienda de Adobe (adobe brick houses) in Guatemala; and Adobe and Vivienda de Minifalda (wooden houses with heavy bases) in Nicaragua.[39]

Following different typologies of Central American buildings are reported.

Index	***Name, illustration***	***Reference, short description***
MF	Minifalda	→ EERI WHE: Lang *et al.* (2008)
		The term 'minifalda', translated 'miniskirt' refers to the building's walls which consist of masonry or concrete in the lower part, while the upper part is made of a light wood construction (also 'madera y concreto'). The combination of a more stable and consolidated base made of concrete or masonry and a light and flexible upper part of the walls made of wood frame construction, provides these houses with some advantages. However, the heavy roofs, which consist mostly of tiles, increase the vulnerability of the buildings especially during earthquake action.
AD	Adobe brick masonry	→ EERI WHE: Lang *et al.* (2007)
		Buildings made of adobe brick masonry can still be found in all parts of Central and Latin America. Generally adobe houses are characterized by only one story, no basement, and sometimes an irregular plan shape. The main use is residential or small commercial (retail trade) purposes.
TP	Tapial (rammed earth)	Wikipedia
	 	Comparable adobe brick masonry except for the fact that the construction process for the walls is different. Using it involves a process of compressing a damp mixture of earth into an externally supported frame that molds the shape of a wall section creating a solid wall of earth.
BH	Bahareque	→ EERI WHE: Lang *et al.* (2007)

<table>
<tr><th>Index</th><th>Name, illustration</th><th>Reference, short description</th></tr>
<tr><td rowspan="2"></td><td></td><td>The bahareque construction type refers to a mixed timber, bamboo and mud wall construction technique. The term ‘bahareque’ (also ‘bajareque’) has no precise equivalent in English, however in some Latin American countries this construction type is known as ‘quincha’ (engl.: wattle and daub).</td></tr>
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<tr><td rowspan="2">TZ</td><td>Taquezal</td><td>→ EERI WHE: Lang et al. (2007)</td></tr>
<tr><td></td><td>Predominantly can be found in Nicaragua. This building type is comparable with bahareque (see BH) only that the structural walls are composed of vertical timber elements and horizontal struts which are always made of timber slats. One- and two-story high structures can be found.</td></tr>
<tr><td>CLu</td><td>Unreinforced claybrick masonry</td><td>→ HAZUS Technical Manual:</td></tr>
</table>

Index	***Name, illustration***	***Reference, short description***
		chapter 5.2.1
		This building type is comparable with HAZUS model building type URML (predominantly low-rise). All different kinds of slab types can be found ranging from wood to RC.
	Acc. to HAZUS Technical Manual, chapter 5.2.1 (Unreinforced Masonry Bearing Walls - URM): These buildings include structural elements that vary depending on the building's age and, to a lesser extent, its geographic location. In buildings built before 1900, the majority of floor and roof construction consists of wood sheathing supported by wood framing. In large multi-story buildings, the floors are cast-in-place concrete supported by the unreinforced masonry walls and/or steel or concrete interior framing. In unreinforced masonry constructed after 1950 (outside California) wood floors usually have plywood rather than board sheathing. In regions of lower seismicity, buildings of this type constructed more recently can include floor and roof framing that consists of metal deck and concrete fill supported by steel framing elements. The perimeter walls, and possibly some interior walls, are unreinforced masonry. The walls may or may not be anchored to the diaphragms. Ties between the walls and diaphragms are more common for the bearing walls than for walls that are parallel to the floor framing. Roof ties usually are less common and more erratically spaced than those at the floor levels. Interior partitions that interconnect the floors and roof can reduce diaphragm displacements.	
CLri	Internal reinforced claybrick masonry	
	– no illustration available –	Compare with CLu except that the masonry walls are reinforced by mostly vertical steel bars which are internally arranged.
CLrc	Confined claybrick masonry	
		The masonry walls are additionally strengthened by horizontal and vertical reinforced-concrete refinements. These are arranged in regular distances so that a kind of column-beam impression is created. However, the bearing capacity of these confined walls is totally different from that of a frame system.

Index	***Name, illustration***	***Reference, short description***
	Acc. to HAZUS Technical Manual, chapter 5.2.1 (Reinforced Masonry Bearing Walls with Precast Concrete Diaphragms - RM2): These buildings have bearing walls similar to those of reinforced masonry bearing wall structures with wood or metal deck diaphragms, but the roof and floors are composed of precast concrete elements such as planks or tee-beams and the precast roof and floor elements are supported on interior beams and columns of steel or concrete (cast-in-place or precast). The precast horizontal elements often have a cast-in-place topping.	
CBu	Unreinforced concrete block masonry 	Comparable with CLu but using larger precast concrete blocks instead of claybricks.
CBri	Internal reinforced concrete block masonry 	Comparable with CLri but using larger precast concrete blocks instead of claybricks.
CBrc	Confined concrete block masonry 	Comparable with CLrc but using larger precast concrete blocks instead of claybricks.

Index	*Name, illustration*	*Reference, short description*
PdC	Piedra de cantera	
		Traditional construction technique which can be solely found in rural areas. "Confinement" of quarry stones by vertically arranged wooden trusses.
BP	Bloque panel	→ HAZUS Technical Manual: chapter 5.2.1
	 	Confined (precast) concrete panels which can be either arranged vertically (welded steel connections) and horizontally (wood connection to roofing). Residential, commercial, office, light roofing, span width ~ 3 m, pre-stressed concrete elements; Bloque Panel is a building system which is not very spread out in the country. Some ONG's are pushing this system into the construction practice but it is relegated to low income housing, yet. It consists of confined, pre-cast concrete panels, with pre-cast columns erected on a foundation beam and a concrete collar beam at the top connected to the roof system (Juayua, Sonsonate, El Salvador). Comparable with HAZUS model building type PC1 ().
LT	Laminada troquelada	→ HAZUS Technical Manual: chapter 5.2.1

Index	***Name, illustration***	***Reference, short description***
		Comparable with HAZUS model building type S3 (steel light frame). The steel frames are covered by corrugated light-weight decorated steel plates. Often used for educational buildings, hospitals or storage halls.
	Acc. to HAZUS Technical Manual, chapter 5.2.1 (Steel light frame – S3): These buildings are pre-engineered and prefabricated with transverse rigid frames. The roof and walls consist of lightweight panels, usually corrugated metal. The frames are designed for maximum efficiency, often with tapered beam and column sections built up of light steel plates. The frames are built in segments and assembled in the field with bolted joints. Lateral loads in the transverse direction are resisted by the rigid frames with loads distributed to them by diaphragm elements, typically rod-braced steel roof framing bays. Tension rod bracing typically resists loads in the longitudinal direction.	

Table 17. Description of local building types [40]

4.3. Schools, hospitals and health centres

Schools and hospitals in Central American countries are structural typology very different comparing with each other.

Height schools and ten hospitals located in Guatemala, El Salvador and Nicaragua were visually inspected and non-invasive material tests were conducted at the primary structural concrete elements. Thereby, the geometrical percentage of reinforcing steel and the rebound number for concrete were identified. Following, some of hospitals (H) and schools (S) inspected are reported, divided for country. On inspected buildings, also the questionnaires were applied.

Observed school buildings in Central America are generally low-rise structures with one or two stories; the structural system is usually very simple, preferably rectangular base plan shapes and repetitive span lengths; reinforced concrete elements are comparably slender and roofs are often of corrugated metal sheets resting on wooden beams (rafters). In contrast, hospital buildings are generally more engineered structures with higher story numbers, larger concrete cross-sections and longer beam spans.

In fact, considering only reinforced concrete structures, schools are generally two level building with a length/width ratio greater than 2.5; structural elements are slender and often under dimensioned; the maintenance state is often inadequate.

Instead, public hospitals, where only poor people go, are higher then 3 level, apart for Nicaragua in which the 1976 earthquake destroyed the entire city of Managua, so during the reconstruction only low buildings were realised. Considering the structural peculiarities, hospitals are well thought up, in fact engineers in the design of elements follow a sort of code provisions or simple the constructive experience (there is no code, apart for El Salvador), reached also after big earthquakes which often occur in these places.

An overview of hospitals and schools in Central America is presented in the following, with analogies and differences between various Countries and distinct use purposes.

Table 18 and Table 19 show inspected schools and hospitals with the indication of number of levels, the ratio between length and width, the plan shape and structural typology.

Index	***Name***	***Country***	***N***	***L/W***	***Plan Shape***	***Type***
S-01	Centro Educativo Republica de Ecuador, San Salvador	ELS	1	n.a.	□	masonry
S-02	Centro Educativo Republica de Guatemala, San Salvador	ELS	1	n.a.	E	adobe
S-03	Complejo Educativo Catholico Santo Domingo, Chiltiupan	ELS	2	n.a.	□	RC frames
S-04	Escuela Centro Escolar Catolico A. R. M. Mazzini, San Salvador	ELS	2	n.a.	□	mixed
S-05	C. E. Republica de Colombia, Guatemala City	GUA	2	2.8	□	RC frames
S-06	C. E. Republica de Austria, Guatemala City	GUA	2	3.4	□	RC frames
S-07	Escuela Instituto Nacional Azarias H. Pallais, Managua	NIC	2	5.5	□	RC frames
S-08	Escuela Nacional Rigoberto Lopez Perez, Managua	NIC	2	3.9	□	RC frames

Table 18. Inspected schools with the indication of levels, L/W ratio, plan shape and structural typology

Index	Name	Country	N	L/W	Plan Shape	Type
H-01	Dr. Luis Edmundo Vasquez, Chalatenango	ELS	6	3.2.	□	RC frames
H-02	Laboratorio Max Bloch, San Salvador	ELS	3	2.5	□	RC frames
H-03	Hospital National San Juan de Dios, Santa Ana	ELS	2	n.a.	□	RC frames
H-04	Hospital National de Chalchuapa, Chalchuapa	ELS	1	3.1	□	confined masonry
H-05	Hospital National de Ninos Benjamin Bloom, San Salvador	ELS	11	2.2	□	RC frames
H-06a	Hospital National San Rafael, San Rafael	ELS	5	n.a.	Y	RC frames
H-06b	Hospital National San Rafael, San Rafael	ELS	3	n.a.	□	RC frames
H-07	Roosevelt Hospital - Maternidad, Guatemala	GUA	6	4.9	□	RC frames
H-08	IGSS Hospital Pediatria, Guatemala	GUA	5	6.5	T	RC frames
H-09	Hospital Robero Calderon Gutierrez, Managua	NIC	1	n.a.	□	mixed
H-10	Hospital Velez Pais, Managua	NIC	2	n.a.	V	RC frames

Table 19. Inspected hospitals with the indication of levels, L/W ratio, plan shape and structural typology

In the Chapter 5, a school and an hospital will be studied; the school is representative of Central American school typology: reinforced concrete, two levels, with a L/W greater than 2.5, with a rectangular plan shape. For hospitals, a six levels height RC structure will be studied, as a mean of structural typologies in case of hospitals, found in Central America.

4.3.1. El Salvador

After the 2001 earthquakes (January 13 and February 13), which characterized the collapse of a lot high-priority buildings, a code for hospitals was written. As above mentioned (2.3.1), it was the first time that a dedicated code was introduced in a

Central American country. Following the Department of Health began a project of retrofit for hospitals according to the code.

Before, a seismic code was introduced in 1989, inspired to American Codes.

All structures previous to 1989 were designed according to American Codes or, generally for small buildings, to common practice.

Highest buildings in the Central America are in El Salvador; in fact, inspected hospitals are between three and eleven levels high, as shown in Table 19. The plan-shape is often rectangular. Instead, schools' structures are always two level high.

Index	***Name***	***Illustration***
H-01	Dr. Luis Edmundo Vasquez Chalatenango	
H-02	Laboratorio Max Bloch, San Salvador	
H-03	Nacional San Juan de Dios Santa Ana, Santa Ana	

Index	Name	Illustration
H-04	Nacional de Chalchuapa, Chalchuapa	
H-05	National de Niños Benjamin Bloom, San Salvador	
H-06	Nacional San Rafael, San Rafael	

Table 20. Hospital inspected in El Salvador

Index	Name	Illustration
S-01	C.E. Republica de Ecuador, San Salvador	

Index	***Name***	***Illustration***
S-02	C.E. Republica de Guatemala, San Salvador	
S-03	Complejo E. Catholico Santo Domingo, Chiltiupan	
S-04	Centro Escolar Catolico Alberto Ricardo M. Mazzini, San Salvador	

Table 21. Schools inspected in El Salvador

4.3.2. Guatemala

Guatemala has no code, they are inspired by American code, without follow all design indications; in fact, they have standard cross-section to use in design: columns have always cross-section *20 cm* × *20 cm* with four longitudinal steel bars (steel percentage about 2.50 %), beams are not greater than *20 cm* × *35 cm* with a steel geometric percentage between 1.5 % and 2.0 %.

Foundations are superficial and isolated; often they are placed on the ground without any enticement surface.

Fortunately, in design of hospitals, more attention is posed, so common cross-sections are amplified.

In Guatemala, schools are two levels high, with rectangular plan-shape and a length/width ratio greater than 2.5; instead hospitals are higher than three level, often five with an inter-storey of about *3.8 m*.

Index	***Name***	***Illustration***
H-07	Roosevelt Hospital – Maternidad, Guatemala City	
H-08	IGSS Hospital Pediatria, Guatemala City	

Table 22. Hospitals inspected in Guatemala

Index	***Name***	***Illustration***
S-05	C.E. Republica de Colombia, Guatemala City	

Index	Name	Illustration
S-06	C.E. Republica de Austria, Guatemala City	

Table 23. Schools inspected in Guatemala

4.3.3. Nicaragua

Nicaragua is characterized by low size buildings build after the Managua earthquake in 1976 which destroyed the all city. Also here, schools are two levels high, with a inter-story of about *3 m*. Instead, hospitals are often one level high or almost two levels. Structural elements are dimensioned according to design practice because there is no code.

Index	Name	Illustration
H-09	Roberto Calderon Gutierrez, Managua	
H-10	Velez Pais, Managua	

Table 24. Hospitals inspected in Nicaragua

Index	Name	Illustration
S-07	Instituto Nacional Azarias H. Pallais, Managua	
S-08	Nacional Rigoberto Lopez Perez, Managua	

Table 25. Schools inspected in Nicaragua

4.4. Priority list

Table 26 represents the statistical analysis of the structural vulnerability part of the questionnaires. These results clearly show that the structural vulnerability of both hospitals and schools in Central America.

No.	Factor affecting structural vulnerability	Hospitals	Schools
1	irregularity in plan	37 %	13 %
2	irregularly distributed columns	5 %	0 %
3	inadequately braced building directions	11 %	13 %

No.	Factor affecting structural vulnerability	Hospitals	Schools
4	L/W ratio > 2.5	37 %	75 %
5	eccentric cores	26 %	38 %
6	soft storey	5 %	0 %
7	irregularity in elevation caused by setbacks	22 %	0 %
8	cantilevering upper stories	7 %	50 %
9	heavy mass at the top or at roof level	4 %	0 %
10	pounding effects possible	26 %	25 %
11	short columns	76 %	100 %
12	strong beams–weak columns	19 %	17 %
13	no shear walls	81 %	83 %
14	structural damage in the past	26 %	38 %
15	no retrofitting/strengthening	93 %	100 %

Table 26. Statistical analysis of the questionnaire parts addressing structural vulnerability. Given numbers represent percentages of total investigated buildings that confirm the respective vulnerability-affecting feature

Chapter 5. Analytical Mechanical Vulnerability Assessment

The regional cooperation project RESIS II (*Reduccion de Riesgo Sismico*) is focused on earthquake risk reduction for the Central American countries Guatemala, El Salvador and Nicaragua. Beside a number of project tasks dealing with seismic hazard and risk assessment, a main part of the project is allocated to earthquake vulnerability studies of those buildings that are of major importance to the society.

Among high-priority buildings, great importance should be attached to hospitals and schools in case of a natural disaster such as an earthquake. This because both building types are preferably used as shelter, meeting point, or organizational hub during the aftermath of a disaster. In addition, both hospitals and schools are characterized by extremely high occupancy rates (i.e. people/m^2) with a high number of very low-resilient occupants such as patients and children. According to FEMA 174 (1989), daytime occupancy rates for hospitals and schools in the United States are estimated to 5.0 people/100 m^2 (with a 24/7 occupancy) and 20.0 people/100 m^2, respectively. From experience these occupancy rates are much higher for developing countries. Even though the direct economic losses caused by earthquake shaking to school buildings are comparably low, there are good reasons to draw attention to these buildings. [41]

Observed school buildings in Central America are generally low-rise structures with one or two stories; the structural system is usually very simple, preferably rectangular base plan shapes and repetitive span lengths; reinforced concrete elements are comparably slender and roofs are often of corrugated metal sheets resting on wooden beams (rafters). In contrast, hospital buildings are generally more engineered structures with higher story numbers, larger concrete cross-sections and longer beam spans.

Eight schools and ten hospitals located in Guatemala, El Salvador and Nicaragua were visually inspected and non-invasive material tests were conducted at the primary structural concrete elements. Thereby, the geometrical percentage of reinforcing steel and the rebound number for concrete were identified. Had knowledge of reinforcing steel percentage of *k* elements of total *n* structural elements of each building, the remaining (*n-k*) elements are evaluated with a simulated design according to Eurocode 8.

Structural seismic capacity curves and vulnerability functions are derived for some existing buildings which are representative for a larger number of schools and hospitals in most Central American countries.

The geometric and mechanic characteristics of the structure establish the basis to model the real behaviour of the structure, and to perform structural analyses combining gravity and seismic loads.[42]

5.1. "Republica de Colombia" elementary school – Guatemala City

5.1.1. Site-dependent seismic demand

According to available geological information, local soil conditions at the building site consist of very dense soils and soft rocks, with shear-wave velocity $v_{s,30}$ between 360 m/s and 760 m/s (i.e. NEHRP soil class C).

5.1.2. Reference code

The building's performance is evaluated according to:

- Eurocode 2 "Design of concrete structures – Part 1-1: General rules and rules for buildings", EN 1992-1-1, December 2004;
- Eurocode 8 "Design of Structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings", EN 1998-1, December 2004;
- Eurocode 8 "Design of Structures for earthquake resistance – Part 3: Strengthening and repair of buildings", EN 1998-3, June 2005;
- Eurocode 8 "Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects", EN 1998-5, November 2004.

5.1.3. Safety evaluation

As above mentioned, safety evaluation of existing buildings have to consider a limit state more than in new design because they don't satisfy both resistance hierarchy and elements' ductility. Security requires refer to structural damage state defined by:

- Limit State of Damage Limitation (DL);
- Limit State of Significant Damage (SD);
- Limit State of Near Collapse (NC).

The procedure adopted to evaluate the building follows the next steps:

- Data analysis;
- Definition of knowledge level;
- Definition of seismic action based on various limit states;
- Modelling and analysis;
- Evaluation of results.

5.1.4. Building description

The main building of "Republica de Colombia" elementary school is located in Guatemala City, Zone 11. The structure was constructed in 1965 and survived the *M* 7.5 Guatemala earthquake on February 4, 1976 which caused 23,000 deaths and considerable structural damage in Guatemala City (Marroquin and Gándora, 1976; USGS, 1976).

Figure 21. View from outside - School Republica de Colombia

The primary load-bearing structure consists of a reinforced-concrete frame with beams in both directions. The two-story building has a rectangular plan with a length-to-width ratio of 2.75 (length 56.9 *m*, width 20.6 *m*; Figure 28). The upper story is accessible by two external stair cases which are symmetrically arranged at the middle of the buildings longer sides. The roof of stair cases has a ceiling lower than this one at the upper story. On both levels the classrooms are located at the central parts of the plan and accessible through a porch.

Figure 22. Porch at ground floor - School Republica de Colombia

Figure 23. Porch at first level- School Republica de Colombia

The inter-story height is equal to 3.0 m; the second floor has a double-pitched roof with a gable height of 0.80 m (Figure 28).

The main load-resisting structure consists of reinforced-concrete frames; in both lateral directions, the beams cross-sections are 0.20 m × 0.20 m (geometric steel percentage 2.45 %) or 0.20 m × 0.35 m (steel percentage 1.45 %). All columns have cross-sections of 0.20 m × 0.20 m and a steel percentage of steel equal to 2.54% (see Structural Peculiarities in 5.1.5). The structural system neither includes cores nor shear walls.

Figure 24. Roof of corrugated metal sheets supported by wooden trusses - School Republica de Colombia

The slabs of 0.10 m thickness are constituted by joist every 1.2 *m* (Figure 25). The slabs' suspension directions are indicated in Figure 28.

Figure 25. Slab - School Republica de Colombia

The roof (Figure 24) consists of corrugated metal sheets supported by wooden trusses (0.075 *m* × 0.20 *m*, every 1.25 *m*) and longitudinal planking (0.04 *m* × 0.04 *m*, every 1.0 *m*). The entire roof bears on five reinforced concrete longitudinal trusses and on a transversal riddle of 0.20 *m* × 0.20 *m*.

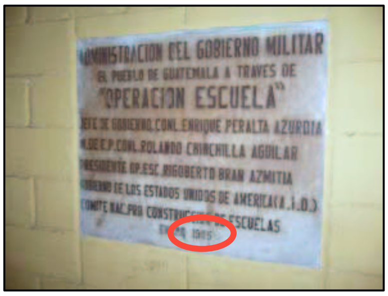


Figure 26. Year of construction - School Republica de Colombia

Figure 27. View from inside - School Republica de Colombia

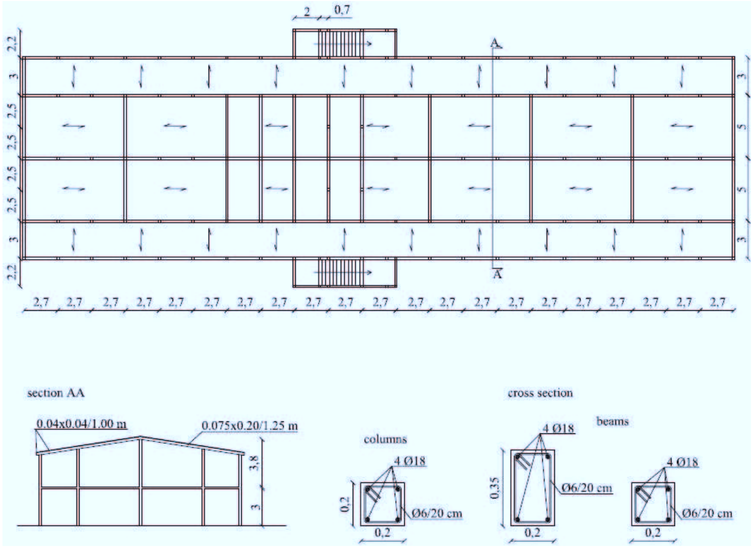


Figure 28. Plan and cross-section of the building as well as sections of the main concrete elements - School Republica de Colombia [41]

The actual state of the building is characterized by evident spalling of concrete coverage leading to corrosion of bars. In addition, in-situ testing shows high carbonation results of concrete.

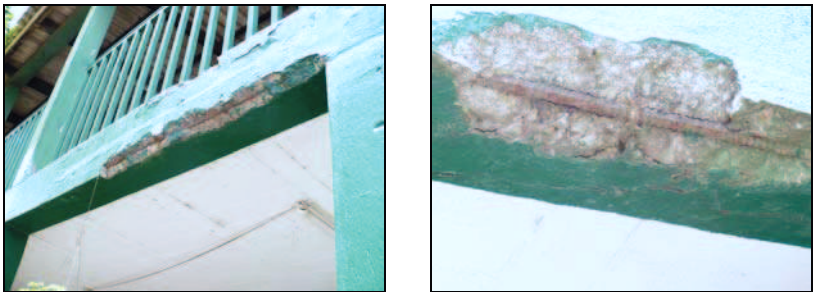


Figure 29. Spalling of concrete coverage - School Republica de Colombia

5.1.5. Evaluation data

As above mentioned, necessary fonts to evaluate data are:

- Design tables;
- Geometrical and structural relief;
- In-situ testing;
- Practice technical and code prescriptions.

Quantity and quality of obtained data define levels of knowledge; this is characterized by following aspects:

1. geometry of structural elements;
2. structural peculiarities and bars' placement and mechanical steel percentage;
3. mechanical property of materials.

Geometry. For 'Republica de Colombia' school there aren't any available designs, so a geometric relief is done; it allows to know all resisting frames to gravity and seismic loads, structural elements and their dimensions.

Figure 30. Short column on the stair case School Republica de Colombia

Figure 31. Beam – column joint - School Republica de Colombia

Structural peculiarities. Mechanical steel percentage and bars' placement are located by pacometric tests and caliber measures, allowed only in some structural elements. In fact, only *k* elements of total *n* are investigated; for the other (*n-k*) elements, a simulated design is done according to [27]. For beams, the strains is computed from linear combination considering G+Q and assumed steel percentage in function of the ratio between bending moment and the product between the base and the square of the useful height, and of the ratio between cover and useful height (ACI 63). For columns it is adopted the recommendation from ACI 63 (geometric percentage = 0.01 – 0.08 gross section).

Figure 32. Caliper of longitudinal bars' diameter - School Republica de Colombia

Figure 33. Caliper of stirrup diameter - School Republica de Colombia

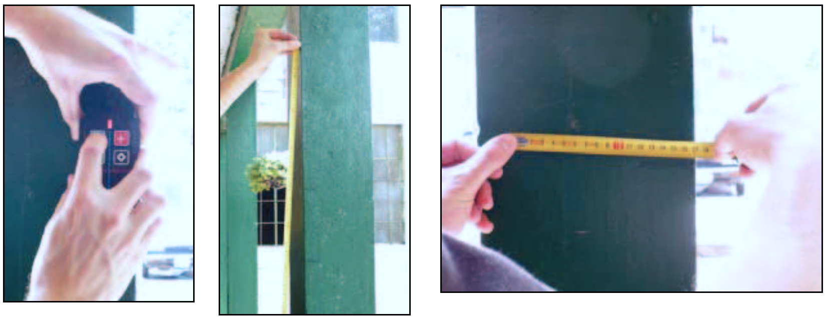


Figure 34. Pacometric test on columns - School Republica de Colombia

Mechanical property of materials. Materials used to model the structural behaviour are concrete with a mean value of cylindrical compression strength f_c equal to *21 MPa* and a steel characterized by mean value of yield strength f_y equal to *310 MPa*. These adopted values derive from FEMA [28]: in table *"Default Lower-Bound Tensile and Yield Properties of Reinforcing Bars for Various Periods"* steel in years 1959-1966, for an intermediate – hard grade has a value of minimum yield equal to 40,000-50,000 psi (about 310 MPa); for concrete the table "*Default Lower-Bound Compressive Strength of Structural Concrete"* by [28] was used, the lower value of compressive strength suggested by FEMA for frame build in 1950-1969, is 3,000 psi (about 21 MPa).

In situ tests with rebound hammer (Figure 35) were done, but results were not used because by visual inspection a high level of concrete carbonation results, so there is a 50% strength increase.

Column C1 (ground floor outer axis)

43	45	43	43	42
45	44	42	47	42
42	43	44	40	42
42	42	45	43	47
46	43	43	36	49

Rcub = 46.06 N/mm2

Column C2 (ground floor outer axis)

43	39	37	42	39
44	41	42	40	36
37	43	47	41	43
43	44	44	44	47
46	42	41	47	39

Rcub = 42.7 N/mm2

Column C3 (ground floor inner axis)

46	38	42	40	43
44	40	40	42	43
41	36	37	42	44
41	37	36	44	45
47	42	40	47	41

Rcub = 41.38 N/mm2

Column C4 (ground floor)

38	42	44	42	45
40	45	42	45	42
40	44	44	45	44
38	37	43	45	40
42	45	44	41	44

Rcub = 43.73 N/mm2

Column C5 (ground floor inner axis)

42	40	44	38	45
43	43	44	45	44
40	44	48	49	41
47	46	44	48	45
36	49	46	48	40

Rcub = 47.79 N/mm2

Column C6 (upper floor)

40	45	38	40	43
42	38	43	38	40
42	40	42	48	48
45	44	42	41	45
45	40	45	43	43

Rcub = 43.63 N/mm2

Column C7 (upper floor west wing)

32	34	38	32	32
38	40	37	40	40
38	37	40	38	38
40	42	40	38	32
35	32	40	36	39

Rcub = 31.19 N/mm2

Column C8 (upper floor west wing)

40	43	36	40	38
42	40	42	40	40
34	36	42	38	38
38	37	40	38	40
39	36	33	35	38

Rcub = 34.24 N/mm2

Column C9 (upper floor east wing)

43	45	45	42	45
35	35	39	42	40
33	29	42	40	42
36	40	44	44	38
33	40	39	43	44

Rcub = 37.47 N/mm2

Column C10 (upper floor east wing)

36	47	33	48	42
40	38	46	43	40
36	39	36	36	39
43	38	36	38	40
42	38	42	38	40

Rcub = 37.09 N/mm2

Figure 35. Rebound hammer test results - School Republica de Colombia

Information's availability based on geometrical study, structural peculiarities and mechanical property of materials, allows to reach knowledge level enough to study the structural behaviour.

5.1.6. Seismic action

To define seismic action, the IBC-2006 is used. The soil is a very dense soil and soft rock, with shear-wave velocity $v_{s,30}$ between 360 m/s and 760 m/s, site class C in the IBC.

Known the soil characters, elastic spectra in terms of spectral acceleration in function of period T and of spectral displacement is obtained.

5.1.7. Nonlinear static analysis

Nonlinear static (pushover) analysis is a non-linear static analysis under constant gravity loads and monotonically increasing horizontal loads. It is based on the assumption that the response of the structure can be related to the response of an equivalent Single Degree Of Freedom system (SDOF), that is used to determine seismic demand.

The analysis continues until a predefined limit state is reached or until structural collapse is detected.

Seismic loads generally act in combination with the (static) gravity loads. According to the applied code provision [43] the different load cases have to be defined. After the Eurocode EN 1998, earthquake loads E are multiplied by an importance factor γ_I .

Following, it is reported the equation used to combine seismic and static loads:

$$\gamma_I \cdot E + G + P + \sum \Psi_2 \cdot Q. \tag{37}$$

For Pushover analysis, seismic loads are applied after a deformed configuration due to the static load case ($G + \Psi_2 \cdot Q$).

Occupancy type	Ψ_2
Residential, office	0.30
Public, commercial, schools, hospitals	0.60
Rood, no trod	0.50
Actives, libraries, stair cases	0.80
Wind, thermal variation	0.00

Table 27. Combination coefficient for variable actions

5.1.7.1. Calculation of dead loads

Dead loads (G) affecting the beams (in [kN/m]) are computed as the product of the specific load of the slab (in [kN/m^2]) and its impact depth (in [m]) on the beam. The self weight of the elements is also considered.

First story:

Slab self weight:

$$g_1 = \gamma_{slab} \times thickness = 18 \times 0.1 = 1.8\, {kN}/{m^2}$$

Screed:

$$g_2 = g_{screed} = 0.6\, {kN}/{m^2}$$

Flooring:

$$g_3 = g_{floor} = 0.4\, {kN}/{m^2}$$

Masonry partitions (considering partitions equally distributed on the area):

$$g_4 = g_{partitions} = 0.4\, {kN}/{m^2}$$

$$G_I = g_1 + g_2 + g_3 + g_4 = 3.2\, {kN}/{m^2}$$

Roof:

Specific load at the roof is computed as the ratio between the sum of loads at the roof and the area.

Wooden beams $0.075 \times 0.04\, m^2$

$$g_1 = \gamma_w \times V = 4 \times (45 \times 2 \times 8.00 \times 0.075 \times 0.20) = 43.2\, kN$$

Wooden beams $0.04 \times 0.04\, m^2$

$$g_2 = \gamma_w \times V = 4 \times (18 \times 55.00 \times 0.04 \times 0.04) = 6.3\, kN$$

R.C. beams $0.20 \times 0.35\, m^2$

$$g_3 = \gamma_{cls} \times V = 25 \times (5 \times 56.7 \times 0.20 \times 0.35) = 496.13\, kN$$

R.C. beams $0.20 \times 0.20\, m^2$

$$g_4 = \gamma_{cls} \times V = 25 \times (4 \times 3.03 \times 0.20 \times 0.20) = 12.12\, kN$$

$$g_5 = \gamma_{cls} \times V = 25 \times (4 \times 5.06 \times 0.20 \times 0.20) = 20.24\, kN$$

Columns in R.C. $0.20 \times 0.20\, m^2$

$$g_6 = \gamma_{cls} \times \frac{V}{2} = 25 \times \frac{(48 \times 2.8 \times 0.20 \times 0.20)}{2} = 67.2\, kN$$

$$g_7 = \gamma_{cls} \times \frac{V}{2} = 25 \times \frac{(44 \times 3.03 \times 0.20 \times 0.20)}{2} = 66.7\, kN$$

$$g_8 = \gamma_{cls} \times \frac{V}{2} = 25 \times \frac{(24 \times 3.8 \times 0.20 \times 0.20)}{2} = 45.6\, kN$$

$$G_{roof} = g_1 + g_2 + g_3 + g_4 + g_5 + g_6 + g_7 + g_8 = 757.5\, kN$$

$$G_r = \frac{G_{roof}}{A_{roof}} = \frac{757.5}{967.2} = 0.78\, {kN}/{m^2}$$

Stair cases:

Slab self weight:

$$g_1 = \gamma_{slab} \times thickness = 18 \times 0.1 = 1.8\, {kN}/{m^2}$$

Screed:

$$g_2 = g_{screed} = 0.6\, {kN}/{m^2}$$

Flooring:

$$g_3 = g_{floof} = 0.4\, {kN}/{m^2}$$

Masonry partitions (considering the partitions equally distributed on the area):

$$g_4 = g_{partitions} = 0.4\, {kN}/{m^2}$$

$$G_{sc} = g_1 + g_2 + g_3 + g_4 = 3.2\, {kN}/{m^2}$$

5.1.7.2. Live loads

Live loads (Q) affecting the beams (in [kN/m]) are computed as the product of the live loads suggested by the code (in [kN/m^2]) and their impact depths (in [m]) on the beam.

First story:

$Q_I = 3.0\, kN/m^2$ (Building subject to crowding);

Roof:

$Q_R = 0.5\, kN/m^2$ (Roof doesn't be inaccessible);

Stair cases:

$Q_{sc} = 4.0\, kN/m^2$ (Stair cases subject to crowding).

5.1.7.3. Seismic loads

Seismic load (W) is computed as the sum of dead loads and live loads multiplied with a reduction factor. Dead loads are the product of the volume and the specific load of the sum of beams at the respective floor with half of the total column height at the up level and at the down level. The seismic load is divided by gravity acceleration to obtain the seismic mass (m).

First story:

Slab self weight:

$$g_1 = \gamma_{slab} \times thickness \times A_{slab} = 18 \times 0.1 \times 841.5 = 1514.7kN$$

Screed:

$$g_2 = g_{screed} \times A_{slab} = 0.6 \times 841.5 = 504.9kN$$

Flooring:

$$g_3 = g_{floof} \times A_{slab} = 0.4 \times 841.5 = 336.6kN$$

Masonry partitions (considering partitions equally distributed on the area):

$$g_4 = g_{partitions} \times A_{slab} = 0.4 \times 841.5 = 336.6kN$$

R.C. beams $0.20 \times 0.35\, m^2$

$$g_5 = \gamma_{cls} \times V = 25 \times (5 \times 56.7 \times 0.20 \times 0.35) = 496.13\, kN$$

$$g_6 = \gamma_{cls} \times V = 25 \times (4 \times 3.0 \times 0.20 \times 0.35) = 21.0\, kN$$

$$g_7 = \gamma_{cls} \times V = 25 \times (22 \times 5.0 \times 0.20 \times 0.35) = 192.5\, kN$$

Columns in R.C. $0.20 \times 0.20\, m^2$

$$g_8 = \gamma_{cls} \times \frac{V}{2} = 25 \times \frac{(118 \times 3.0 \times 0.20 \times 0.20)}{2} = 177.0\,kN$$

$$g_9 = \gamma_{cls} \times \frac{V}{2} = 25 \times \frac{(48 \times 2.8 \times 0.20 \times 0.20)}{2} = 67.2\,kN$$

$$g_{10} = \gamma_{cls} \times \frac{V}{2} = 25 \times \frac{(44 \times 3.03 \times 0.20 \times 0.20)}{2} = 66.7\,kN$$

$$g_{11} = \gamma_{cls} \times \frac{V}{2} = 25 \times \frac{(24 \times 3.8 \times 0.20 \times 0.20)}{2} = 45.6\,kN$$

$$G_{I\,level} = g_1 + g_2 + g_3 + g_4 + g_5 + g_6 + g_7 + g_8 + g_9 + g_{10} + g_{11} = 3759.0\,kN$$

$$Q_{I\,level} = q \times A_{roof} = 3.0 \times 841.52 = 2524.6\,kN$$

$$W_{I\,level} = G_{I\,level} + \psi_2 \times Q_{I\,level} = 3759.0 + 0.6 \times 2524.6 = 5273.7\,kN$$

Seismic mass: $m_{I\,level} = \frac{W_{I\,level}}{g} = 537.6\ tons$

Moment of inertia in rotational direction:

$I_{t,I} = m_{I\,level} \cdot \rho^2 = 156509\, kNm/\sec^2$.

Stair cases I level (both stair cases are equal)*:*

Structural load slab:

$$g_1 = \gamma_{slab} \times thickness \times A_{sc} = 18 \times 0.1 \times 18.26 = 32.9kN$$

Screed:

$$g_2 = g_{screed} \times A_{sc} = 0.6 \times 18.26 = 11.0kN$$

Flooring:

$$g_3 = g_{floof} \times A_{sc} = 0.4 \times 18.26 = 7.3kN$$

Masonry partitions (considering partitions equally distributed on the area):

$$g_4 = g_{partitions} \times A_{sc} = 0.4 \times 18.26 = 7.3kN$$

R.C. beams $0.20 \times 0.20\,m^2$

$$g_5 = \gamma_{cls} \times V = 25 \times (3 \times 2.7 \times 0.20 \times 0.20) = 8.1\,kN$$

$$g_6 = \gamma_{cls} \times V = 25 \times (2 \times 2.2 \times 0.20 \times 0.20) = 4.4\,kN$$

Columns in R.C. $0.20 \times 0.20\,m^2$

$$g_7 = \gamma_{cls} \times \frac{V}{2} = 25 \times \frac{(6 \times 3.0 \times 0.20 \times 0.20)}{2} = 9.0\,kN$$

$$g_8 = \gamma_{cls} \times \frac{V}{2} = 25 \times \frac{(6 \times 2.28 \times 0.20 \times 0.20)}{2} = 6.8\,kN$$

$$G_{sc\,I\,level} = g_1 + g_2 + g_3 + g_4 + g_5 + g_6 + g_7 + g_8 = 86.8\,kN$$

$$Q_{sc\,I\,level} = q \times A_{sc} = 4.0 \times 18.26 = 73.0\,kN$$

$$W_{sc\,I\,level} = G_{sc\,I\,level} + \psi_2 \times Q_{sc\,I\,level} = 86.8 + 0.6 \times 73.0 = 130.6\,kN$$

$$m_{I\,level} = \frac{W_{sc\,I\,level}}{g} = 13.3\ ton$$

$$m_{scI\,level} \cdot \rho^2 = 82\,{kNm}/{\sec^2}\,.$$

Stair cases II level (both stare cases are equally)*:*

Structural load slab:

$$g_1 = \gamma_{slab} \times thickness \times A_{sc} = 18 \times 0.1 \times 18.26 = 32.9kN$$

Screed:

$$g_2 = g_{screed} \times A_{sc} = 0.6 \times 18.26 = 11.0kN$$

Flooring:

$$g_3 = g_{floof} \times A_{sc} = 0.4 \times 18.26 = 7.3kN$$

Masonry partitions (considering partitions equally distributed on the area):

$$g_4 = g_{partitions} \times A_{sc} = 0.4 \times 18.26 = 7.3kN$$

R.C. beams $0.20 \times 0.20\,m^2$

$$g_5 = \gamma_{cls} \times V = 25 \times (6 \times 2.7 \times 0.20 \times 0.20) = 16.2\,kN$$

$$g_6 = \gamma_{cls} \times V = 25 \times (2 \times 2.2 \times 0.20 \times 0.20) = 4.4\,kN$$

Columns in R.C. $0.20 \times 0.20\,m^2$

$$g_7 = \gamma_{cls} \times \frac{V}{2} = 25 \times \frac{(6 \times 2.28 \times 0.20 \times 0.20)}{2} = 6.8\,kN$$

$$G_{sc\,II\,level} = g_1 + g_2 + g_3 + g_4 + g_5 + g_6 + g_7 = 85.9\,kN$$

$$Q_{sc\,II\,level} = q \times A_{sc} = 4.0 \times 18.26 = 73.0\,kN$$

$$W_{sc\,II\,level} = G_{sc\,II\,level} + \psi_2 \times Q_{sc\,II\,level} = 85.9 + 0.6 \times 73.0 = 129.7\,kN$$

$$m_{II\,level} = \frac{W_{sc\,II\,level}}{g} = 13.2\ ton$$

$$m_{sc\,II\,level} \cdot \rho^2 = 81\,{kNm}/{\sec^2}\,.$$

Roof:

$$\gamma_w = 4\,{}^{kN}\!/_{m^3}$$

Wooden beams $0.075 \times 0.04\,m^2$

$$g_1 = \gamma_w \times V = 4 \times (45 \times 2 \times 8.00 \times 0.075 \times 0.20) = 43.2\,kN$$

Wooden beams $0.04 \times 0.04\,m^2$

$$g_2 = \gamma_w \times V = 4 \times (18 \times 55.00 \times 0.04 \times 0.04) = 6.3\,kN$$

$$\gamma_{cls} = 25\,{}^{kN}\!/_{m^3}$$

R.C. beams $0.20 \times 0.35\,m^2$

$$g_3 = \gamma_{cls} \times V = 25 \times (5 \times 56.7 \times 0.20 \times 0.35) = 496.13\,kN$$

R.C. beams $0.20 \times 0.20\,m^2$

$$g_4 = \gamma_{cls} \times V = 25 \times (4 \times 3.03 \times 0.20 \times 0.20) = 12.12\,kN$$

$$g_5 = \gamma_{cls} \times V = 25 \times (4 \times 5.06 \times 0.20 \times 0.20) = 20.24\,kN$$

Columns in R.C. $0.20 \times 0.20\,m^2$

$$g_6 = \gamma_{cls} \times \frac{V}{2} = 25 \times \frac{(48 \times 2.8 \times 0.20 \times 0.20)}{2} = 67.2\,kN$$

$$g_7 = \gamma_{cls} \times \frac{V}{2} = 25 \times \frac{(44 \times 3.03 \times 0.20 \times 0.20)}{2} = 66.7\,kN$$

$$g_8 = \gamma_{cls} \times \frac{V}{2} = 25 \times \frac{(24 \times 3.8 \times 0.20 \times 0.20)}{2} = 45.6\,kN$$

$$G_{roof} = g_1 + g_2 + g_3 + g_4 + g_5 + g_6 + g_7 + g_8 = 757.5\,kN$$

$$Q_{roof} = q \times A_{roof} = 0.5 \times 967.18 = 483.6\,kN$$

$$W_{roof} = G_{roof} + \psi_2 \times Q_{roof} = 757.5 + 0.6 \times 483.6 = 1047.6\,kN$$

$$m_{roof} = \frac{W_{roof}}{g} = 106.8\ ton$$

Since it is a pitched roof, it can be considered divided into two parts, each part with

a mass $m_{roof(1/2)} = \dfrac{m_{roof}}{2} = 53.4\ tons$

and a Moment of Inertia $m_{roof(1/2)} \cdot \rho^2 = 15062\,{}^{kNm}\!/_{s^2}$.

5.1.8. Structural modelling

Spatial model is realized with the help of calculation software, and it is defined by elements' axis line. Structural model is composed by frame elements connected each others with horizontal diaphragms, given value the hypothesis of rigid diaphragm; so, each level is characterized by three liberty degree, the two translations along both directions (longitudinal and transversal building axis) and the rotation around vertical axes for the mass centre.

In the modelling, it is not considered both the contribute of not structural elements and the section reduction of elements in elevation. Nevertheless, structural model represents the distribution of masses and effective rigidity. Following, it is reported the model:

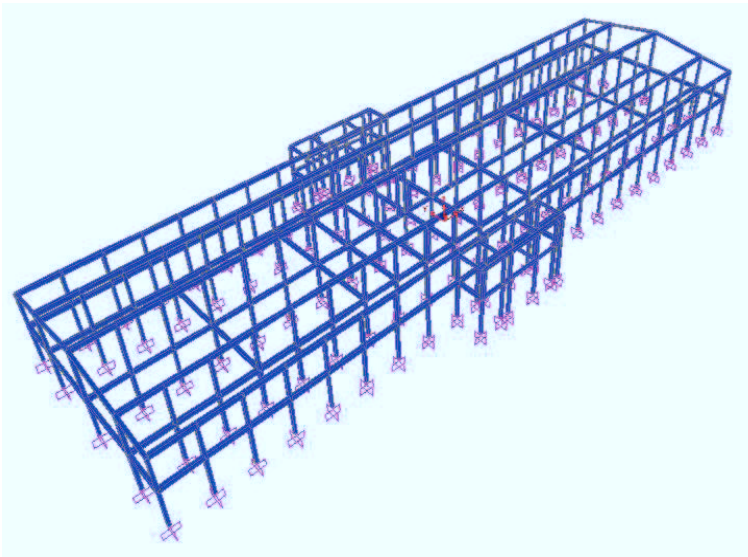


Figure 36. Spatial structural model - School Republica de Colombia

5.1.8.1. Roof modeling

Since the school buildings are characterized by roofs of corrugated metal sheets, two different ways to model the roof are considered. The first model considers the Roof as a Rigid (from now on RR) level with a diaphragm between all joints; the second one considers the Roof Deformable (from now on DR) with wooden beams and shells between the beams.

Figure 37 shows the spatial model considering deformable roof.

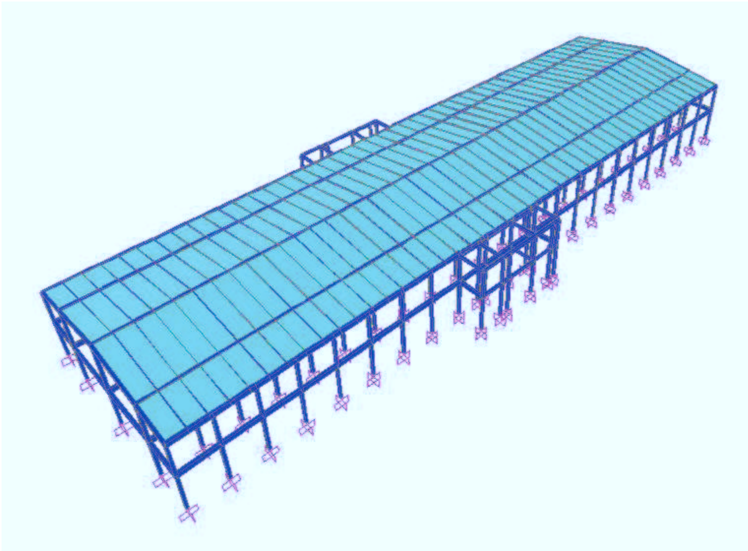


Figure 37. Spatial structural model considering Deformable Roof - School Republica de Colombia

5.1.8.2. Consideration of P-delta effect (Theory 2nd order)

First order analysis assumes small deflection behaviour; the resulting forces and moments take no account of the additional effect due to the deformation of the structure under load.

As the structures become more slender and less resistant to deformation, the need to consider second order P-delta effects arises. The mentioned P-delta effect is a non-linear effect which occurs when elements are subject to axial load. The magnitude of the P-delta effect is related to the magnitude of the axial load, the stiffness/slenderness of the structure as a whole, and the slenderness of individual elements.

The displacement response of the inelastic system subject to second order P-delta actions can often be dominated by the combined influence of the elastic stiffness reduction and more importantly the lower post-yield stiffness that governs the level of plastic deformation.

In case of the described Central American school buildings, all columns are slender elements, so that the consideration of second order effect in the evaluation of capacity function is required. Therefore, the effects of P-delta are implemented as a reduction of a moment rotation relationship in hinge properties. In fact, considering the flexural behaviour in terms of force-displacement with the inclusion of P-delta effects, a reduction of force and stiffness will occur depending on the applied axial load. Consequently, a new relation in terms of moment-rotation is obtained.

With respect to Figure 38, the continuous line represents the hinge characteristics disregarding P-delta effects. The dashed line represents the new moment-rotation relationship considering P-delta effects. The moment is obtained as the difference between the M-θ relationship without P-delta effects and the product of axial load *P*, rotation θ and shear length L_v (grey line).

Clearly, considering P-delta effect reduction the moment-rotation relationship becomes:

Cracking point: $\theta_{cr}\,;\; M_{cr} - L_V \cdot \left(P \cdot \theta_{cr}\right)$

Yielding point: $\theta_y\,;\; M_y - L_V \cdot \left(P \cdot \theta_y\right)$

Maximum point: $\theta_{\max}\,;\; M_{\max} - L_V \cdot \left(P \cdot \theta_{\max}\right)$

Ultimate point: $\theta_u\,;\; M_u - L_V \cdot \left(P \cdot \theta_u\right)$

The last point ordinate of moment-rotation relationship is limited by the following relation, to respect the first hypothesis on the relation:

$$M_u - L_V \cdot \left(P \cdot \theta_u\right) \geq M_{\max} - L_V \cdot \left(P \cdot \theta_{\max}\right) \tag{38}$$

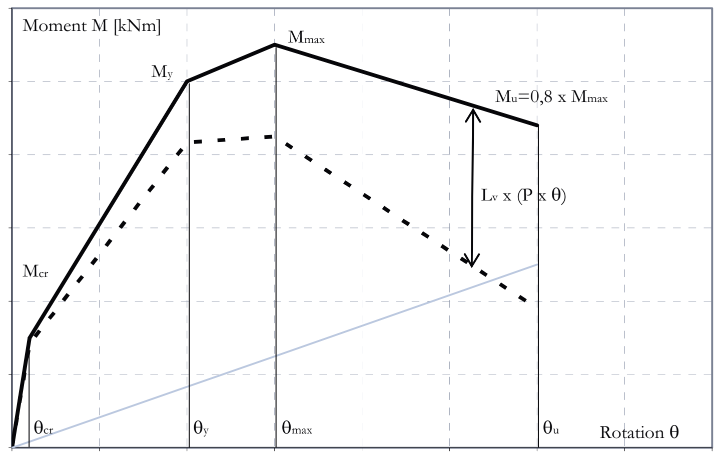


Figure 38. Moment–rotation relationship considering the reduction done to P-delta effect

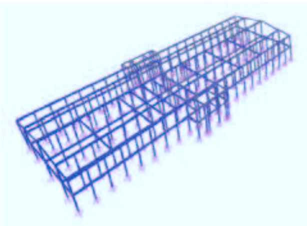
An analysis not considering P-delta effect is done to report the structural behaviour in case of with or without P-delta evaluation.

The consideration of P-delta effects in the analysis (grey curves) lowers the lateral strength and clearly alters the effective periods.

In which way these changes affect the structural behaviour is demonstrated by Figure 42 and Figure 43 which represent the capacity curves for both principal building axes.

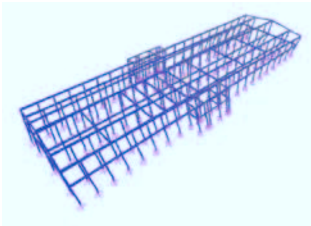
5.1.9. Building's dynamic property

Building dynamic elastic characters' evaluation is conducted with modal analysis on structural model as above mentioned. Analysis is done considering totality of vibration mode of spatial model. Following, it is reported the first three vibration modes with relative evaluated periods, for both kind of models RR and DR:



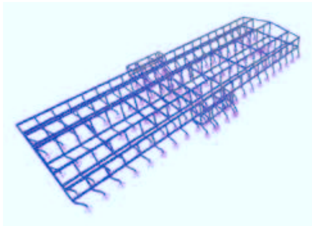
I mode

$T = 0.51 s$



II mode

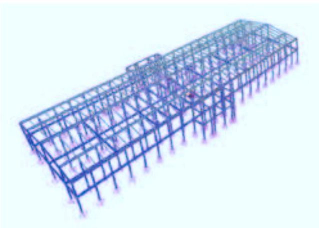
$T = 0.44 s$



III mode

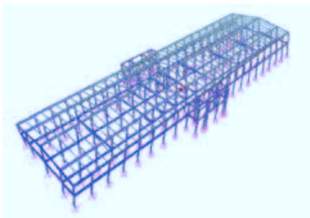
$T = 0.37 s$

Table 28. Period T of first three vibration modes with tri-dimensional view (RR) – School Republica de Colombia



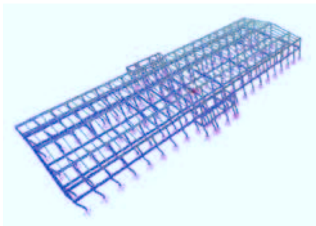
I mode

$T = 0.80 s$



II mode

$T = 0.71 s$



III mode

$T = 0.55 s$

Table 29. Period T of first three vibration modes with tri-dimensional view (DR) – School Republica de Colombia

Table 30 and Table 31 shows periods of 12 vibration modes and associated participating masse, in longitudinal and transversal building axis, and the rotation component.

Mode	Period [sec]	Participation masses [%]		
		X	Y	XY
1	0.51	0.00	86.36	3.91
2	0.44	0.00	3.89	90.91
3	0.37	99.15	0.00	0.00
4	0.20	0.00	9.54	0.22
5	0.17	0.00	0.21	4.96
6	0.13	0.85	0.00	0.00
7	0.03	0.00	0.00	0.00
8	0.03	0.00	0.00	0.00
9	0.02	0.00	0.00	0.00
10	0.02	0.00	0.00	0.00
11	0.01	0.00	0.00	0.00
12	0.01	0.00	0.00	0.00

Table 30. First 12 vibration modes (RR) – School Republica de Colombia

Mode	Period [sec]	Participation masses [%]		
		X	Y	XY
1	0.80	0.00	83.57	4.40
2	0.71	0.00	4.54	85.44
3	0.55	94.96	0.00	0.00
4	0.23	0.00	11.04	0.00
5	0.22	0.00	0.74	9.44
6	0.19	5.04	0.00	0.00
7	0.19	0.00	0.00	0.00
8	0.13	0.00	0.00	0.00
9	0.10	0.00	0.00	0.00
10	0.09	0.00	0.00	0.00
11	0.09	0.00	0.00	0.00
12	0.07	0.00	0.00	0.00

Table 31. First 12 vibration modes (DR) – School Republica de Colombia

As shown, model with deformable roof is more deformable than this one with rigid roof.

5.1.10. Nonlinear static analysis

As above mentioned (3.3), the analysis consists in the application of dead loads to building, as in seismic combination, and an adequately system of horizontal static forces that, increasing, give a monotonic grow in horizontal displacement of control point, generally in the middle of the roof.

Methodology consists in:

- Knowledge of a force-displacement relationship generalized between the resulting force, base shear V_b, and the displacement d_c of a control point, usually chosen in the centre of the roof level;
- Characters determination of a single degree of freedom system, called SDOF, with equivalent bilinear behaviour;
- Determination of maximum response in displacement of SDOF system with the use of displacement spectrum;
- Conversion of equivalent displacement system as above mentioned in the effective building's configuration;
- Verification of displacement compatibility (elements/ductile mechanism) and resistances (elements/fragile mechanisms).

Particularly, it has to apply to building, the second of two different distributions of horizontal forces applied in the centre of mass point at each level, following related:

1. a force distribution proportional to masses, applied separately in both analysis' directions, longitudinal and transversal one;
2. a force distribution proportional to masses for deformed correspondent to first vibration mode in both analysis directions, longitudinal and transversal, applied separately.

In Table 32 values combined for distribution, as above mentioned, are reported:

Level	***Mass*** $\left[kN \cdot s^2/m\right]$	***Mode 1X (RR)***	***Mode 1Y (RR)***	***Mode 1X (DR)***	***Mode 1Y (DR)***
1	564.20	0.80	0.49	0.62	0.46
Roof stair case 1	26.44	0.99	0.92	0.99	0.98
Roof stair case 2	26.44	0.99	0.92	0.99	0.98
2 (flake 1)	106.80	1.00	1.00	1.00	1.00
2 (flake 2)	106.80	1.00	1.00	1.00	1.00

Table 32. Masses and first vibration mode in both analysis cases (RR and DR) – School Republica de Colombia

5.1.10.1. Nonlinear modelling

Spatial model has been realized with a computational software and it is defined by axis line of elements, considering rigidity of joint with rigid pieces at the end of elements. Structural model is constituted by frame elements connected by horizontal diaphragms, given true the hypothesis of infinitely rigid level.

Linear elastic behaviour of elements is evaluated considering the geometric dimensions (transversal section and length) and mechanical characters of materials (elastic modulus of concrete). The post-elastic behaviour (nonlinear) is evaluated by a lumped plastic model; this model considers plasticity concentrated in plastic hinges at the end of elements. Here, after the elastic limitation, inelastic deformations are concentrated.

The advantages of this kind of modelling are simplicity and computational efficiency. One of model limitation is given by a fixed point of inflexion point during analysis (prefixed shear length L_V). Lumped plasticity model doesn't allow the computation of plastic hinge in the middle of element, caused by interaction of gravitational loads and horizontal ones.

Nevertheless, a chosen characteristic curve of plastic hinge allows to describe various phenomenon as flexional behaviour, shear deformability, steel wrapping. Definition of curve is effected by nonlinear behaviour of end section of element and shear length L_V. Particularly, in the hypothesis of not consider gravitational loads' effects, moment distribution is linear so the element can be seen as a bracket of length L_V with a force on the free end section.

Generally, the nonlinear behaviour of end section, defined by moment curvature relationship $M-\phi$, can be express with a quadri-linear relationship defined by: a first phase linear elastic until the first cracking (ϕ_{cr}, M_{cr}); a second phase (cracking one) during the formation of other crack until the yielding (ϕ_y, M_y); a third phase post-elastic characterized by a rigidity decrease, so a deformability increase until a top level of flexional strength (ϕ^*, $M_{\max}$) and a decreasing phase characterized by a reduction of strength capacity and a high deformation capacity until of ultimate condition (ϕ_u, $\alpha \cdot M_{\max}$), adopted $\alpha = 0.8$. So, the relationship moment-rotation adopted for the element is shown in Figure 6.

Evaluation of shear length L_V is not easy; so a linear analysis is adopted to know the inflection point in the linear structural behaviour. Generally shear length can be considered equal to half of the element length ($L_V = 0.5 \cdot L$). In this case shear length adopted is equal to half of the element length for columns, and is the null point of shear forces in beams.

5.1.11. Force – displacement relationship

Defined plastic hinge properties, for each element beam and column, nonlinear analysis is done. Analysis proceeds with force control, increasing horizontal forces with a coefficient, so the structural behaviour is described until the top of capacity curve.

Results of nonlinear analysis are reported following, referring to force distribution proportional to product of masses for deformed shape at first vibration mode. By way of illustration, it is reported only one analysis result in positive transversal direction, considering RR.

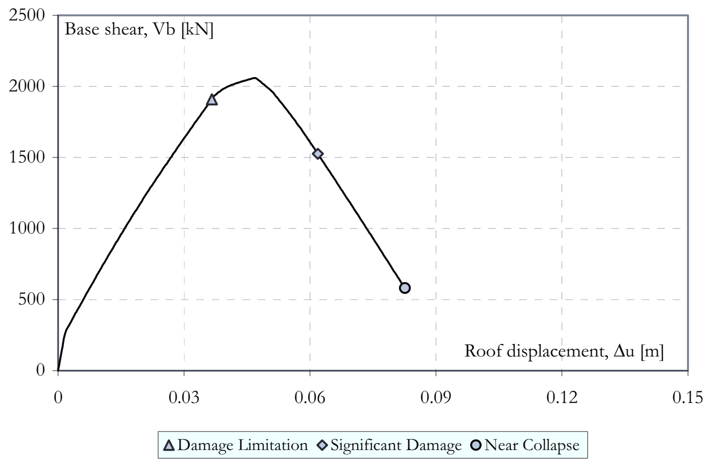


Figure 39. Capacity curve in longitudinal building axis – School Republica de Colombia

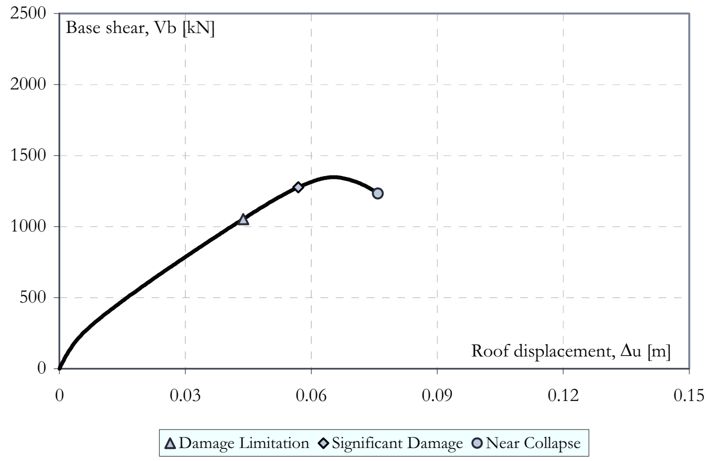


Figure 40. Capacity curve in transversal building axis – School Republica de Colombia

Figure 39 and Figure 40 show the relationship between structural base shear V_b and the control point displacement Δ_u, chosen coincident with the mass centre of roof.

5.1.12. Equivalent bilinear system

Equivalent bilinear system is evaluated to know structural displacement demand for each limit state. Therefore, it is essential to transform multiple degree of freedom system (from now on MDOF) in a single degree of freedom (from now on SDOF), as above mentioned (3.3.1.5). Participating factor Γ is evaluated:

$$\Gamma = \frac{\sum m_i \cdot \Phi_i}{\sum m_i \cdot \Phi_i^2} \tag{39}$$

where Φ is the displacement vector representative of structural deformed shape during the first vibration mode in the considered building axis, normalized to unit value; m_i is the mass at building level i and the sum at numerator represents generalized mass (equivalent mass of SDOF). In this case participating factor in both direction is:

Building axis	***m* [ton]***	***Γ***
X (RR)	585	1.18
Y (RR)	409	1.54
X (DR)	481	1.38
Y(DR)	394	1.56

Table 33. SDOF generalized mass m and modal participation factor Γ for RR and DR models – School Republica de Colombia*

MDOF capacity curve has to be scaled by participating factor Γ to determine the SDOF force – displacement curve ($F^* - d^*$) with the following relationship:

$$F^* = {V_b}/{\Gamma} \tag{40}$$

$$d^* = {\Delta_u}/{\Gamma} \tag{41}$$

Known SDOF system $F^* - d^*$ curve, it is necessary to define an equivalent bilinear low force – displacement, the adopted procedure is this one of areas' equivalence. In this way, rigidity k^*, and consequently elastic period T^* of SDOF system can be obtained by the following equation:

$$T^* = 2\pi\sqrt{\frac{m^*}{k^*}} \tag{42}$$

where m^* is the generalized mass equal to $\sum m_i \cdot \Phi_i$.

In Figure 41 SDOF a force – displacement curves are reported.

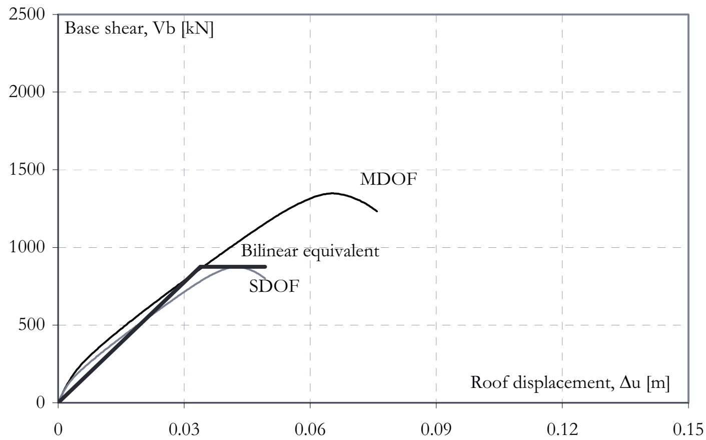


Figure 41. SDOF in transversal building axis (RR model) - School Republica de Colombia

5.1.13. Results

Considering variability in mechanical thresholds, Capacity Curves are obtained. The following figures describe the structural capacity.

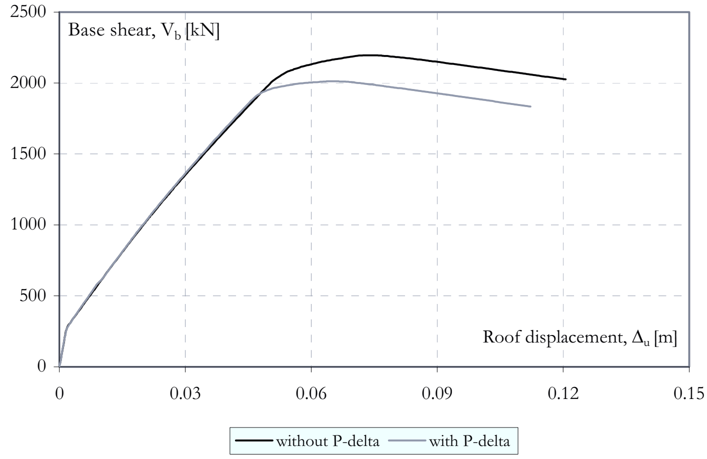


Figure 42. Capacity curve for the longitudinal building axis with and without consideration of P-delta effect - School Republica de Colombia

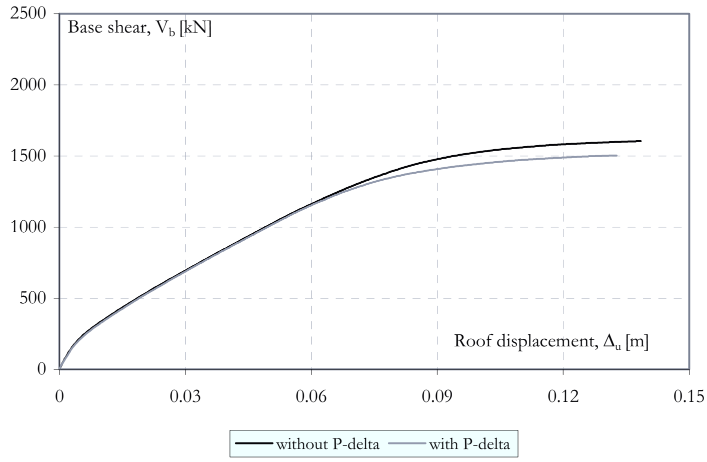


Figure 43. Capacity curve for the transversal building axis with and without consideration of P-delta effect - School Republica de Colombia

By way of illustration, only analysis results in positive directions are reported.

Non-linear static analysis has been applied to 25 structural models generated combining lognormal distribution of yielding and plastic rotation values, as above mentioned; Table 34 shows various structural models given by combination considering 16, 34, 50, 66 and 84 percentiles for both rotations. Each structure is been analyzed considering the two kind of roofs: rigid roof (RR) and deformable one (DR).

		$\theta_{pl\ percentile}$				
		0.16	***0.34***	***0.50***	***0.66***	***0.84***
$\theta_{y\ percentile}$	***0.16***	1	2	3	4	5
	0.34	6	7	8	9	10
	0.50	11	12	13	14	15
	0.66	16	17	18	19	20
	0.84	21	22	23	24	25

Table 34. Models generated considering variability in rotations – School Republica de Colombia

The variability in output for pushover analysis is investigated for both the structural model with rigid roof (RR) and deformable roof (DR). Figure 44 and Figure 45 illustrate the increasing displacement capacity, in both directions, corresponding to the plastic rotation variability adopted in the mechanical characterization of the structural elements for the RR case. As it could be expected, given yield rotation, structural displacement capacity increases with the increasing of plastic rotation.

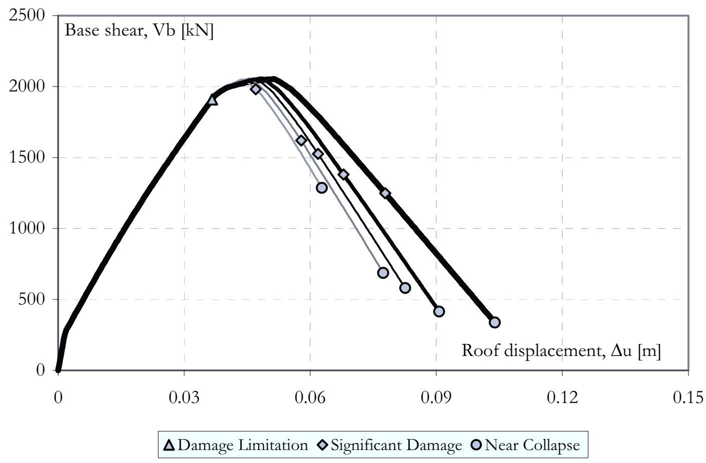


Figure 44. Capacity curves in the longitudinal building axis for (RR), considering P-delta effect for mean yield rotation with increasing plastic rotation - School Republica de Colombia

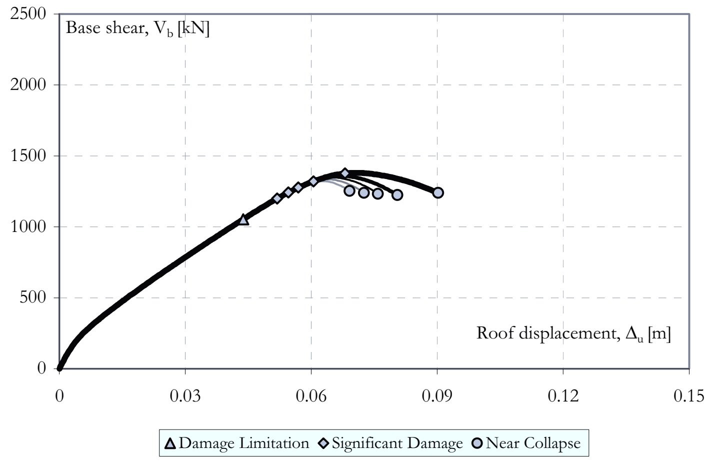


Figure 45. Capacity curves in the transversal building axis for (RR), considering P-delta effect for mean yield rotation with increasing plastic rotation - School Republica de Colombia

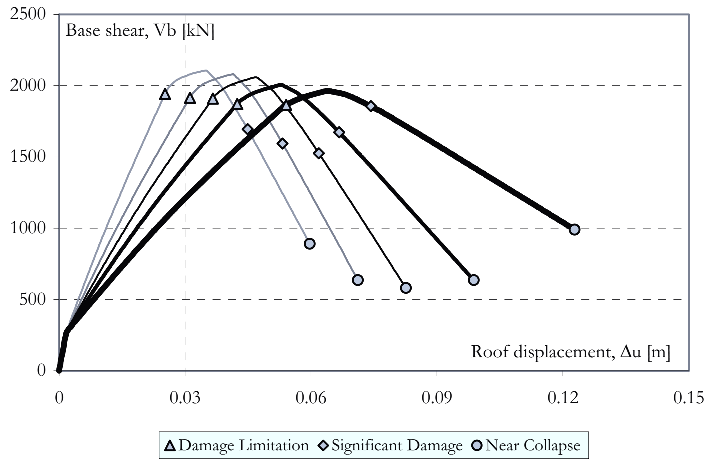


Figure 46. Capacity curves in the longitudinal building axis for (RR), considering P-delta effect for mean plastic rotation with increasing yield rotation - School Republica de Colombia

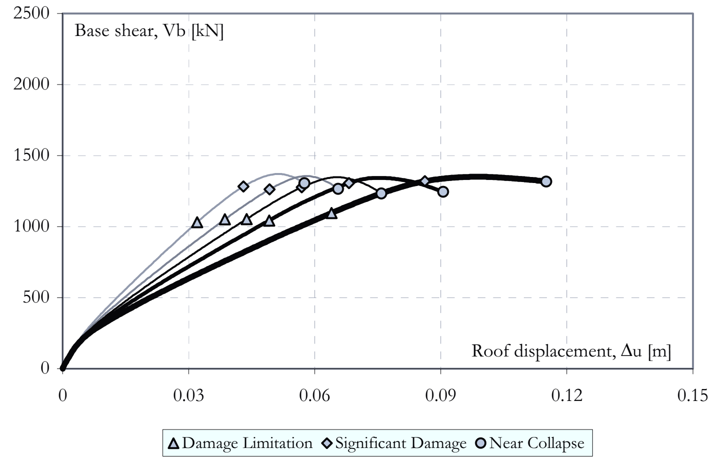


Figure 47. Capacity curves in the transversal building axis for (RR), considering P-delta effect for mean plastic rotation with increasing yield rotation - School Republica de Colombia

Figure 46 and Figure 47 illustrate the increase of the effective period through the system's stiffness corresponding to the yield rotation variability adopted in the mechanical characterization of the structural elements. For a given plastic rotation and considering adopted variability in yielding one, structural deformability increases as yield rotation also increases. Evidently, the global stiffness reduction leads to the increasing, of the system's effective period, as it will be clarified in the following. Figure 48, Figure 49, Figure 50 and Figure 51 show the same analysis results in both building directions referring to deformable roof (DR) case. Similar considerations apply.

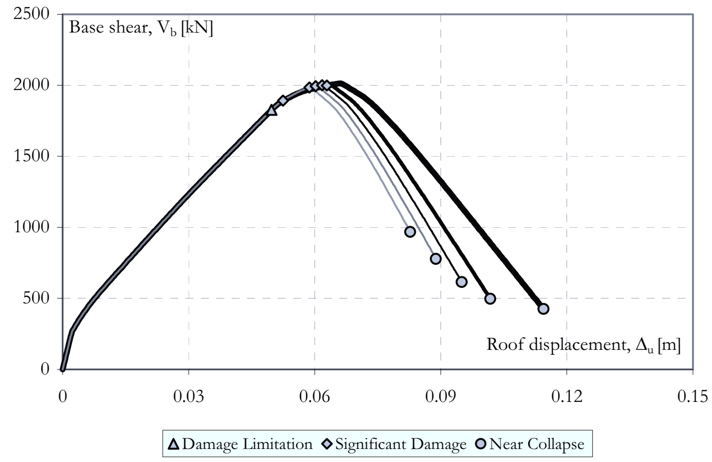


Figure 48. Capacity curves in the longitudinal building axis for (DR) considering P-delta effect for mean yield rotation with increasing plastic rotation - School Republica de Colombia

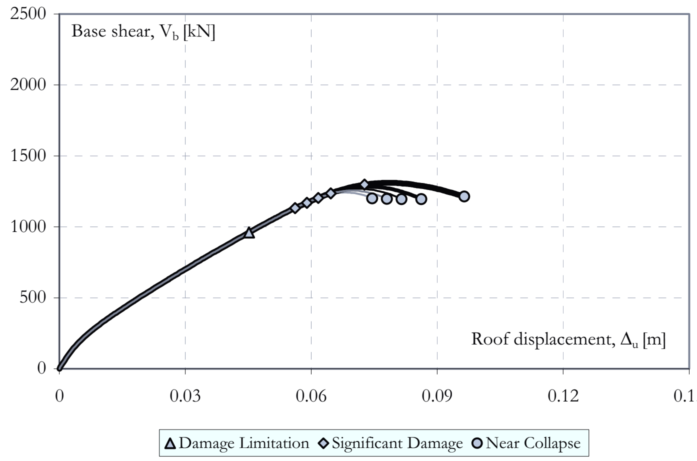


Figure 49. Capacity curves in the transversal building axis for (DR), considering P-delta effect for mean yield rotation with increasing plastic rotation - School Republica de Colombia

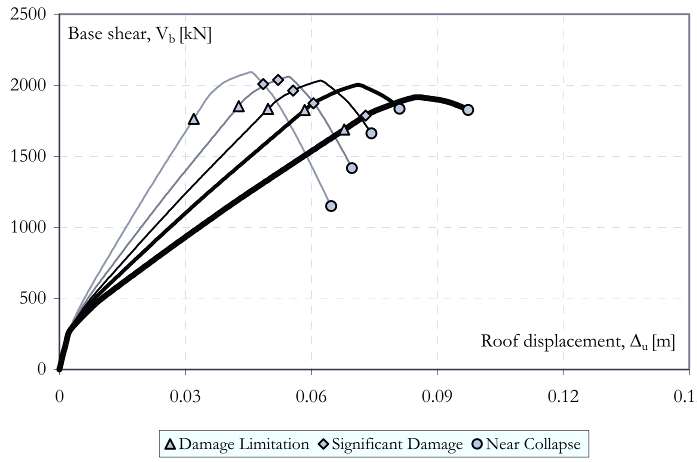


Figure 50. Capacity curves in the longitudinal building axis for (DR), considering P-delta effect for mean plastic rotation with increasing yield rotation School Republica de Colombia

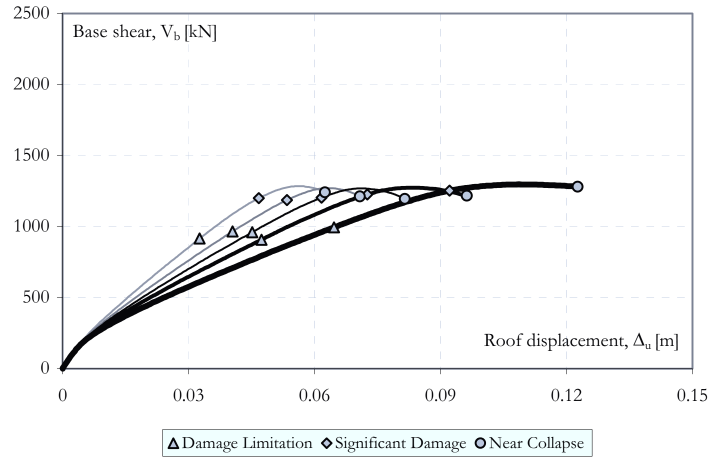


Figure 51. Capacity curves in the transversal building axis for (DR), considering P-delta effect for mean plastic rotation with increasing yield rotation - School Republica de Colombia

The 'pushover' capacity curve, which represents the nonlinear relationship between base shear and roof displacement for the Multi Degree Of Freedom (MDOF) system analyzed, is utilized as an instrument to characterize the force-displacement curve of a Single Degree Of Freedom (SDOF) system that is considered to be dynamically equivalent to the MDOF. Different formulations exist to institute the MDOF-SDOF equivalence. Here, we refer to the method suggested in Fajfar [34] which is adopted by Eurocode 8 [29]. The force-displacement curve for a SDOF, suitably idealized, allows the identification of the "capacity parameters" in terms of limit state displacement C_d, base shear coefficient C_s and effective period T which are necessary to evaluate the seismic demand with a spectral approach.

Starting from the results of the pushover analyses the range of variability of the capacity parameters C_d, C_s and T for the different damage levels investigated are obtained.

In particular, the displacement capacity for DL, SD and NC limit states ranges, between, 0.0214-0.0454 m, 0.0285-0.0812 m and 0.038-0.1039 m for the RR case in the longitudinal direction. In transversal direction, the ranges are 0.0208-0.0416 m for DL, 0.0235-0.0671 m for SD and 0.0320-0.0810 for NC. With regard to effective periods, the ranges are 0.5246-0.7572 sec in longitudinal direction and 0.6891-0.9252 sec in transversal one. The capacity strength Cs has generally little variation, ranging from 0.293-0.310 g in longitudinal direction and 0.214-0.221 g in the transversal one.

Non-linear parameters for RR structural models in longitudinal building axis and transversal one, are reported in Table 35 and Table 36, respectively.

In case of DR model, instead, the displacement capacity ranges, are 0.0232-0.0529 m for DL, 0.0292-0.0562 m for SD and 0.0390-0.1006 m for NC, in longitudinal building axis. In transversal direction, the ranges are 0.0209-0.0414 m for DL, 0.0253-0.0639 m for SD and 0.0337-0.0852 for NC. As it can be observed, the displacement capacities for the equivalent SDOF systems in the case of RR in longitudinal direction are higher with respect to DR ones, differently from what it could be expected by the pushover curves. This variation is due the different values of the modal participation coefficients Γ, obtained for the two kind of model in longitudinal direction (Table 33).

For the effective periods, the ranges are 0.5637-0.8157 sec in longitudinal direction and 0.730-0.968 sec in transversal one. The capacity strength ranges are characterized, in this case too, by just a small variation; in fact, in longitudinal direction the range is 0.3007-0.318 g, in transversal one the range is 0.205-0.217 g. Non-linear parameters for DR structural models in longitudinal building axis and transversal one, are reported in Table 37 and Table 38, respectively.

As it could be expected, effective periods are higher for DR models than for RR ones.

	ϖ_y	ϖ_{pl}	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{NC} [m]
1	0.16	0.16	0.5246	0.3103	0.0214	0.0285	0.0380
2	0.16	0.34	0.5252	0.3106	0.0214	0.0339	0.0452
3	0.16	0.50	0.5255	0.3105	0.0214	0.0379	0.0505
4	0.16	0.66	0.5257	0.3103	0.0214	0.0448	0.0598
5	0.16	0.84	0.5257	0.3103	0.0214	0.0537	0.0716
6	0.34	0.16	0.5811	0.3058	0.0264	0.0364	0.0485
7	0.34	0.34	0.5819	0.3065	0.0264	0.0402	0.0535
8	0.34	0.50	0.5823	0.3066	0.0264	0.0452	0.0602
9	0.34	0.66	0.5825	0.3063	0.0264	0.0428	0.0570
10	0.34	0.84	0.5826	0.3058	0.0264	0.0596	0.0795
11	0.50	0.16	0.6260	0.3024	0.0310	0.0398	0.0531
12	0.50	0.34	0.6269	0.3032	0.0310	0.0491	0.0655
13	0.50	0.50	0.6273	0.3035	0.0310	0.0524	0.0699
14	0.50	0.66	0.6277	0.3034	0.0310	0.0576	0.0768
15	0.50	0.84	0.6277	0.3027	0.0310	0.0660	0.0879
16	0.66	0.16	0.6758	0.2984	0.0361	0.0436	0.0581
17	0.66	0.34	0.6766	0.2992	0.0361	0.0446	0.0595
18	0.66	0.50	0.6772	0.2997	0.0361	0.0438	0.0584
19	0.66	0.66	0.6776	0.2997	0.0361	0.0536	0.0714
20	0.66	0.84	0.6777	0.2991	0.0361	0.0489	0.0652
21	0.84	0.16	0.7553	0.2926	0.0454	0.0685	0.0914
22	0.84	0.34	0.7558	0.2928	0.0454	0.0634	0.0846
23	0.84	0.50	0.7564	0.2932	0.0454	0.0629	0.0838
24	0.84	0.66	0.7569	0.2935	0.0454	0.0570	0.0760
25	0.84	0.84	0.7572	0.2932	0.0454	0.0711	0.0948

Table 35. Non-linear parameters in the longitudinal building axis for (RR) - School Republica de *Colombia*

	ϖ_y	ϖ_{pl}	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{NC} [m]
1	0.16	0.16	0.6893	0.2139	0.0208	0.0237	0.0316
2	0.16	0.34	0.6920	0.2190	0.0208	0.0260	0.0347
3	0.16	0.50	0.6939	0.2222	0.0208	0.0281	0.0374
4	0.16	0.66	0.6958	0.2251	0.0208	0.0309	0.0412
5	0.16	0.84	0.6984	0.2285	0.0208	0.0369	0.0492
6	0.34	0.16	0.7422	0.2134	0.0251	0.0282	0.0376
7	0.34	0.34	0.7444	0.2171	0.0251	0.0300	0.0400
8	0.34	0.50	0.7462	0.2198	0.0251	0.0320	0.0426
9	0.34	0.66	0.7480	0.2225	0.0251	0.0345	0.0460
10	0.34	0.84	0.7508	0.2260	0.0251	0.0397	0.0529
11	0.50	0.16	0.7864	0.2144	0.0285	0.0337	0.0449
12	0.50	0.34	0.7880	0.2166	0.0285	0.0354	0.0471
13	0.50	0.50	0.7894	0.2186	0.0285	0.0370	0.0493
14	0.50	0.66	0.7910	0.2206	0.0285	0.0392	0.0523
15	0.50	0.84	0.7938	0.2239	0.0285	0.0440	0.0586
16	0.66	0.16	0.8370	0.2156	0.0319	0.0411	0.0548
17	0.66	0.34	0.8378	0.2166	0.0319	0.0425	0.0567
18	0.66	0.50	0.8388	0.2178	0.0319	0.0441	0.0589
19	0.66	0.66	0.8401	0.2193	0.0319	0.0459	0.0612
20	0.66	0.84	0.8427	0.2219	0.0319	0.0495	0.0661
21	0.84	0.16	0.9285	0.2234	0.0416	0.0549	0.0732
22	0.84	0.34	0.9239	0.2198	0.0416	0.0552	0.0736
23	0.84	0.50	0.9231	0.2191	0.0416	0.0561	0.0748
24	0.84	0.66	0.9231	0.2192	0.0416	0.0575	0.0767
25	0.84	0.84	0.9246	0.2205	0.0416	0.0607	0.0809

Table 36. Non-linear parameters in the transversal building axis for (RR) - School Republica de Colombia

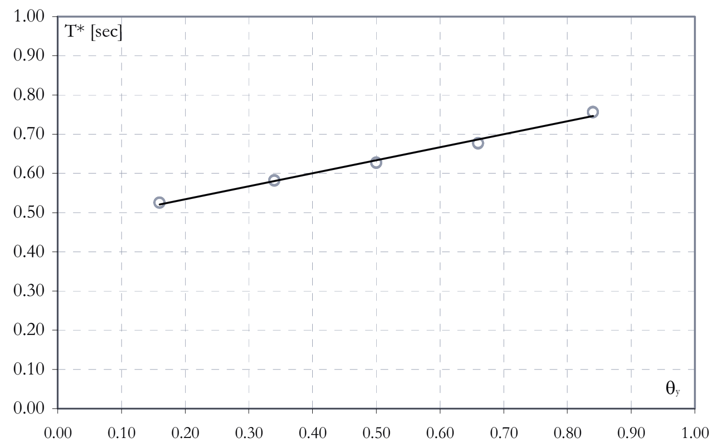


Figure 52. Effective period versus yielding rotation for RR models in longitudinal direction - School Republica de Colombia

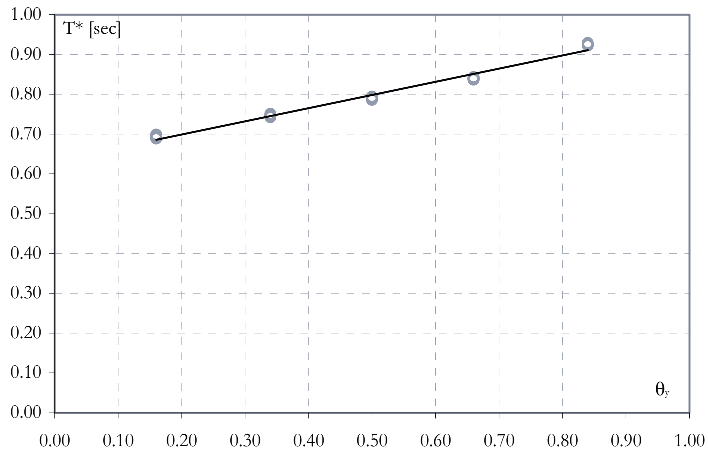


Figure 53. Effective period versus yielding rotation for RR *models in transversal direction - School Republica de Colombia*

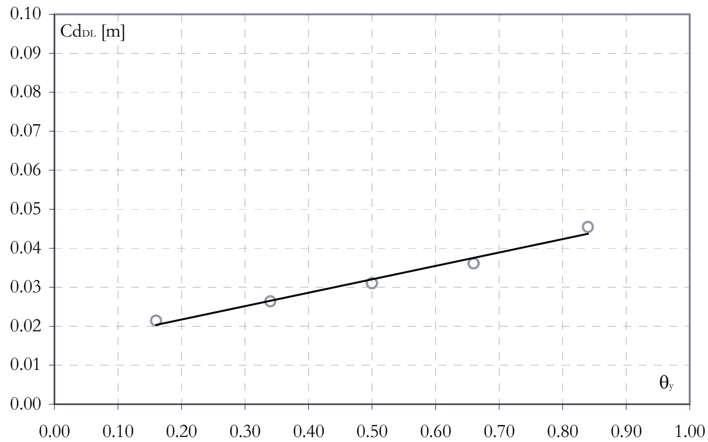


Figure 54. Cd_{DL} versus yielding rotation for RR *models in longitudinal direction - School* Republica *de Colombia*

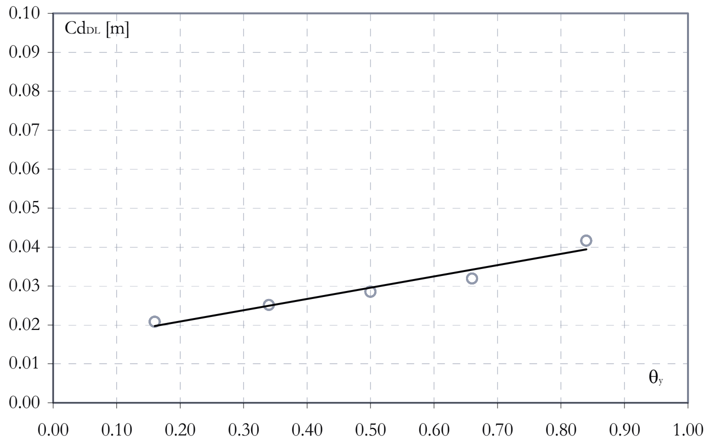


Figure 55. Cd_{DL} versus yielding rotation for RR *models in transversal direction - School* Republica *de Colombia*

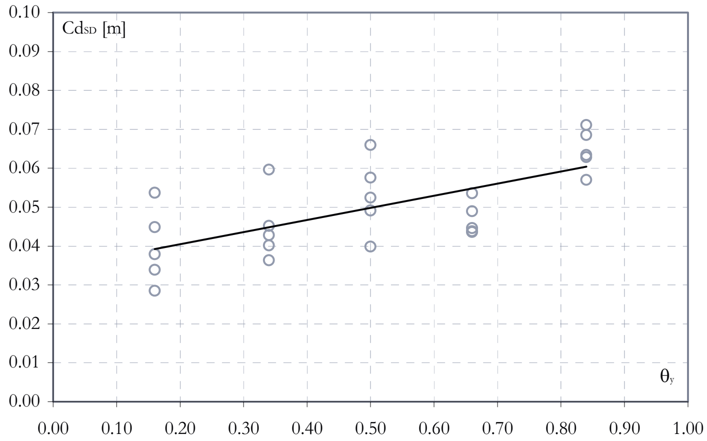


Figure 56. Cd_{SD} versus yielding rotation for RR *models in longitudinal direction - School* Republica *de Colombia*

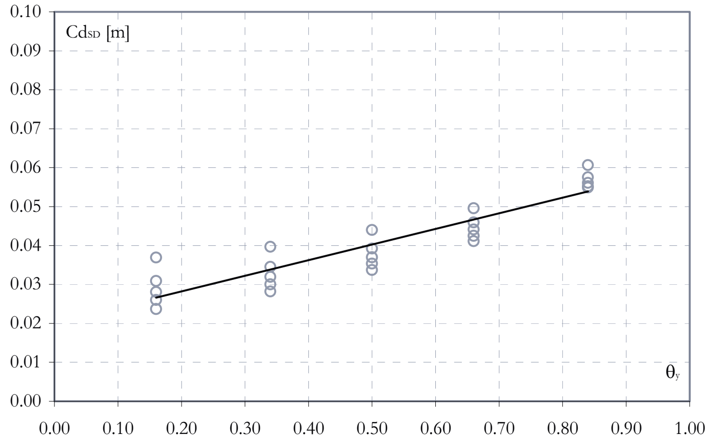


Figure 57. Cd_{SD} versus yielding rotation for RR models in transversal direction - School Republica de Colombia

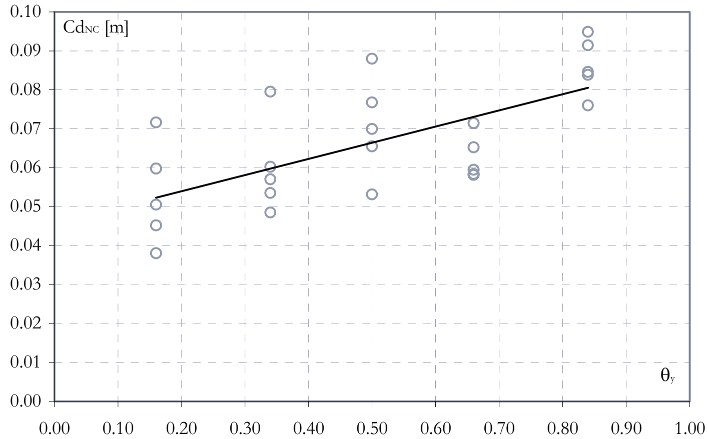


Figure 58. Cd_{NC} versus yielding rotation for RR models in longitudinal direction - School Republica de Colombia

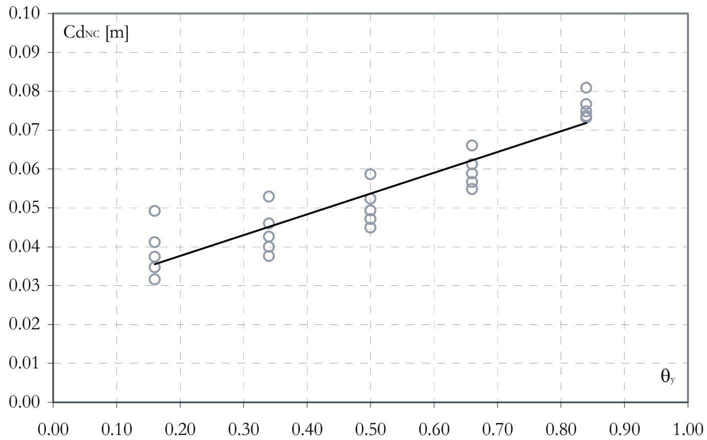


Figure 59. Cd_{NC} versus yielding rotation for RR *models in transversal direction - School Republica de Colombia*

Figure 56, Figure 57, Figure 58 and Figure 59 show a higher dispersion in results, it is easily understanding that for several damage and near collapse limit states, there is a major influence of plastic part or chord rotation. Following, different kind of mechanical response is reported; in longitudinal direction, there is a soft storey level (the first one), as shown in Figure 60 and Figure 61; instead, in transversal direction, a global mechanism characterize structural behaviour (Figure 62 and Figure 63). The same mechanisms arrives in nonlinear analysis of DR structural models for both directions.

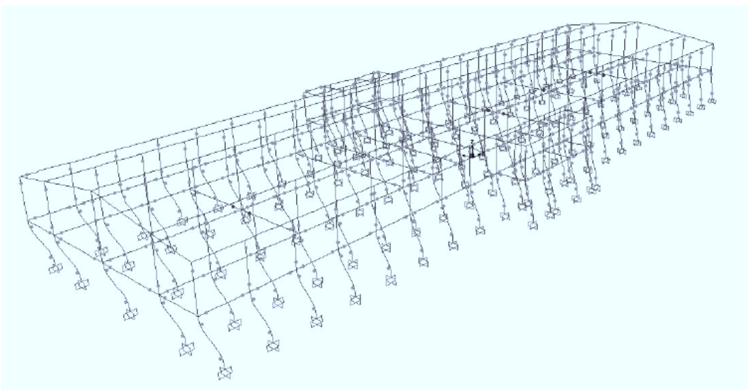


Figure 60. Plastic hinges in tri-dimension RR *structural model in nonlinear analysis in longitudinal direction - School Republica de Colombia*

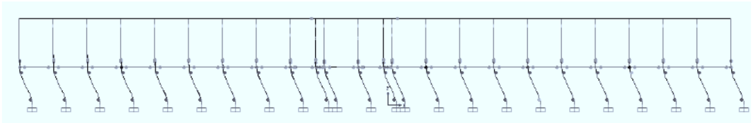


*Figure 61. Soft storey mechanism in longitudinal direction (*RR *structural model) School* Republica *de Colombia*

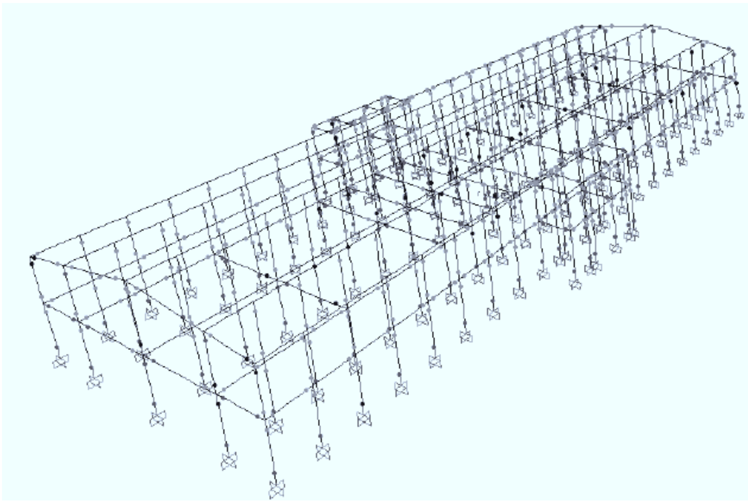


Figure 62. Plastic hinges in tri-dimension RR *structural model in nonlinear analysis in transversal direction - School* Republica de Colombia

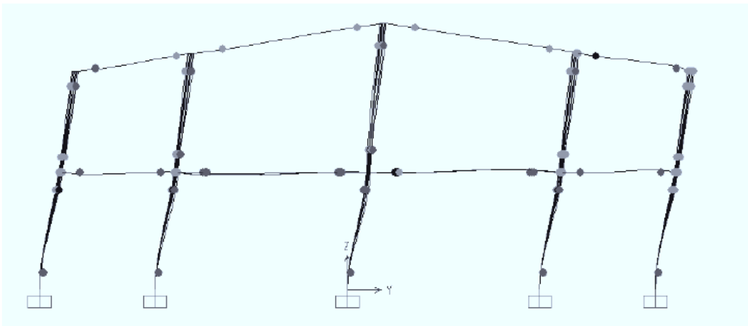


*Figure 63. Global mechanism in transversal direction (*RR *structural model) - School* Republica de Colombia

	$\square_y$	$\square_{pl}$	***T* [sec]***	***Cs [g]***	***Cd_{DL} [m]***	***Cd_{DS} [m]***	***Cd_{NC} [m]***
1	0.16	0.16	0.5637	0.3179	0.0232	0.0292	0.0390
2	0.16	0.34	0.5647	0.3195	0.0232	0.0326	0.0434
3	0.16	0.50	0.5653	0.3202	0.0232	0.0351	0.0468
4	0.16	0.66	0.5657	0.3206	0.0232	0.0325	0.0433
5	0.16	0.84	0.5657	0.3206	0.0232	0.0316	0.0421
6	0.34	0.16	0.6251	0.3126	0.0309	0.0368	0.0491
7	0.34	0.34	0.6261	0.3143	0.0309	0.0383	0.0510
8	0.34	0.50	0.6269	0.3152	0.0309	0.0377	0.0503
9	0.34	0.66	0.6276	0.3161	0.0309	0.0385	0.0514
10	0.34	0.84	0.6282	0.3164	0.0309	0.0399	0.0531
11	0.50	0.16	0.6739	0.3085	0.0360	0.0380	0.0600
12	0.50	0.34	0.6749	0.3101	0.0360	0.0426	0.0644
13	0.50	0.50	0.6757	0.3111	0.0360	0.0436	0.0669
14	0.50	0.66	0.6765	0.3120	0.0360	0.0447	0.0738
15	0.50	0.84	0.6775	0.3127	0.0360	0.0456	0.0829
16	0.66	0.16	0.7273	0.3040	0.0423	0.0424	0.0772
17	0.66	0.34	0.7284	0.3055	0.0423	0.0434	0.0790
18	0.66	0.50	0.7292	0.3065	0.0423	0.0439	0.0800
19	0.66	0.66	0.7300	0.3074	0.0423	0.0456	0.0831
20	0.66	0.84	0.7311	0.3083	0.0423	0.0486	0.0886
21	0.84	0.16	0.8128	0.2983	0.0529	0.0530	0.0937
22	0.84	0.34	0.8131	0.2984	0.0529	0.0540	0.0965
23	0.84	0.50	0.8138	0.2992	0.0529	0.0543	0.0962
24	0.84	0.66	0.8146	0.3000	0.0529	0.0559	0.0982
25	0.84	0.84	0.8157	0.3009	0.0529	0.0562	0.1006

Table 37. Non-linear parameters in the longitudinal building axis for (DR) - School Republica de Colombia

	ϖ_y	ϖ_{pl}	***T* [sec]***	***Cs [g]***	***Cd_{DL} [m]***	***Cd_{DS} [m]***	***Cd_{NC} [m]***
1	0.16	0.16	0.7304	0.2047	0.0209	0.0253	0.0337
2	0.16	0.34	0.7335	0.2102	0.0209	0.0278	0.0370
3	0.16	0.50	0.7357	0.2138	0.0209	0.0301	0.0402
4	0.16	0.66	0.7381	0.2173	0.0209	0.0329	0.0439
5	0.16	0.84	0.7416	0.2216	0.0209	0.0387	0.0516
6	0.34	0.16	0.7819	0.2046	0.0259	0.0301	0.0401
7	0.34	0.34	0.7846	0.2087	0.0259	0.0320	0.0427
8	0.34	0.50	0.7866	0.2116	0.0259	0.0341	0.0455
9	0.34	0.66	0.7890	0.2147	0.0259	0.0367	0.0489
10	0.34	0.84	0.7930	0.2194	0.0259	0.0418	0.0557
11	0.50	0.16	0.8261	0.2068	0.0290	0.0359	0.0479
12	0.50	0.34	0.8279	0.2092	0.0290	0.0376	0.0502
13	0.50	0.50	0.8295	0.2112	0.0290	0.0393	0.0524
14	0.50	0.66	0.8316	0.2136	0.0290	0.0416	0.0554
15	0.50	0.84	0.8356	0.2180	0.0290	0.0465	0.0620
16	0.66	0.16	0.8777	0.2097	0.0304	0.0433	0.0577
17	0.66	0.34	0.8785	0.2107	0.0304	0.0448	0.0597
18	0.66	0.50	0.8797	0.2120	0.0304	0.0464	0.0619
19	0.66	0.66	0.8815	0.2138	0.0304	0.0487	0.0649
20	0.66	0.84	0.8850	0.2172	0.0304	0.0522	0.0697
21	0.84	0.16	0.9738	0.2213	0.0414	0.0575	0.0767
22	0.84	0.34	0.9676	0.2167	0.0414	0.0583	0.0777
23	0.84	0.50	0.9665	0.2159	0.0414	0.0592	0.0789
24	0.84	0.66	0.9666	0.2160	0.0414	0.0607	0.0809
25	0.84	0.84	0.9683	0.2175	0.0414	0.0639	0.0852

Table 38. Non-linear parameters in the transversal building axis for (DR) - School Republica de Colombia

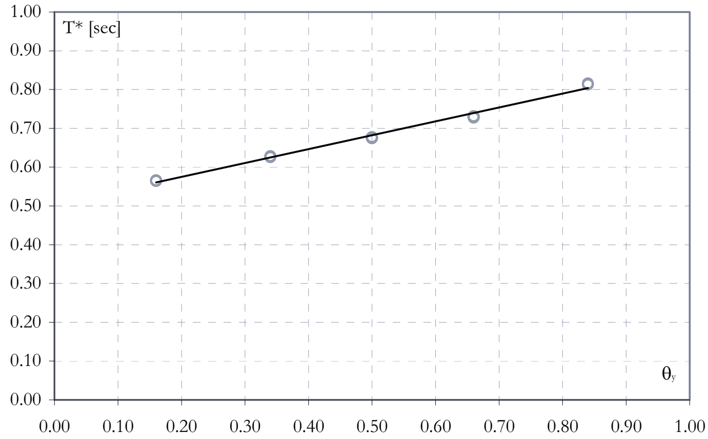


Figure 64. Effective period versus yielding rotation for DR models in longitudinal direction - School Republica de Colombia

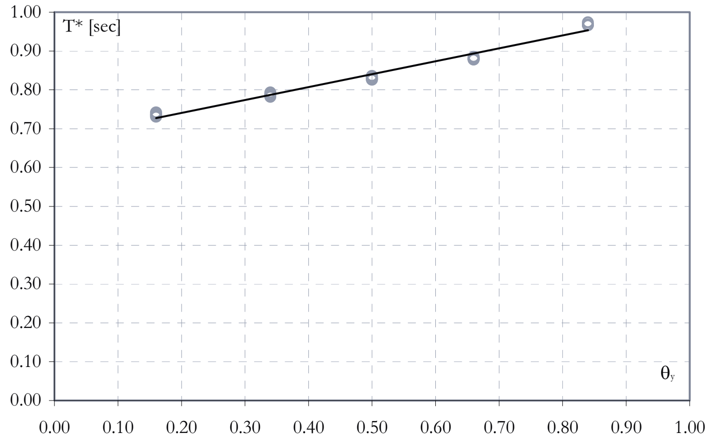


Figure 65. Effective period versus yielding rotation for DR models in transversal direction - School Republica de Colombia

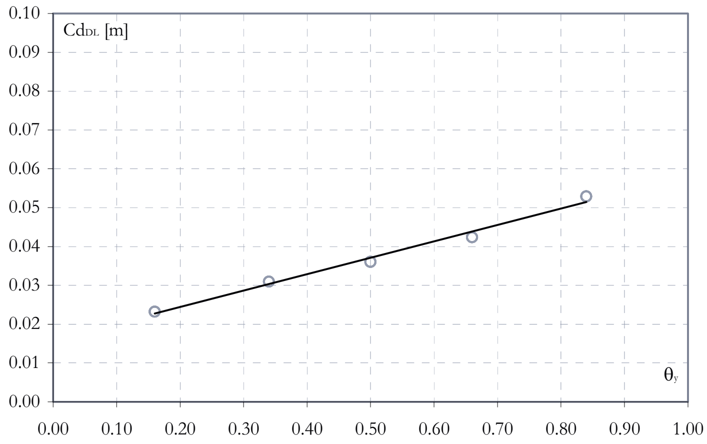


Figure 66. Cd$_{DL}$ versus yielding rotation for DR models in longitudinal direction - School Republica de Colombia

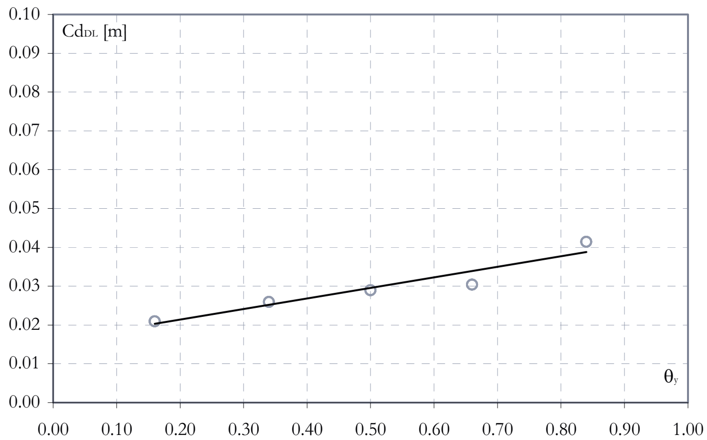


Figure 67. Cd$_{DL}$ versus yielding rotation for DR models in transversal direction - School Republica de Colombia

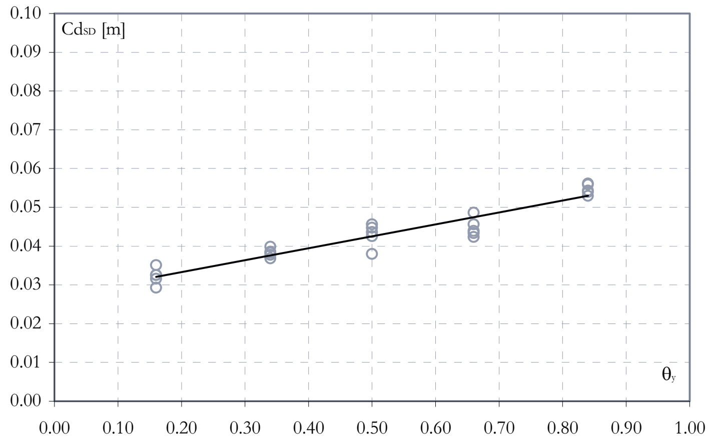


Figure 68. Cd_{SD} versus yielding rotation for DR models in longitudinal direction - School Republica de Colombia

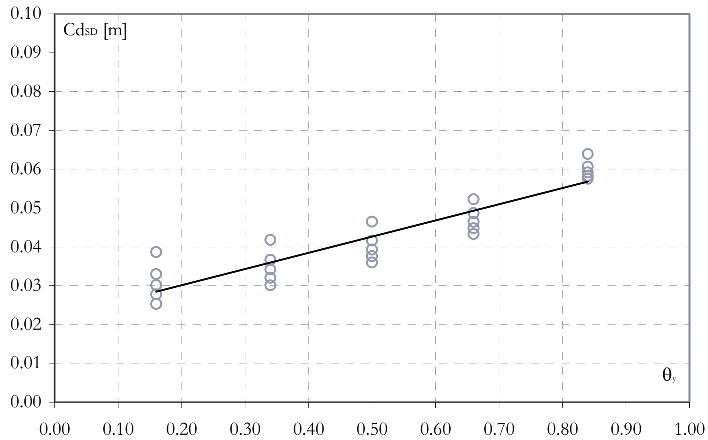


Figure 69. Cd_{SD} versus yielding rotation for DR models in transversal direction - School Republica de Colombia

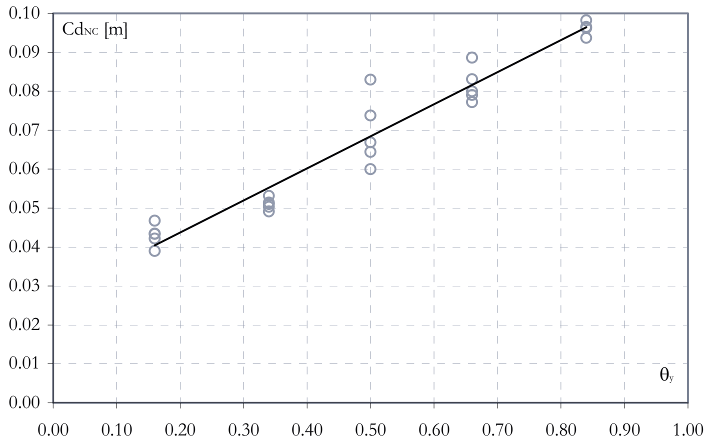


Figure 70. Cd_{NC} versus yielding rotation for DR models in longitudinal direction - School Republica de Colombia

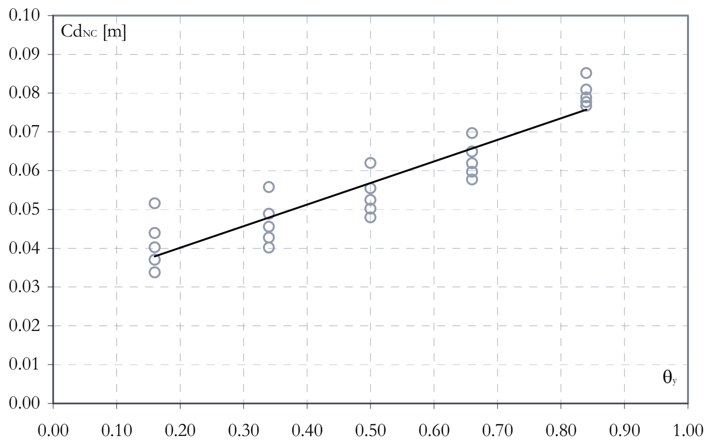


Figure 71. Cd_{NC} versus yielding rotation for DR models in transversal direction - School Republica de Colombia

To evaluate fragility curve parameters for the building the procedure as applied in 3.3.5 was adopted. In particular, the simulation consists of extracting a vector of the input parameters from the distributions of yielding and plastic rotations, that are the only random variables considered in this study.

To define seismic action, the current U.S. building code IBC-2006 (ICC, 2006) is used. Local soil conditions at the building site are characterized by near-surface shear-wave velocities $v_{s,30}$ between 360 m/s and 760 m/s, i.e. NEHRP site class C according to IBC-2006 (ICC, 2006; Figure 72) that corresponds to soil type B for Eurocode 8 (CEN, 2003).

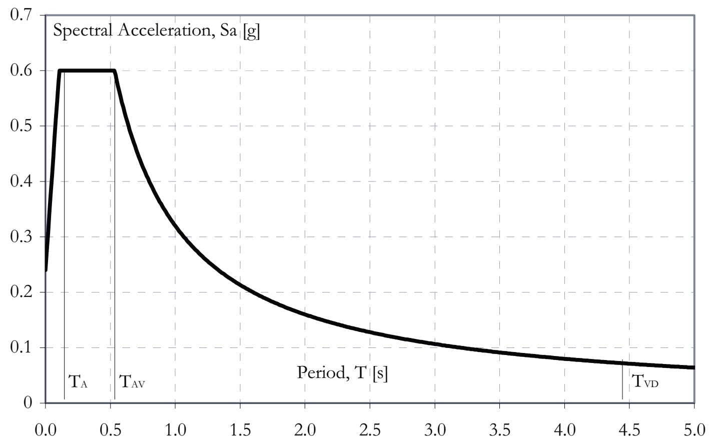


Figure 72. Elastic design response spectrum for soil class C according to U.S. seismic building code IBC-2006 (ICC, 2006)

For each simulation, the spectrum is entered with the T value corresponding to the random extraction and the elastic displacement demand S_d is derived. The inelastic demand is evaluated multiplying the elastic displacement demand by a modification factor C_R that depends on effective period T and on the spectral reduction factor R (Ruiz-Garcia and Miranda, 2003). Consequently, non-linear displacement capacity and demand may be compared in each run checking for failure at each of the three damage states considered. Scaling the elastic spectrum in order to investigate the demand range of interest allows to derive the fragility curves.

Figures from Figure 73 to Figure 78 show the generated fragility curves for the three considered limit states: damage limitation DL (continuous line), significant damage SD (dash line) and near collapse NC (dash-dot line) for both cases of rigid (RR) and deformable roof (DR) and in both building directions.

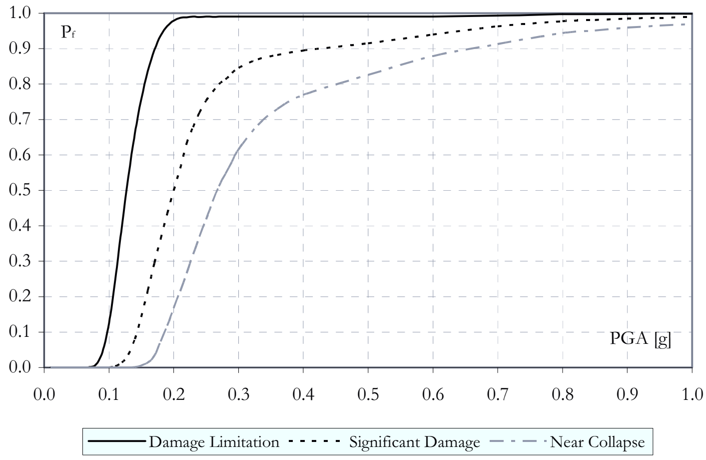


Figure 73. Fragility curves in the longitudinal building axis in case of RR - *School* Republica de Colombia

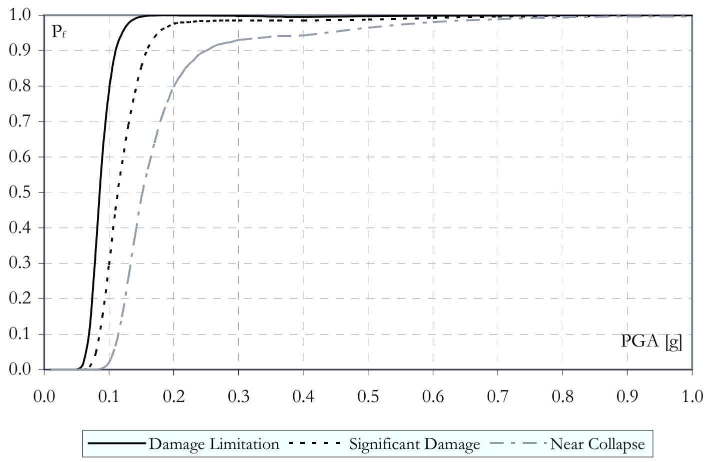


Figure 74. Fragility curves in the transversal building axis in case of RR - *School* Republica de Colombia

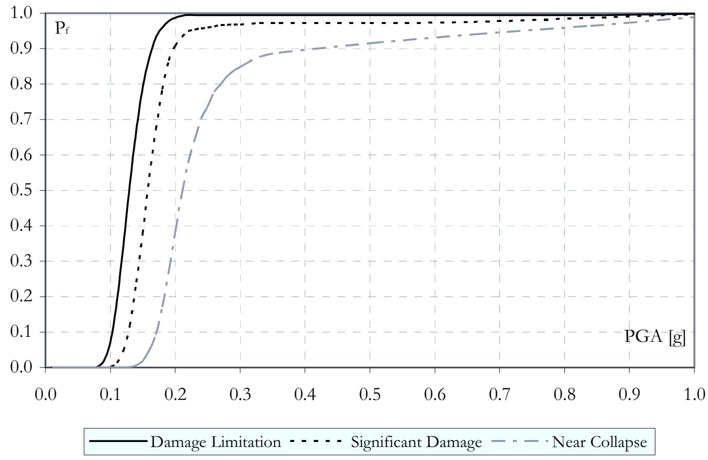


Figure 75. Fragility curves in the longitudinal building axis in case of DR - School Republica de Colombia

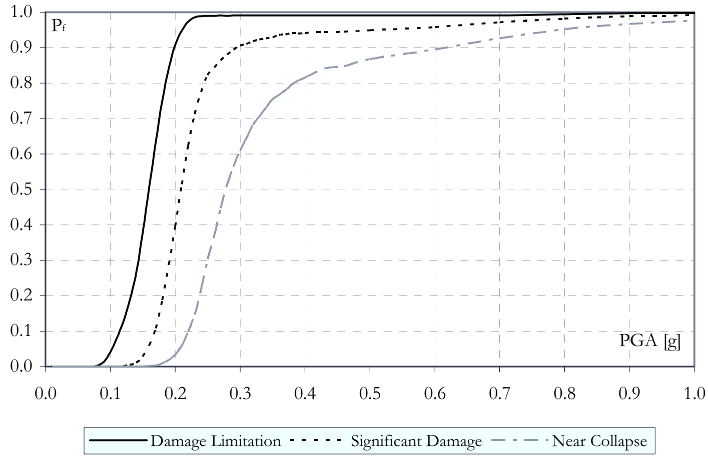


Figure 76. Fragility curves in the transversal building axis in case of DR - School Republica de Colombia

Considering in fragility function the variability due to the different model in roof, RR and DR, other two functions' groups are obtained, as shown in Figure 77 and Figure 78. In this case, the probability of failure is between the values assumed in case of RR model and DR model, analysing lonely, in both building axis.

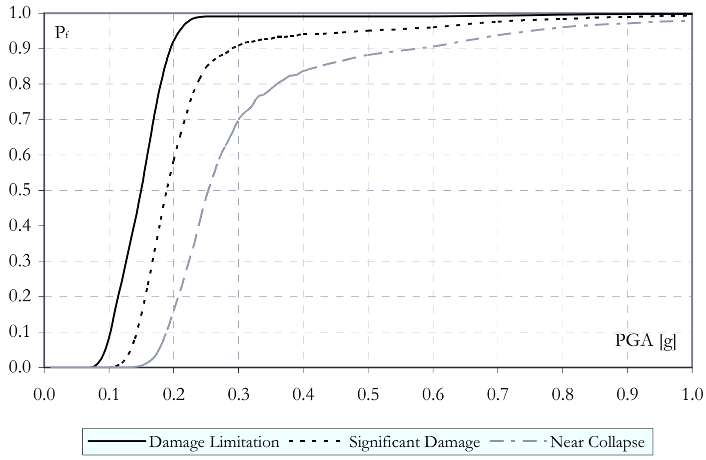


Figure 77. Fragility curves in the longitudinal building axis - School Republica de Colombia

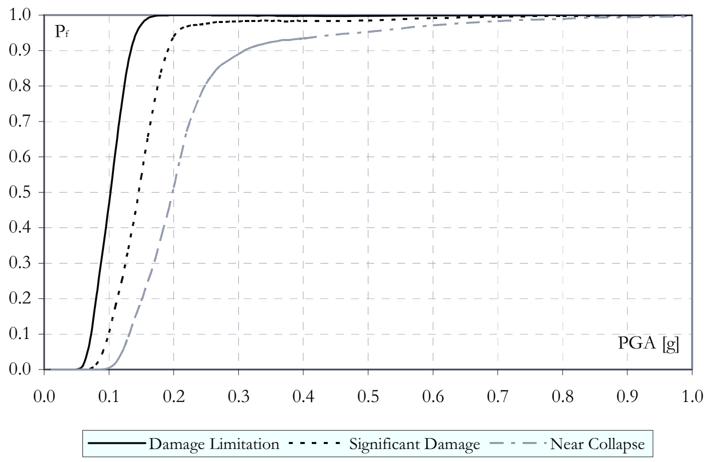


Figure 78. Fragility curves in the transversal building axis - School Republica de Colombia

Comparing fragility function with HAZUS approach, the standard deviation β of natural logarithm of spectral displacement for the considered damage state, is obtained. In Table 39, β as got and median value of PGA are reported.

	DL		***SD***		***NC***	
	β	PGA_m	β	PGA_m	β	PGA_m
RR – longitudinal direction	0.20	0.13	0.27	0.20	0.32	0.27
RR – transversal direction	0.20	0.09	0.17	0.12	0.20	0.15
DR – longitudinal direction	0.20	0.13	0.17	0.16	0.20	0.21
DR – transversal direction	0.18	0.16	0.17	0.21	0.20	0.28
RR and DR – longitudinal direction	0.30	0.15	0.30	0.19	0.32	0.25
RR and DR – transversal direction	0.26	0.10	0.25	0.15	0.30	0.20

Table 39. β for all fragility function – School Republica de Colombia

As written in HAZUS, the variability of capacity curves and the damage-state thresholds are influenced by:

- Uncertainty in capacity curve properties and the thresholds of damage states;
- Building population (i.e., individual building or group of buildings).

Relatively low variability of damage states would be expected for an individual building with well known properties (e.g., complete set of as-built drawings, material test data, etc.) and whose performance and failure modes are known with confidence. The taller the building the greater the variability in damage state due to uncertainty in the prediction of response and damage using pushover analysis. Relatively high variability of damage states would be expected for a group of buildings whose properties are not well known and for which the user has low confidence in the results (of pushover analysis) that represent performance and failure modes of all buildings of the group. The original development of damage-state fragility curves for generic model building were based on capacity variability, $\beta_C = 0.3$, demand variability $\beta_D = 0.45$ and damage-state threshold variability, $\beta_{T,ds} = 0.4$ (Structure).[44]

Considered only variability in thresholds and demand, by HAZUS equation β is equal to 0.18.

So comparing results reported in Table 39 with HAZUS' indication for β, it is observable that they are little different than HAZUS' ones, even if it is respected the increase of the β given by both variability (roof model and thresholds).

A comparison with the equivalent PGA adopted in case of Pre-Code designed structures, for concrete moment frame (C1L: 2 stories – Low-Rise) is done. In HAZUS, it is 0.10 g, 0.12 g and 0.21 g for slight, moderate and extensive limit state, respectively. So, considering that the definition of limit states adopted in this work isn't the same in HAZUS, results are satisfactory.

Using a HAZUS formula, with β reported in Table 39, fragility curves are derived. It is clear the very good approximation between the adopted approach and the HAZUS' one. From Figure 79 to Figure 84 fragility curves are derived with HAZUS approach, considering PGA mean and β value reported in Table 39, for all considered cases.

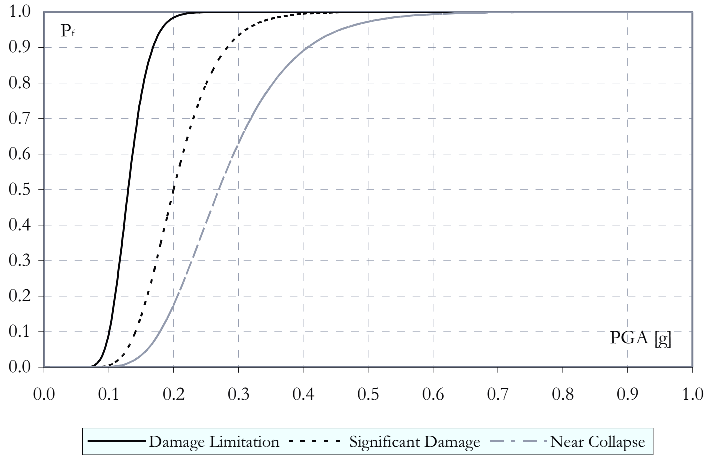


Figure 79. Fragility curves derived with HAZUS' formulation in case of RR *longitudinal direction - School* Republica de Colombia

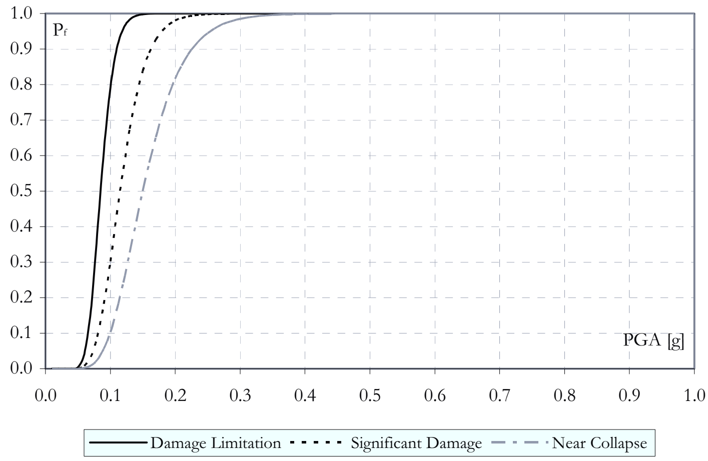


Figure 80. Fragility curves derived with HAZUS' formulation in case of RR transversal direction - School Republica de Colombia

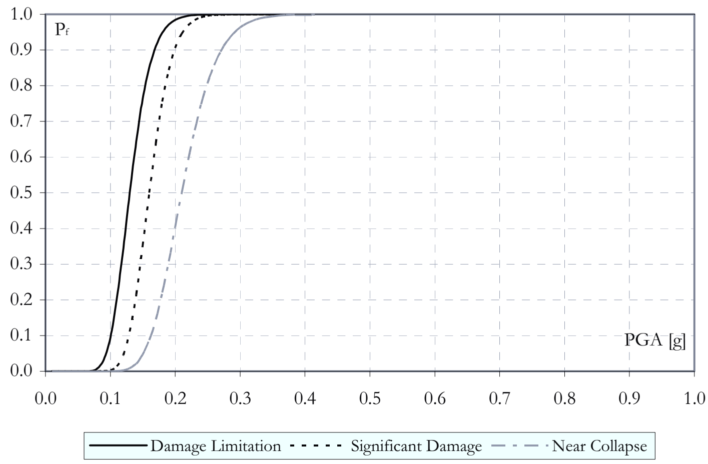


Figure 81. Fragility curves derived with HAZUS' formulation in case of DR longitudinal direction - School Republica de Colombia

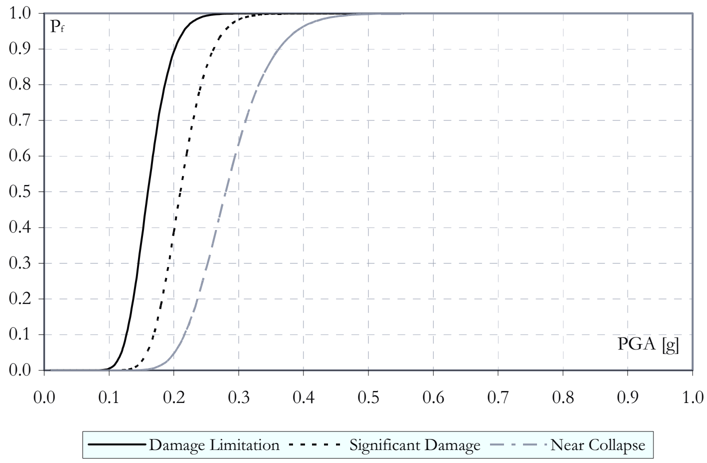


Figure 82. Fragility curves derived with HAZUS' formulation in case of DR transversal direction - School Republica de Colombia

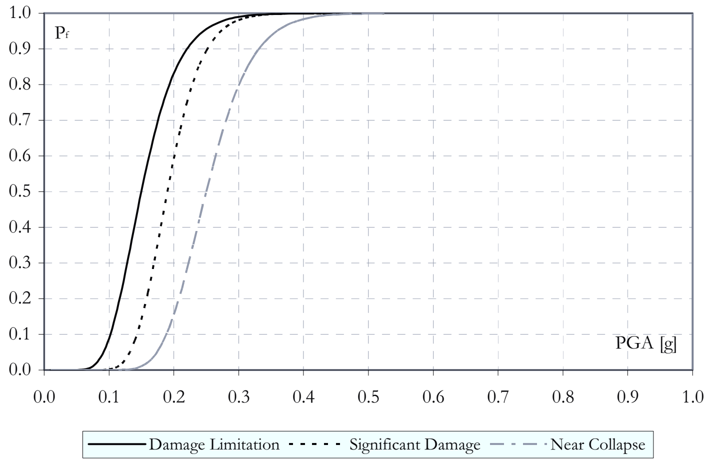


Figure 83. Fragility curves derived with HAZUS' formulation in case of all models in longitudinal direction - School Republica de Colombia

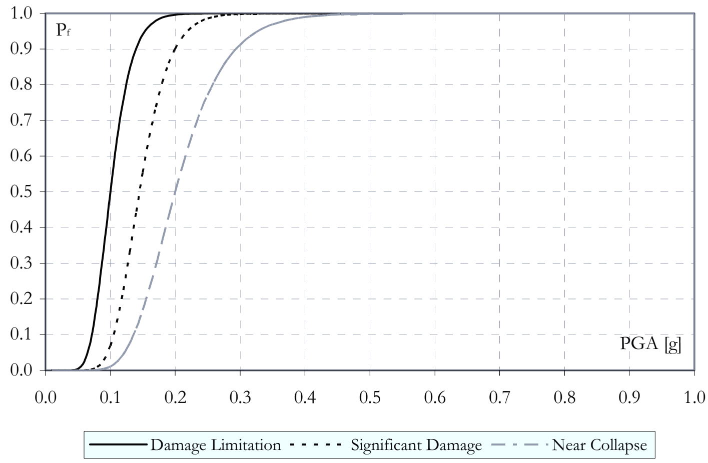


Figure 84. Fragility curves derived with HAZUS' formulation in case of all models in longitudinal direction - School Republica de Colombia

5.2. Hospital Dr. Luis Edmundo Vasquez, Chalatenango – San Salvador

5.2.1. Site-dependent seismic demand

According to available geological information, local soil conditions at the building site consist of very dense soils and soft rocks, with shear-wave velocity $v_{s,30}$ between 360 m/s and 760 m/s (i.e. NEHRP soil class C).

5.2.2. Reference code

The building's performance is evaluated according to:

- Eurocode 2 "*Design of concrete structures – Part 1-1: General rules and rules for buildings*", EN 1992-1-1, December 2004;
- Eurocode 8 "Design of Structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings", EN 1998-1, December 2004;
- Eurocode 8 "Design of Structures for earthquake resistance – Part 3: Strengthening and repair of buildings", EN 1998-3, June 2005;
- Eurocode 8 "Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects", EN 1998-5, November 2004.

Accidental loads are evaluated according to:

- El Salvador Hospital Code 2004.

5.2.3. Safety evaluation

As above done for school, safety evaluation of existing buildings have to consider a limit state more than in new design because they don't satisfy both resistance hierarchy and elements' ductility. Security requires refer to structural damage state defined by:

- Limit State of Damage Limitation (DL);
- Limit State of Significant Damage (SD);
- Limit State of Near Collapse (NC).

The procedure adopted to evaluate the building follows the next steps:

- Data analysis;
- Definition of knowledge level;
- Definition of seismic action based on various limit states;
- Modelling and analysis;

- Evaluation of results.

5.2.4. Building description

The hospital Dr. Luis Edmundo Vasquez complex is located in Chalatenango – San Salvador. It was constructed in 1971 and survived the big earthquakes in 2001: M 7.7 earthquake in January 13 and M 6.6 one in February 13, which caused more than 1,000 deaths and a lot of structural damage in El Salvador (see Table 5).

The entire complex is composed by eight buildings separated each others. All buildings are no more than tree levels high a part the tower, high six levels.

Figure 85. Tower view from outside – Hospital Dr. Luis Edmundo Vasquez

The six-story building has a rectangular plan with a length-to-width ratio of 3.28 (length 42.0 *m*, width 12.80 m; Figure 86). An emergency stair case is in the extremity of the longer side, it is in reinforced concrete with broken axis beams of cross-section equal to 0.30 *m* x 0.50 *m*, connected with landing beams with cross-section 0.30 *m* x 0.60 *m* (see structural model, Figure 94). This tower is separated from the other parts by a seismic joint large 6 cm. All levels are accessible by elevators and stair cases placed in a core separated by seismic joint from the tower (Figure 87).

The inter-story height is equal to 3.8 *m*.

The main load-resisting structure consists of reinforced-concrete frames; in both lateral directions, the beams cross-section are reported in Table 40. All columns at the ground floor have cross-section of 0.40 *m* × 0.65 *m*; instead the others have cross section of 0.40 *m* × 0.55 *m*. The structural system neither includes cores nor shear walls.

Level	***Longitudinal beams - perimeter***	***Longitudinal beams - centre***	***Transversal beams***
I	*0.25 m × 0.55 m*	*0.25 m × 0.60 m*	*0.30 m × 0.70 m*
II	*0.25 m × 0.55 m*	*0.25 m × 0.60 m*	*0.30 m × 0.70 m*
III	*0.25 m × 0.55 m*	*0.25 m × 0.60 m*	*0.30 m × 0.65 m*
IV	*0.25 m × 0.55 m*	*0.25 m × 0.55 m*	*0.30 m × 0.65 m*
V	*0.25 m × 0.55 m*	*0.25 m × 0.55 m*	*0.30 m × 0.65 m*
VI	*0.25 m × 0.55 m*	*0.25 m × 0.55 m*	*0.30 m × 0.65 m*

Table 40. Beam cross-sections – Hospital Dr. Luis Edmundo Vasquez

The slab, thick 0.12 m, is in reinforced concrete.

The actual state of the building is characterized by a poor preservation state. Carbonation test was negative, in fact a purple color results (see Figure 93).

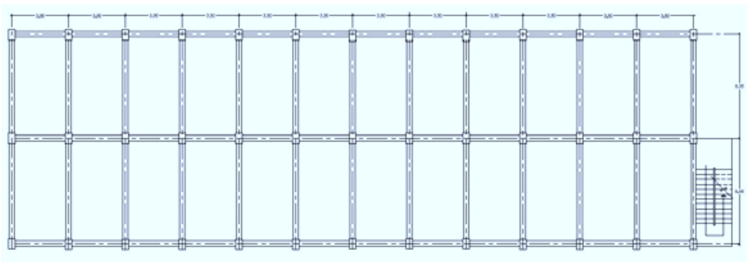


Figure 86. Plan of the building – Hospital Dr. Luis Edmundo Vasquez

Figure 87. Architectonical scheme of the entire complex Hospital Dr. Luis Edmundo Vasquez

Figure 88. Columns' position - Hospital Dr. Luis Edmundo Vasquez

Figure 89. Seismic joint – Hospital Dr. Luis Edmundo Vasquez

5.2.5. Evaluation data

As above mentioned, necessary fonts to evaluate data are:

- Design tables;
- Geometrical and structural relief;

- In-situ testing;
- Practice technical and code prescriptions.

Quantity and quality of obtained data define levels of knowledge; this is characterized by following aspects:

1. geometry of structural elements;
2. structural peculiarities and bars' placement and mechanical steel percentage;
3. mechanical property of materials.

Geometry. For 'Dr. Luis Edmundo Vasquez' hospital the architectural design are available, given by Ministerio de Salud in San Salvador, so the geometric characterization is done by the comparison between design and geometric relief in situ, to know all resisting frames to gravity and seismic loads, structural elements and their dimensions.

Figure 90. Emergency stair case

Structural peculiarities. Structural designs are not available, so mechanical steel percentage and bars' placement are located by pacometric tests and caliber measures, allowed only in some structural elements. In fact, only *k* elements of total *n* are investigated; for the other (*n-k*) elements, a simulated design is done according to [27].

Figure 91. Pacometric test on columns – Hospital Luis Edmundo Vasquez

Mechanical property of materials. Materials used to model the structural behaviour are concrete with a mean value of cylindrical compression strength f_c equal to *19 MPa* and a steel characterized by mean value of yield strength f_y equal to *345 MPa*. As above mentioned, in situ tests were done, even if it was not possible to use results because destructives tests to check materials and calibrates obtained values were not done. So, adopted values derive from [28], in fact in table *"Default Lower-Bound Tensile and Yield Properties of Reinforcing Bars for Various Periods"* steel for an intermediate grade has a value of minimum yield equal to 50,000 psi (about 345 MPa); for concrete the table "*Default Lower-Bound Compressive Strength of Structural Concrete"* by [28] was used calibrates with planning practice in El Salvador, in fact the lower value of compressive strength suggested by FEMA for frame build from 1970 to present, is 3,000 psi (about 21 MPa) but in the model the mean value adopted was reduced.

This evaluation proceeds the indications reported in the Code Hospitales 2004 in El Salvador, in fact, all materials used in constructions, have to be of a good quality. The concrete have to be consistent with ASTM C150, and the steel consistent with ASTM C-31 and ASTM C-39, with a minimum strength of 420 MPa for diameter greater than ½" and of 280 MPa for diameter equal to 3/8".

Figure 92 shows the stone inside a column at the ground floor in the hospital, to prove the difficult in material characterization in situ, given by the lower inerts' mixture.

Steel geometric percentage is unknown, so it's necessary to do a simulated design process. For beams, the strains is computed from linear combination considering G+Q and assumed steel percentage in function of the ratio between bending moment and the product between the base and the square of the useful height, and of the ratio between cover and useful height (ACI 63). For columns it is adopted the recommendation from ACI 63 (geometric percentage = 0.01 – 0.08 gross section), checked in common design in El Salvador. E.g. in hospital Laboratorio Max Bloch Central, steel area is equal to 0.015 gross section.

Information's availability based on geometrical study, structural peculiarities and mechanical property of materials, allows to suppose a satisfy knowledge.

Figure 92. Stone inside a column at ground floor of hospital

Figure 93. Carbonation negative tested with color indicator (purple color)

5.2.6. Seismic action

To define seismic action, the IBC-2006 is used. The soil is a very dense soil and soft rock, with shear-wave velocity $v_{s,30}$ between 360 m/s and 760 m/s, site class C in the IBC.

Known the soil characters, elastic spectra in terms of spectral acceleration in function of period T and of spectral displacement is obtained.

5.2.7. Nonlinear static analysis

Nonlinear static (pushover) analysis is a non-linear static analysis under constant gravity loads and monotonically increasing horizontal loads. It is based on the assumption that the response of the structure can be related to the response of an equivalent Single Degree Of Freedom system (SDOF), that is used to determine seismic demand.

The analysis continues until a predefined limit state is reached or until structural collapse is detected.

Seismic loads generally act in combination with the (static) gravity loads. According to the applied code provision [43] the different load cases have to be defined. After the Eurocode EN 1998, earthquake loads E are multiplied by an importance factor γ_I.

Following, it is reported the equation used to combine seismic and static loads:

$$\gamma_I \cdot E + G + P + \sum \Psi_2 \cdot Q \tag{43}$$

For Pushover analysis, seismic loads are applied after a deformed configuration due to the static load case ($G + \Psi_2 \cdot Q$).

5.2.7.1. Calculation of dead loads

Dead loads (*G*) affecting the beams (in [*kN/m*]) are computed as the product of the specific load of the slab (in [*kN/m²*]) and its impact depth (in [*m*]) on the beam. The self weight of the elements is also considered. In this case the model automatically considered the self weight of elements (slabs, beams) and computed the loads. According to El Salvador Seismic Code 1989 [45], the volumetric weight of reinforced concrete is between 2.20 ton/m^3 and 2.40 ton/m^3, so it was assumed the maximum value for weight.

5.2.7.2. Live loads

Live loads (Q) affecting the beams (in [kN/m]) are computed as the product of the live loads suggested by the code (in [kN/m^2]) and their impact depths (in [m]) on the beam. According to El Salvador Hospitals Code 2004 [46], live loads in hospitals depend by destination of the rooms; so a weighted average, equal to 4.50 kN/m^2 is assumed for the tower and a live load of 5.00 kN/m^2 for stairs.

5.2.7.3. Seismic masses

Seismic masses for each level are:

Level	***Mass [kNs²/m]***
1	617
2	609
3	603
4	602
5	602
6	426

Table 41. Seismic masses at each level – Hospital Dr. Luis Edmundo Vasquez

5.2.8. Structural modelling

Spatial model is realized with the help of calculation software, and it is defined by elements' axis line. Structural model is composed by frame elements connected each others with horizontal diaphragms, given value the hypothesis of rigid diaphragm, and shells between horizontal frames to model the real behavior of slabs; so, each level is characterized by three liberty degree, the two translations along both directions (longitudinal and transversal building axis) and the rotation around vertical axes for the mass centre.

In the modelling, it is not considered both the contribute of not structural elements. Nevertheless, structural model represents the distribution of masses and effective rigidity. Following, it is reported the model:

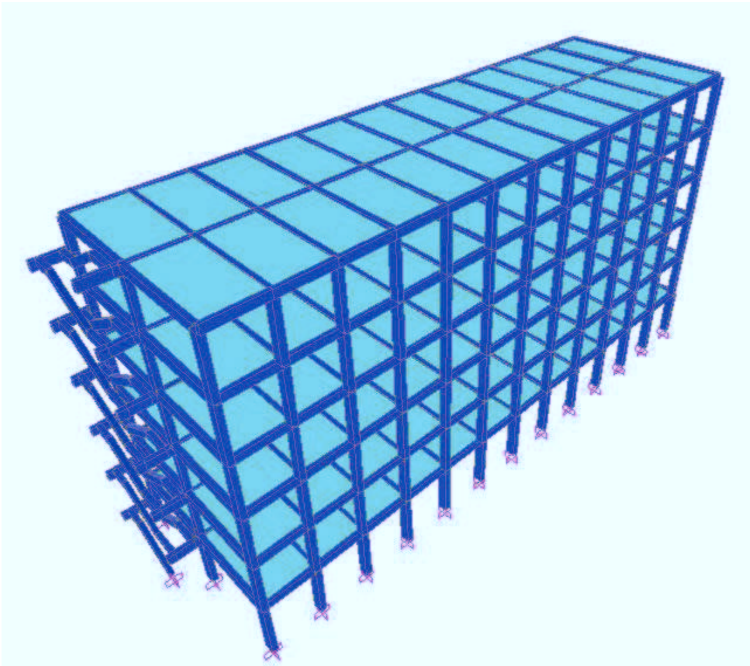


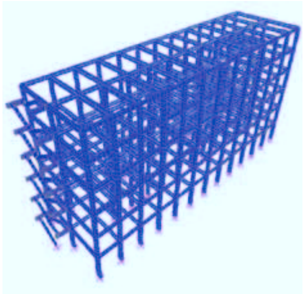
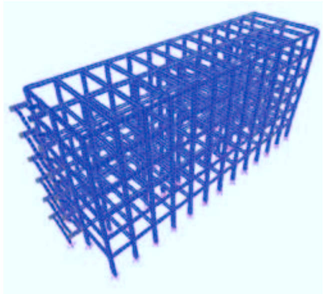
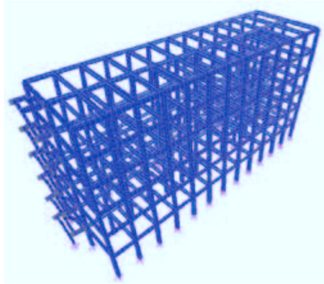
Figure 94. Spatial structural model – Hospital Dr. Luis Edmundo Vasquez

Flexural behavior is modeled according to 3.3.1.2, shear strength is verified for each element and no shear crises are reported.

In this case, both variability in materials' strength and variability in mechanical model are considered. A high number of structures were generated.

5.2.9. Building's dynamic property

Building dynamic elastic characters' evaluation is conducted with modal analysis on structural model as above mentioned. Analysis is done considering totality of vibration mode of spatial model. Following, it is reported the first three vibration modes with relative evaluated periods:

I mode	II mode	III mode
$T = 0.73\,s$	$T = 0.64\,s$	$T = 0.58\,s$

Table 42. Period T of first three vibration modes with tri-dimensional view – Hospital Dr. Luis Edmundo Vasquez

Table 43 shows periods of 12 vibration modes and associated participating masses, in longitudinal and transversal building axis, and the rotation component. From a modal analysis, the first mode is in longitudinal direction, the second one is in transversal direction and the third is a rotational one.

Mode	***Period [sec]***	***Participation masses [%]***		
		X	***Y***	***XY***
1	0.73	83.12	0.02	0.00
2	0.64	0.00	66.48	11.13
3	0.58	0.00	12.86	68.70
4	0.24	10.08	0.00	0.00
5	0.21	0.00	9.45	1.57
6	0.19	0.00	1.86	9.50
7	0.15	3.70	0.00	0.00
8	0.12	0.00	3.56	0.00
9	0.11	0.00	0.71	3.45
10	0.10	1.85	0.00	0.00
11	0.08	0.91	0.00	0.00
12	0.08	0.00	2.07	0.37

Table 43. First 12 vibration modes – Hospital Dr. Luis Edmundo Vasquez

5.2.10. Nonlinear static analysis

As above mentioned (3.3) and following the same procedure adopted in (5.1.10), the analysis consists in the application of dead loads to building, as in seismic combination, and an adequately system of horizontal static forces that, increasing, give a monotonic grow in horizontal displacement of control point, generally in the middle of the roof.

Methodology consists in:

- Knowledge of a force-displacement relationship generalized between the resulting force, base shear V_b, and the displacement d_c of a control point, usually chosen in the centre of the roof level;
- Characters determination of a single degree of freedom system, called SDOF, with equivalent bilinear behaviour;
- Determination of maximum response in displacement of SDOF system with the use of displacement spectrum;
- Conversion of equivalent displacement system as above mentioned in the effective building's configuration;
- Verification of displacement compatibility (elements/ductile mechanism) and resistances (elements/fragile mechanisms).

Particularly, it has to apply to building a distribution of horizontal forces applied in the centre of mass point at each level, proportional to masses for displacement correspondent to first vibration mode in both analysis directions, longitudinal and transversal, applied separately.

In Table 32 values combined for distribution, as above mentioned, are reported:

Level	***Mass*** $[kN \cdot s^2/m]$	***Mode 1X***	***Mode 1Y***
1	617	0.18	0.12
2	609	0.42	0.35
3	603	0.64	0.58
4	602	0.82	0.78
5	602	0.94	0.92
6	436	1.00	1.00

Table 44. Masses and first vibration mode in both building directions – Hospital Dr. Luis Edmundo Vasquez

5.2.10.1. Nonlinear modelling

Spatial model has been realized with a computational software and it is defined by axis line of elements, considering rigidity of joint with rigid pieces at the end of elements. Structural model is constituted by frame elements connected by horizontal diaphragms, given true the hypothesis of infinitely rigid level. At each level, slabs are modelled as shell between frames.

Linear elastic behaviour of elements is evaluated considering the geometric dimensions (transversal section and length) and mechanical characters of materials (elastic modulus of concrete). The post-elastic behaviour (nonlinear) is evaluated by a lumped plastic model; this model considers plasticity concentrated in plastic hinges at the end of elements. Here, after the elastic limitation, inelastic deformations are concentrated.

The advantages of this kind of modelling are simplicity and computational efficiency. One of model limitation is given by a fixed point of inflexion point during analysis (prefixed shear length L_V). Lumped plasticity model doesn't allow the computation of plastic hinge in the middle of element, caused by interaction of gravitational loads and horizontal ones.

In this case, to simplify the computational burden, a tri-linear moment-rotation relationship was adopted to characterize elements at the end section, (the relationship as above mentioned in 3.3.1.2 without maximum point and hypothesizing ultimate moment equal to yielding one) considering a shear length equal to half of the element length ($L_V = 0.5 \cdot L$).

5.2.11. Force – displacement relationship

Defined plastic hinge properties, for each element beam and column, non-linear analysis is done. Analysis proceeds with force control, increasing horizontal forces with a coefficient, so the structural behaviour is described until the top of capacity curve.

Results of non-linear analysis are reported following, referring to force distribution proportional to product of masses for deformed shape at first vibration mode. By way of illustration, it is reported only one analysis result in positive transversal direction, for mean values of material strengths and mean values of both rotations.

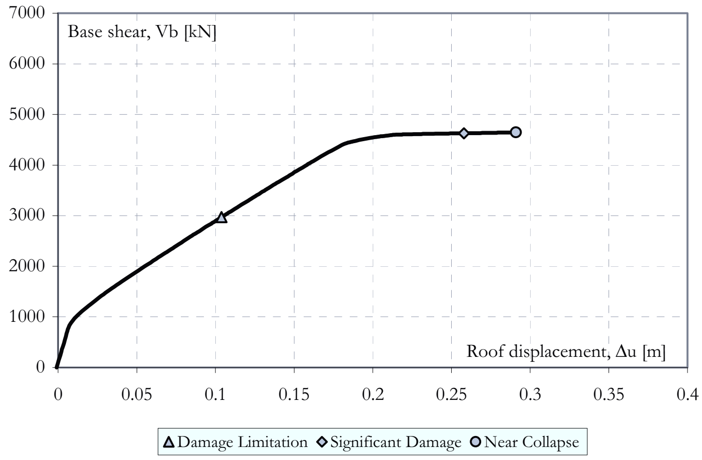


Figure 95. Capacity curve in longitudinal building axis – Hospital Dr. Luis Edmundo Vasquez

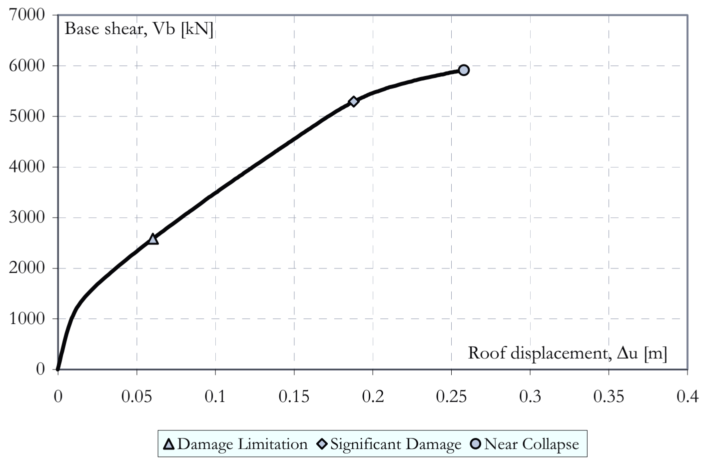


Figure 96. Capacity curve in transversal building axis – Hospital Dr. Luis Edmundo Vasquez

Figure 95 and Figure 96 show the relationship between structural base shear V_b and the control point displacement Δ_u, chosen coincident with the mass centre of roof.

5.2.12. Equivalent bilinear system

Equivalent bilinear system is evaluated to know structural displacement demand for each limit state. Therefore, it is essential to transform multiple degree of freedom system (from now on MDOF) in a single degree of freedom (from now on SDOF), as above mentioned (3.3.1.5). Participating factor Γ is evaluated:

$$\Gamma = \frac{\sum m_i \cdot \Phi_i}{\sum m_i \cdot \Phi_i^2} \tag{44}$$

where Φ is the displacement vector representative of structural deformed shape during the first vibration mode in the considered building axis, normalized to unit value; m_i is the mass at building level i and the sum at numerator represents generalized mass (equivalent mass of SDOF). Considering mean value of materials' strength, participating factor in both direction is:

Building axis	***m* [ton]***	***Γ***
X	2253	1.284
Y	2097	1.308

Table 45. SDOF generalized mass m and modal participation factor Γ for mean structure – Hospital Dr. Luis Edmundo Vasquez*

MDOF capacity curve has to be scaled by participating factor Γ to determine the SDOF force – displacement curve ($F^* - d^*$) with the following relationship:

$$F^* = V_b / \Gamma \tag{45}$$

$$d^* = \Delta_u / \Gamma \tag{46}$$

Known SDOF system $F^* - d^*$ curve, it is necessary to define an equivalent bilinear low force – displacement, the adopted procedure is this one of areas' equivalence. In this way, rigidity k^*, and consequently elastic period T^* of SDOF system can be obtained by the following equation:

$$T^* = 2\pi\sqrt{\frac{m^*}{k^*}} \tag{47}$$

where m^* is the generalized mass equal to $\sum m_i \cdot \Phi_i$.

5.2.13. Results

Considering variability in mechanical thresholds and in materials, Capacity Curves are obtained.

Both, concrete and steel strengths f_c and f_y are considered through a normal distribution with a mean value of 19.00 N/mm² and 345.00 N/mm² and CoV 25% and CoV 8%, respectively; as in 3.3.1.4.

By way of illustration, only analysis results in positive directions are reported.

Non-linear static analysis has been applied to 625 structural models generated combining lognormal distribution of yielding and plastic rotation values with the normal distribution of steel and concrete strength, as above mentioned; 25 structures are generated considering the combination of 16, 34, 50, 66 and 84 percentiles for both rotation, for each structure five kinds of steel and five kinds of concrete are considered, according to (3.3.1.3).

The variability in output for pushover analysis is investigated considering the various variability adopted in input.

Figure 97 and Figure 98 illustrate the increasing displacement capacity, in both directions, corresponding to the plastic rotation variability adopted in the mechanical characterization of the structural elements. As it could be expected, given yield rotation, structural displacement capacity increases with the increasing of plastic rotation.

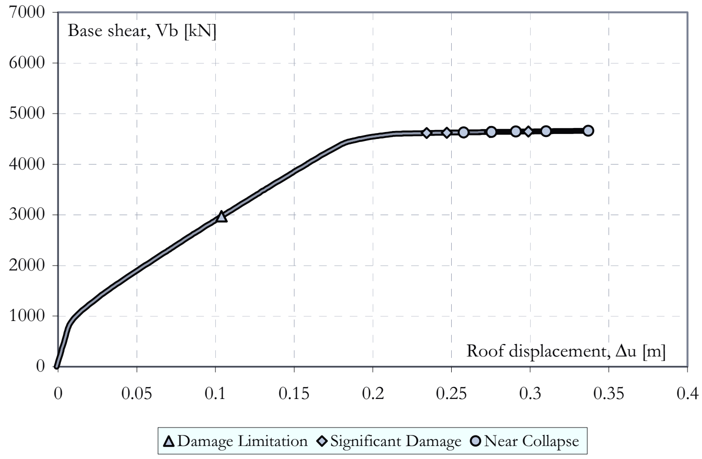


Figure 97. Capacity curves in longitudinal building axis for mean values of materials' strength and yield rotation with increasing plastic rotation – Hospital Dr. Luis Edmundo Vasquez

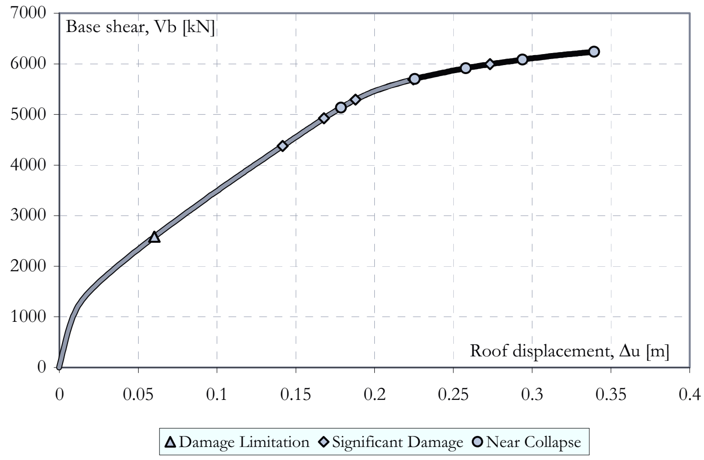


Figure 98. Capacity curves in transversal building axis for mean values of materials' strength and yield rotation with increasing plastic rotation – Hospital Dr. Luis Edmundo Vasquez

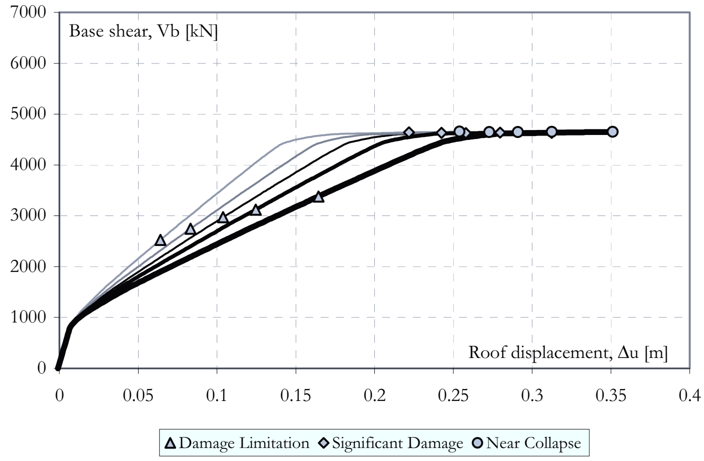


Figure 99. Capacity curves in longitudinal building axis for mean values of materials' strength and plastic rotation with increasing yield rotation – Hospital Dr. Luis Edmundo Vasquez

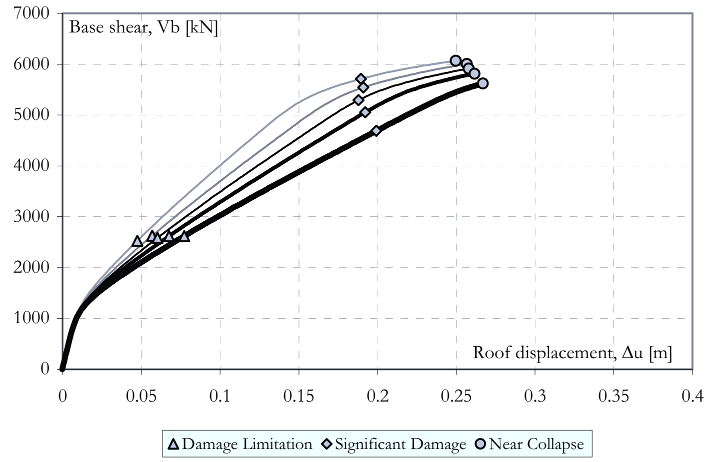


Figure 100. Capacity curves in transversal building axis for mean values of materials' strength and plastic rotation with increasing yield rotation – Hospital Dr. Luis Edmundo Vasquez

Figure 99 and Figure 100 illustrate the increase of the effective period through the system's stiffness corresponding to the yield rotation variability adopted in the mechanical characterization of the structural elements. For a given plastic rotation and considering adopted variability in yielding one, structural deformability increases as yield rotation also increases. Evidently, the global stiffness reduction leads to the increasing, of the system's effective period, as it will be clarified in the following.

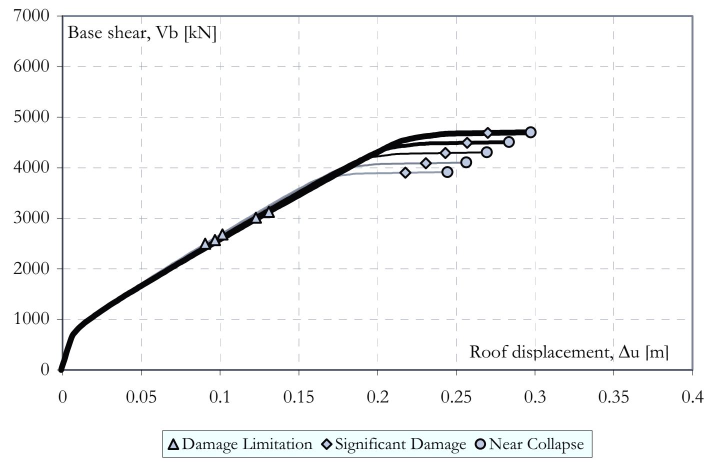


Figure 101. Capacity curves in longitudinal building axis for fixed values of both considered rotations and concrete strength with increasing steel strength – Hospital Dr. Luis Edmundo Vasquez

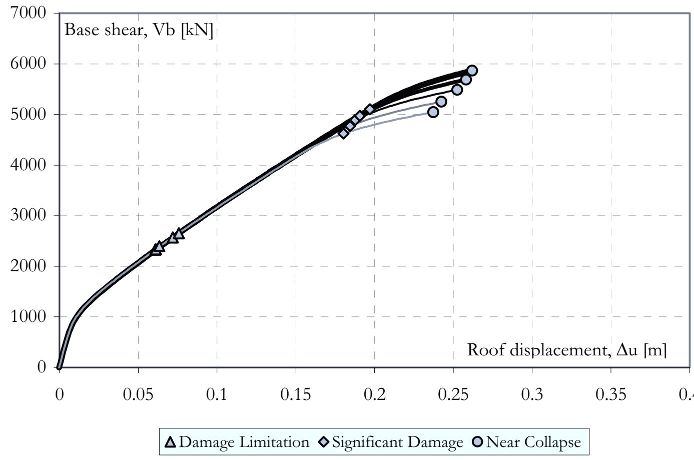


Figure 102. Capacity curves in transversal building axis for fixed values of both considered rotations and concrete strength with increasing steel strength – Hospital Dr. Luis Edmundo Vasquez

Figure 101 and Figure 102 report the increase in capacity strength and in capacity displacement, for all limit states evaluated, considering the increase of steel strength, for both building directions. Results of some analysis, considering the growing in concrete strength, fixed rotations and steel strength, are reported in the Figure 103 and Figure 104; as it can be easily seen growing concrete strength, the structural capacity, in terms of strength and displacement, increases. Moreover, a change in structural stiffness characterizes capacity curves considering the variability in concrete strength resistance.

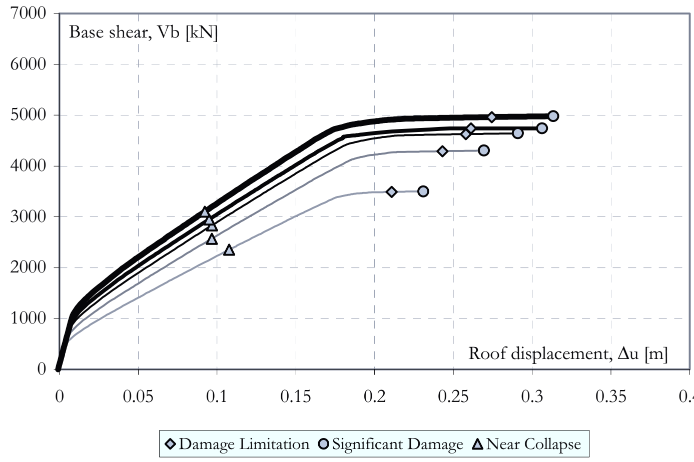


Figure 103. Capacity curves in longitudinal building axis for fixed values of both considered rotations and steel strength with increasing concrete strength – Hospital Dr. Luis Edmundo Vasquez

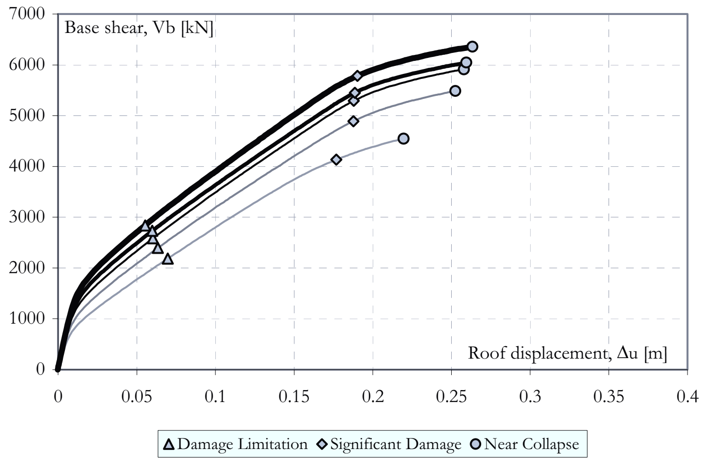


Figure 104. Capacity curves in transversal building axis for fixed values of both considered rotations and steel strength with increasing concrete strength – Hospital Dr. Luis Edmundo Vasquez

The 'pushover' capacity curve, which represents the nonlinear relationship between base shear and roof displacement for the Multi Degree Of Freedom (MDOF) system analyzed, is utilized as an instrument to characterize the force-displacement curve of a Single Degree Of Freedom (SDOF) system that is considered to be dynamically equivalent to the MDOF. Different formulations exist to institute the MDOF-SDOF equivalence. Here, we refer to the method suggested in Fajfar [34] which is adopted by Eurocode 8 [29]. The force-displacement curve for a SDOF, suitably idealized, allows the identification of the "capacity parameters" in terms of limit state displacement C_d, base shear coefficient C_s and effective period T which are necessary to evaluate the seismic demand with a spectral approach.

Starting from the results of the pushover analyses the range of variability of the capacity parameters C_d, C_s and T for the different damage levels investigated are obtained.

In particular, considering both adopted variability, the displacement capacity for DL, SD and NC limit states ranges, between, 0.0366-0.1835 m, 0.1053-0.3089 m and 0.1140-0.3504 m in the longitudinal direction. In transversal direction, the ranges are 0.0295-0.1208 m for DL, 0.0854-0.2213 m for SD and 0.1107-0.2946 for NC. With regard to effective periods, the ranges are 1.06-2.33 sec in longitudinal direction and 1.26-2.02 sec in transversal one. The capacity strength Cs ranges from 0.1092-0.1899 g in longitudinal direction and 0.1324-0.2637 g in the transversal one. Non-linear parameters for structural models in longitudinal building axis and transversal one, are reported from Table 63 to Table 75.

In case of only thresholds' variability considered, instead, the displacement capacity ranges are 0.0500-0.1281 m for DL, 0.1528-0.2723 m for SD and 0.1689-0.3067 m for NC, in longitudinal building axis. In transversal direction, the ranges are 0.0363-0.0590 m for DL, 0.1003-0.2119 m for SD and 0.1346-0.2805 for NC. The capacity strength Cs has generally little variation, in fact especially in longitudinal building axis the range is 0.1636-0.1652, instead in the transversal one the variability in output is higher: 0.1710-0.2314. The effective period range is again wide in both building axis: 1.5294-1.9713 in longitudinal direction and 1.3826-1.7842 in transversal one. Results for adopted thresholds' variability considering mean values for materials strength are reported in Table 47 and Table 48 for longitudinal and transversal directions, respectively.

In case of only materials' variability considered, the displacement capacity ranges are 0.0491-0.1156 m for DL, 0.1409-0.2302 m for SD and 0.1512-0.2569 m for NC, in longitudinal building axis. In transversal direction, the ranges are 0.0381-0.0815 m for DL, 0.1232-0.1507 m for SD and 0.1621-0.2061 for NC. The capacity strength Cs has higher variation than considering only thresholds' variability, in longitudinal building axis the range is 0.1095-0.1892, instead in the transversal one the range is 0.1610-0.2482. The effective period range is again wide in both building axis: 1.10-2.01 in longitudinal direction and 1.43-1.75 in transversal one. Results for adopted materials' variability considering mean values for yield and plastic rotation are reported in Table 49 and Table 50 for longitudinal and transversal directions, respectively.

It is obvious that the seismic joint which divide the structure from the reinforced concrete core, as above mentioned, is not enough in case of earthquake, because the structure under seismic loads moves greater than six centimeters.

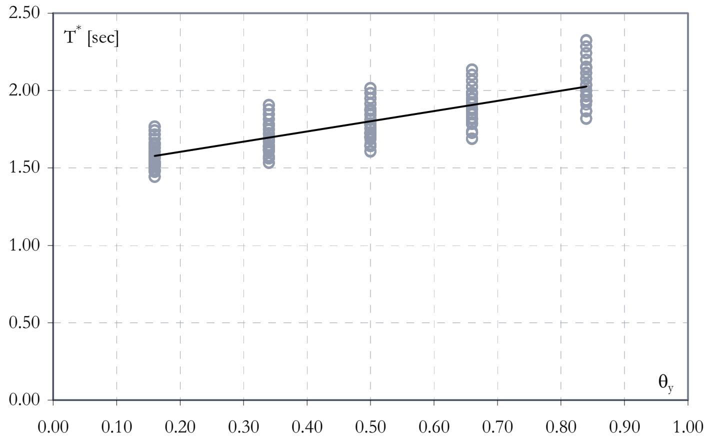


Figure 105. Effective period versus yielding rotation for all models in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

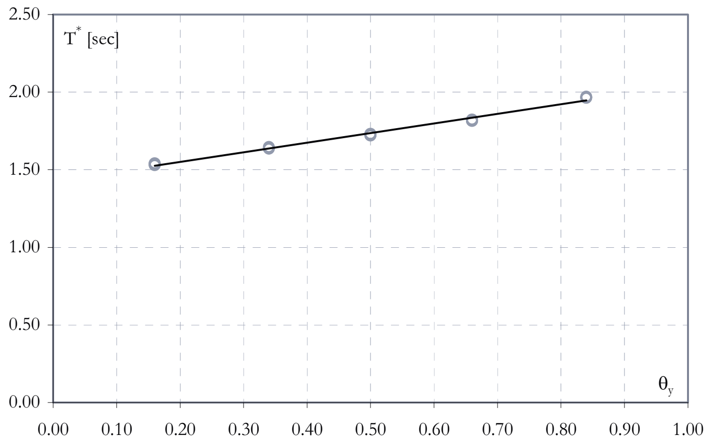


Figure 106. Effective period versus yielding rotation considering variability in thresholds in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

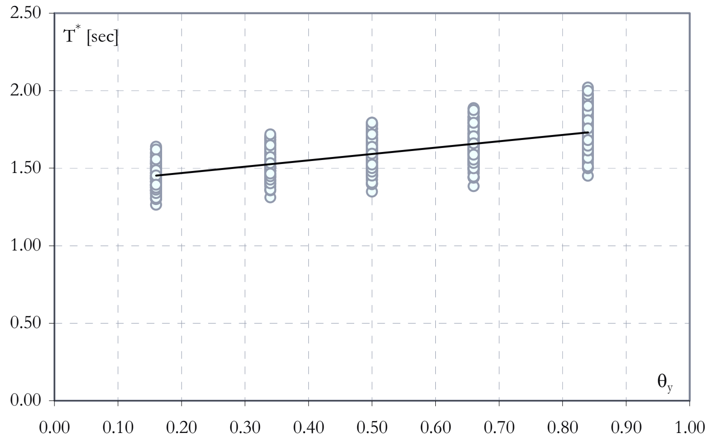


Figure 107. Effective period versus yielding rotation for all models in transversal direction – Hospital Dr. Luis Edmundo Vasquez

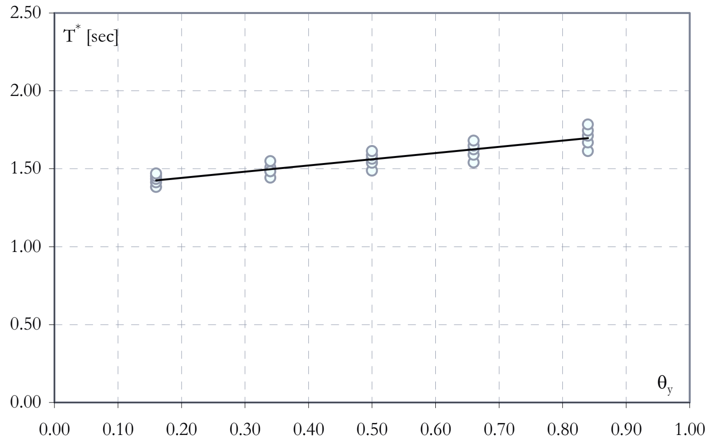


Figure 108. Effective period versus yielding rotation considering variability in thresholds in transversal direction – Hospital Dr. Luis Edmundo Vasquez

Figure 107 and Figure 108 show the linear relationship between the effective period and yield rotation; as it can be easily seen in the following, the coefficient of determination is higher considering only the variability on thresholds than in models with both variability.

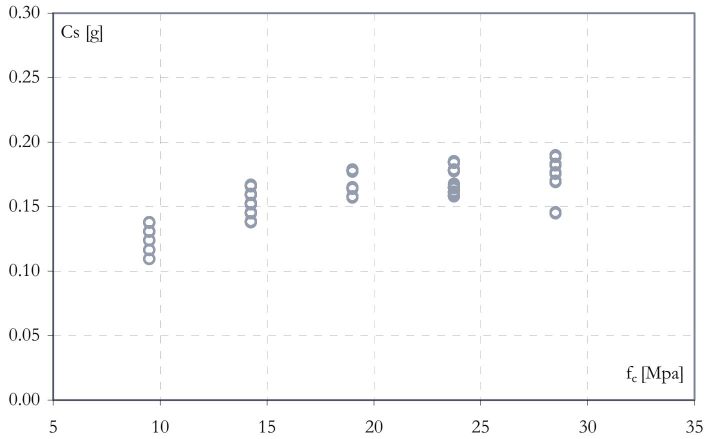


Figure 109. Cs versus concrete strength for all models in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

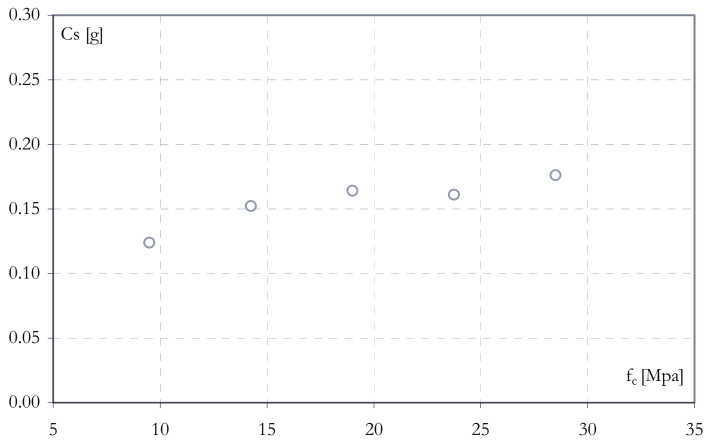


Figure 110. Cs versus concrete strength considering only the variability in concrete in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

The relationship between non-linear capacity strength and concrete resistance is non linear, very well approximated by a second degree equation.

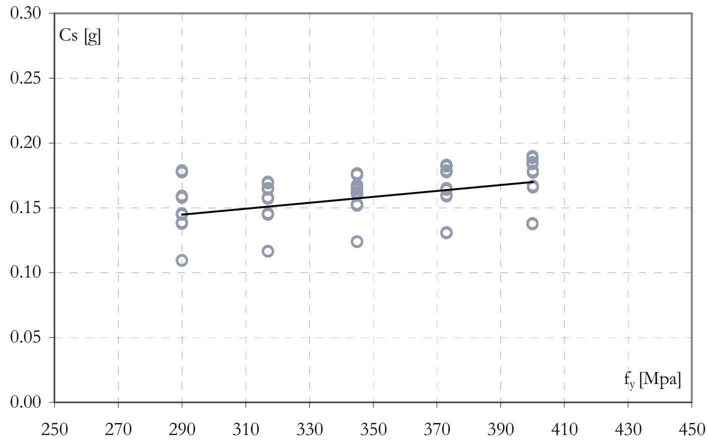


Figure 111. Cs versus steel strength for all models in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

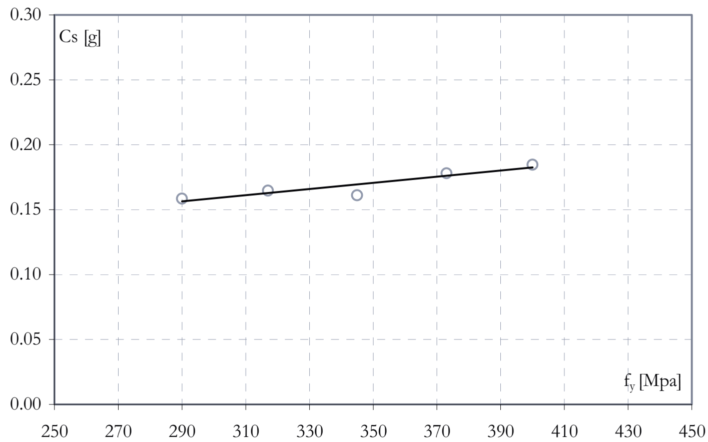


Figure 112. Cs versus steel strength considering only the variability in steel in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

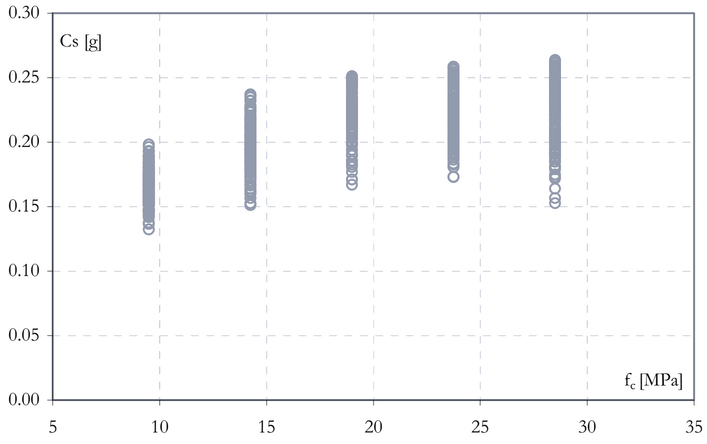


Figure 113. Cs versus concrete strength for all models in transversal direction – Hospital Dr. Luis Edmundo Vasquez

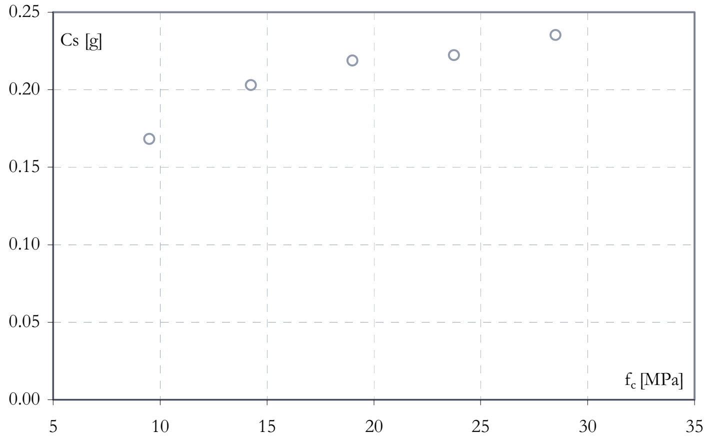


Figure 114. Cs versus concrete strength considering only the variability in concrete in transversal direction – Hospital Dr. Luis Edmundo Vasquez

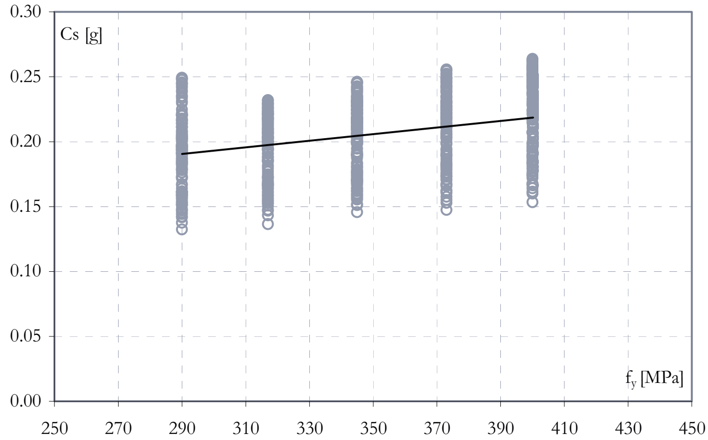


Figure 115. Cs versus steel strength for all models in transversal direction – Hospital Dr. Luis Edmundo Vasquez

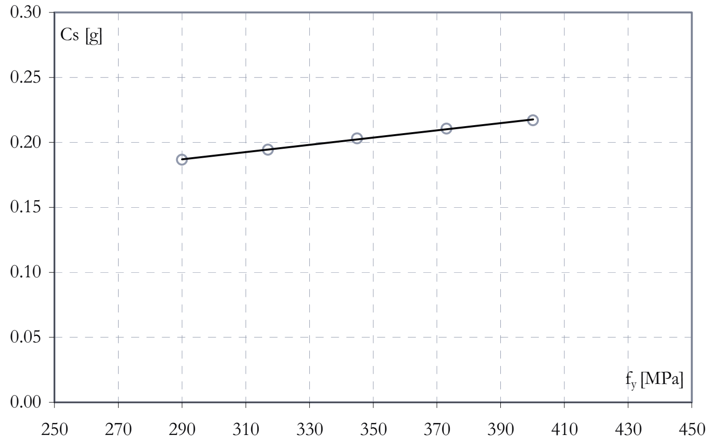


Figure 116. Cs versus steel strength considering only the variability in steel in transversal direction – Hospital Dr. Luis Edmundo Vasquez

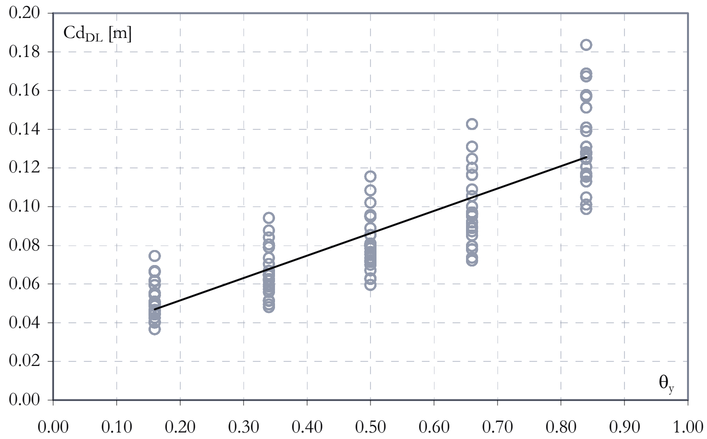


Figure 117. Cd$_{DL}$ versus yield rotation for all models in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

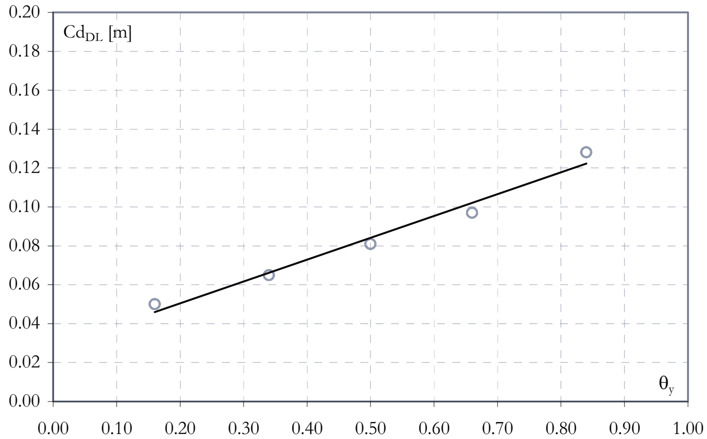


Figure 118. Cd$_{DL}$ versus yielding rotation considering variability in thresholds in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

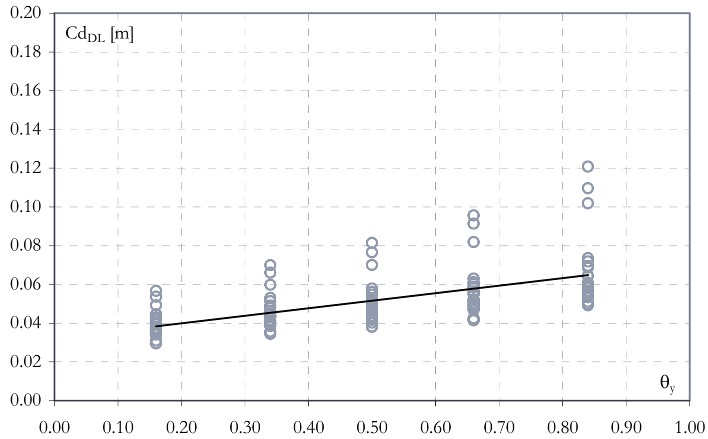


Figure 119. Cd_{DL} versus yield rotation for all models in transversal direction – Hospital Dr. Luis Edmundo Vasquez

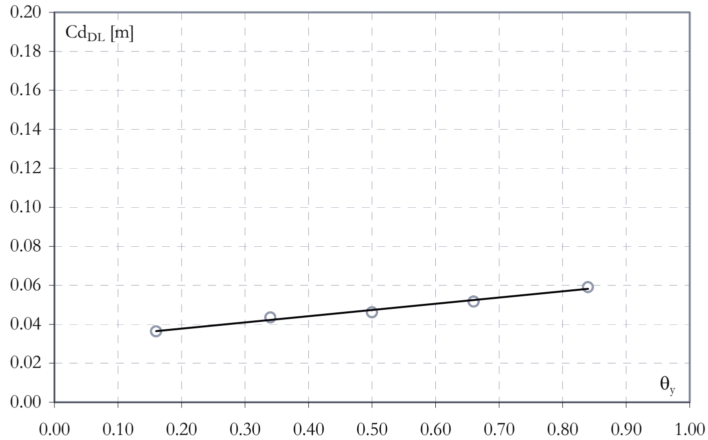


Figure 120. Cd_{DL} versus yielding rotation considering variability in thresholds in transversal direction – Hospital Dr. Luis Edmundo Vasquez

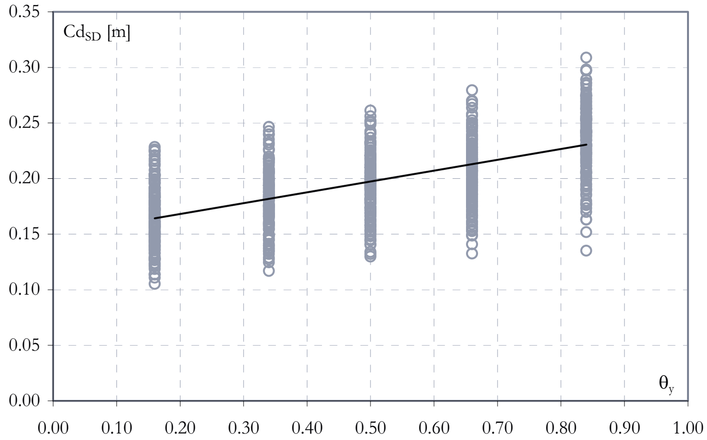


Figure 121. Cd$_{SD}$ versus yield rotation for all models in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

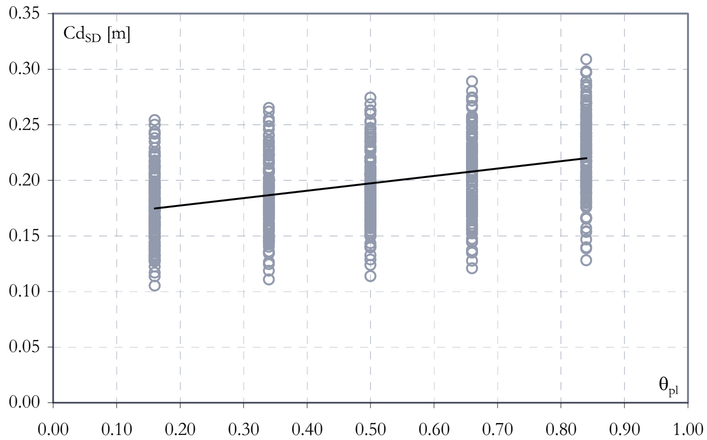


Figure 122. Cd$_{SD}$ versus plastic rotation for all models in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

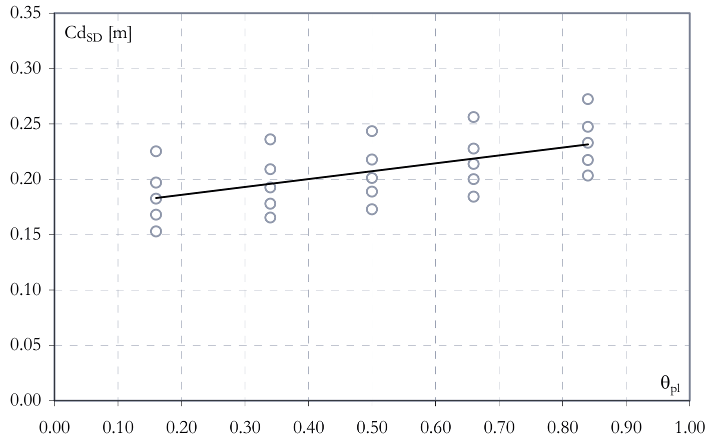


Figure 123. Cd_{SD} versus plastic rotation considering variability in thresholds in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

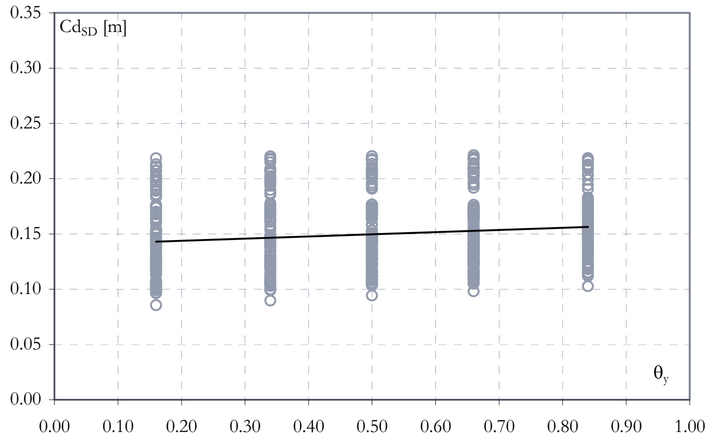


Figure 124. Cd_{SD} versus yield rotation for all models in transversal direction – Hospital Dr. Luis Edmundo Vasquez

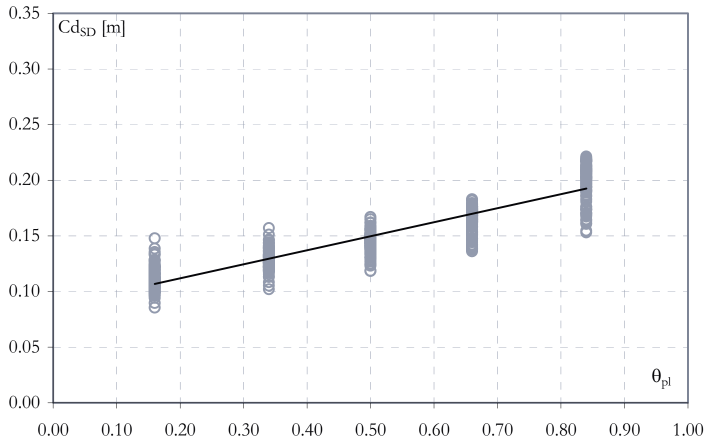


Figure 125. Cd_{SD} versus plastic rotation for all models in transversal direction – Hospital Dr. Luis Edmundo Vasquez

As it can been easily seen in Figure 124, Cd_{SD} is more dependent by plastic rotation then yield one, in fact it is linear growing with the increase of plastic part of chord rotation (see Figure 125).

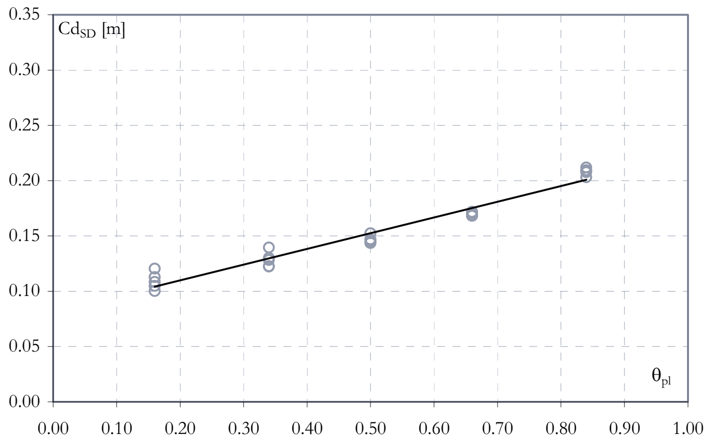


Figure 126. Cd_{SD} versus plastic rotation considering variability in thresholds in transversal direction – Hospital Dr. Luis Edmundo Vasquez

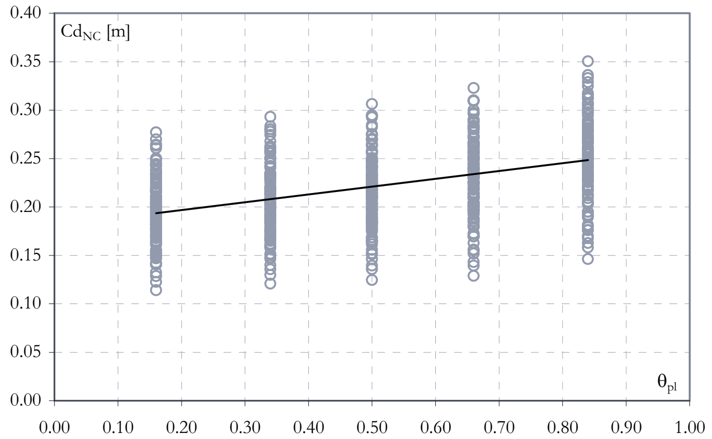


Figure 127. Cd_{NC} versus plastic rotation for all models in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

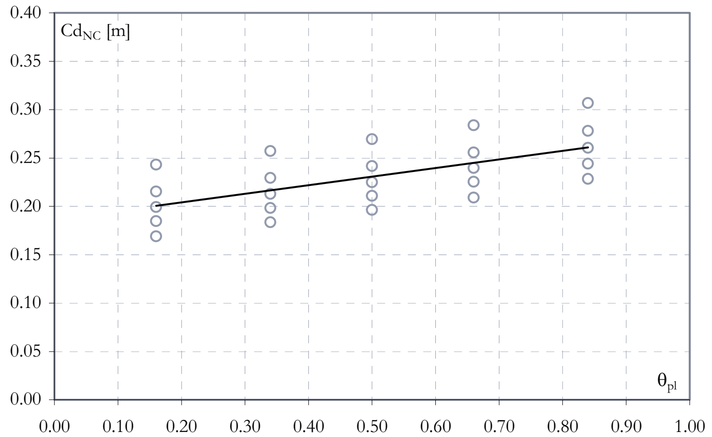


Figure 128. Cd_{NC} versus plastic rotation considering variability in thresholds in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

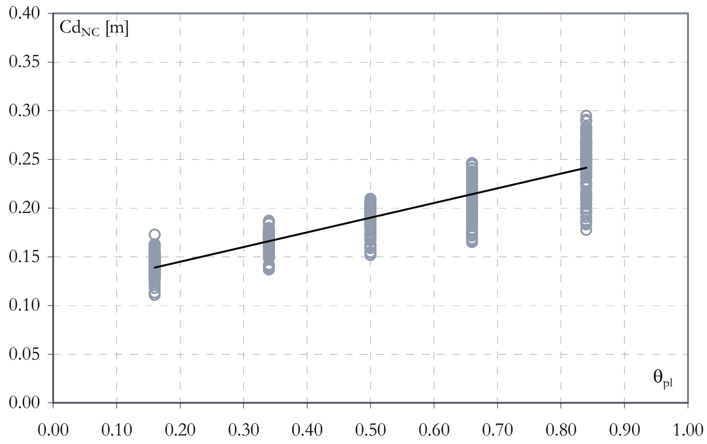


Figure 129. Cd_{NC} versus plastic rotation for all models in transversal direction – Hospital Dr. Luis Edmundo Vasquez

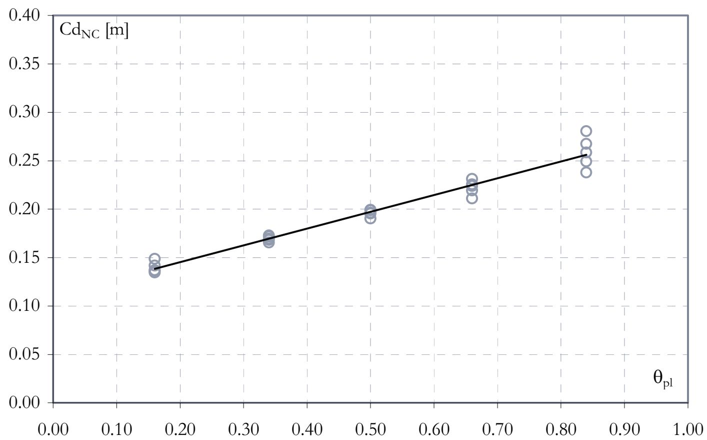


Figure 130. Cd_{NC} versus plastic rotation considering variability in thresholds in transversal direction – Hospital Dr. Luis Edmundo Vasquez

From data analysis, it appears the different dependence of non-linear parameters in output with the input variability in function of building axis. In fact, Cd_{DS} in longitudinal direction is more conditional on yield rotation than plastic one; the reason can be found in the different collapse mechanism: the collapse along longitudinal building axis is a second level mechanism, instead in transversal direction there is a global mechanism. From Figure 131 to Figure 134 show collapse mechanism.

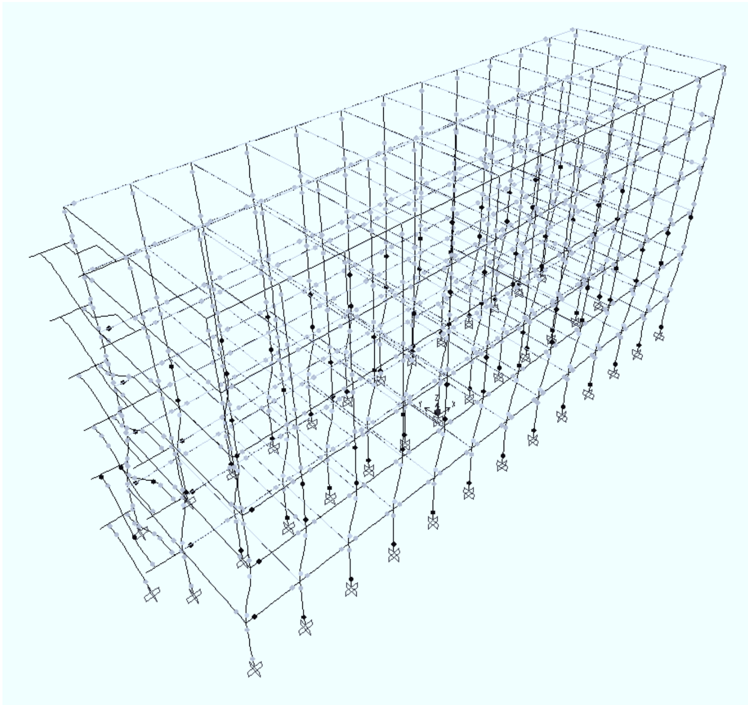


Figure 131. Plastic hinges in tri-dimension structural model in nonlinear analysis in longitudinal direction

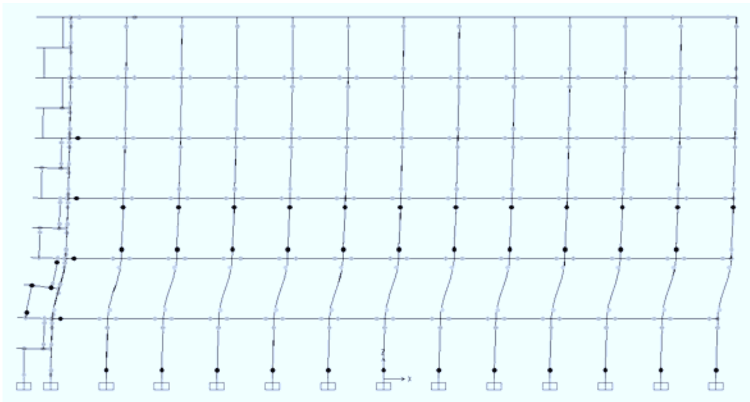


Figure 132. Soft-storey mechanism in longitudinal direction

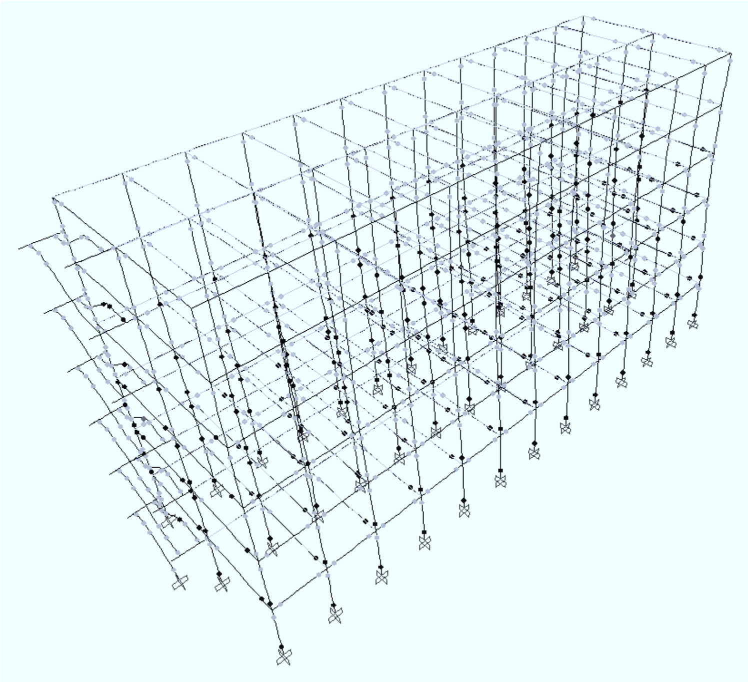


Figure 133. Plastic hinges in tri-dimension structural model in nonlinear analysis in transversal direction

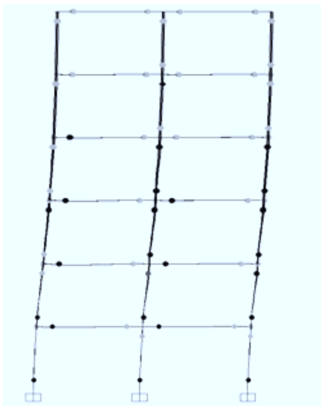


Figure 134. Global mechanism in transversal direction

To evaluate fragility curve parameters for the building the procedure described in 3.3.5 was adopted. In particular, the simulation consists of extracting a vector of the input parameters from the distributions of yielding and plastic rotations, that are the sole random variables considered in this study.

To define seismic action, the current U.S. building code IBC-2006 (ICC, 2006) is used. Local soil conditions at the building site are characterized by near-surface shear-wave velocities $v_{s,30}$ between 360 m/s and 760 m/s, i.e. NEHRP site class C according to IBC-2006 (ICC, 2006; Figure 72) that corresponds to soil type B for Eurocode 8 (CEN, 2003).

As above mentioned, for each simulation, the spectrum is entered with the T value corresponding to the random extraction and the elastic displacement demand S_d is derived. The inelastic demand is evaluated multiplying the elastic displacement demand by a modification factor C_R that depends on effective period T and on the spectral reduction factor R (Ruiz-Garcia and Miranda, 2003). Consequently, non-linear displacement capacity and demand may be compared in each run checking for failure at each of the three damage states considered. Scaling the elastic spectrum in order to investigate the demand range of interest allows to derive the fragility curves.

Figures from Figure 135 to Figure 140 show the generated fragility curves for the three considered limit states: damage limitation DL (continuous line), significant damage SD (dash line) and near collapse NC (dash-dot line) for both cases of rigid (RR) and deformable roof (DR) and in both building directions.

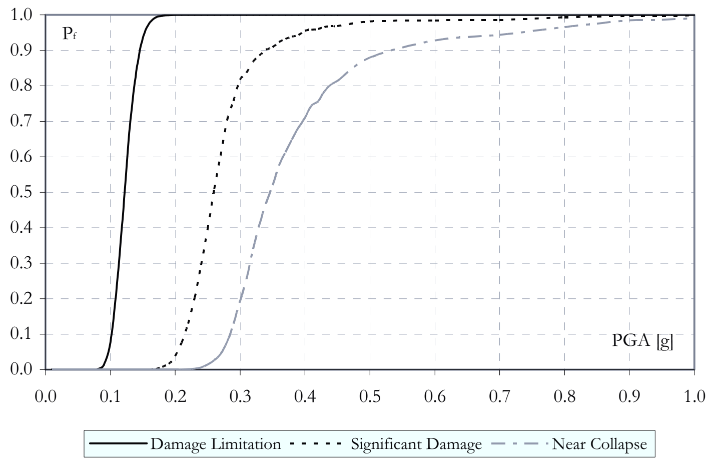


Figure 135. Fragility curves in the longitudinal building axis considering variability in materials – Hospital Dr. Luis Edmundo Vasquez

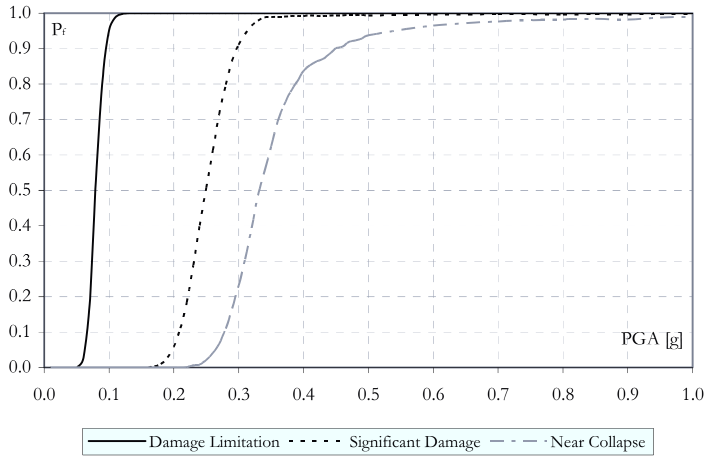


Figure 136. Fragility curves in the transversal building axis considering variability in materials – Hospital Dr. Luis Edmundo Vasquez

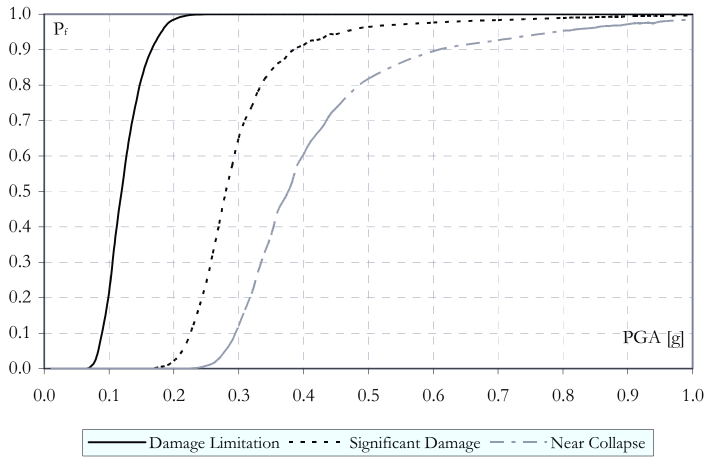


Figure 137. Fragility curves in the longitudinal building axis considering variability in thresholds – Hospital Dr. Luis Edmundo Vasquez

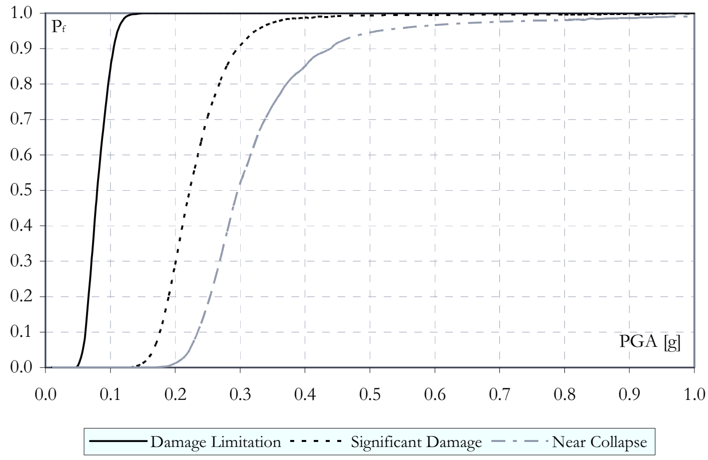


Figure 138. Fragility curves in the transversal building axis considering variability in thresholds – Hospital Dr. Luis Edmundo Vasquez

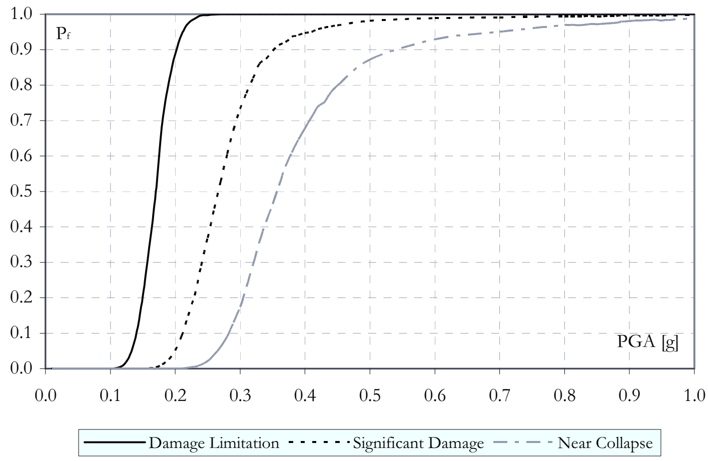


Figure 139. Fragility curves in the longitudinal building axis considering both variability – Hospital Dr. Luis Edmundo Vasquez

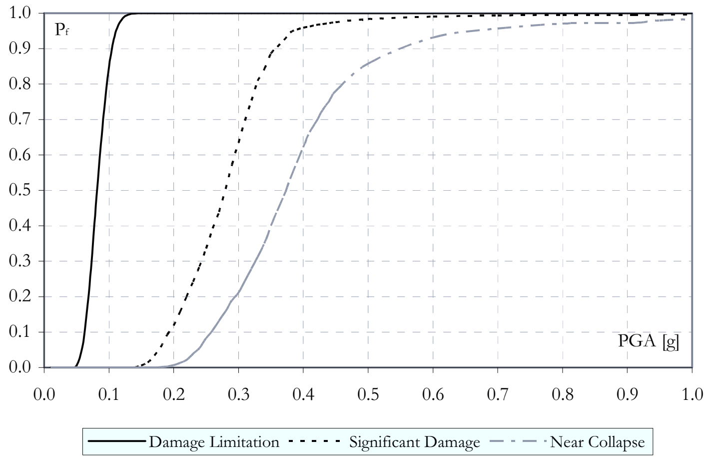


Figure 140. Fragility curves in the transversal building axis considering both variability – Hospital Dr. Luis Edmundo Vasquez

Comparing fragility function with HAZUS approach, the standard deviation β of natural logarithm of spectral displacement for the considered damage state, is obtained. In Table 39, β as got and median value of PGA are reported.

	DL		***SD***		***NC***	
	β	PGA_m	β	PGA_m	β	PGA_m
Materials' variability – longitudinal direction	0.12	0.13	0.15	0.26	0.25	0.35
Materials' variability – transversal direction	0.15	0.08	0.14	0.25	0.16	0.34
Thresholds' variability – longitudinal direction	0.23	0.12	0.17	0.28	0.21	0.38
Thresholds' variability – transversal direction	0.21	0.08	0.20	0.22	0.25	0.30
Both variability – longitudinal direction	0.15	0.17	0.20	0.27	0.20	0.36
Both variability – transversal direction	0.19	0.08	0.21	0.28	0.25	0.38

Table 46. β for all fragility function – Hospital Dr. Luis Edmundo Vasquez

As written in HAZUS, the original development of damage-state fragility curves for generic model building were based on capacity variability, $\beta_C = 0.3$ in case of pre-code buildings, damage-state threshold variability, $\beta_{T,ds} = 0.4$ (Structure) and demand $\beta_D = 0.45$ [44]. In this work, the β becomes 0.42 because all variability are considered:

$$\beta = \sqrt{CONV\left(\beta_C, \beta_D\right)^2 + \beta_{T,ds}^2} \tag{48}$$

So comparing results reported in Table 39 with HAZUS' indication for β_C, β_D and $\beta_{T,ds}$, it is observable that they are little lower than HAZUS' ones.

A comparison with the equivalent PGA adopted in case of Pre-Code designed structures, for concrete moment frame (C1M: 5 stories – Mid-Rise) is done. In HAZUS, it is 0.09 g, 0.13 g and 0.26 g for slight, moderate and extensive limit state, respectively. So, considering that the definition of limit states adopted in this work isn't the same in HAZUS, results are satisfactory.

Using a HAZUS formula, with β reported in Table 46, fragility curves are derived. It is clear the very good approximation between the adopted approach and the HAZUS' one. From Figure 141 to Figure 146, fragility curves derived with HAZUS approach, considering PGA mean and β value reported in Table 46, for all cases.

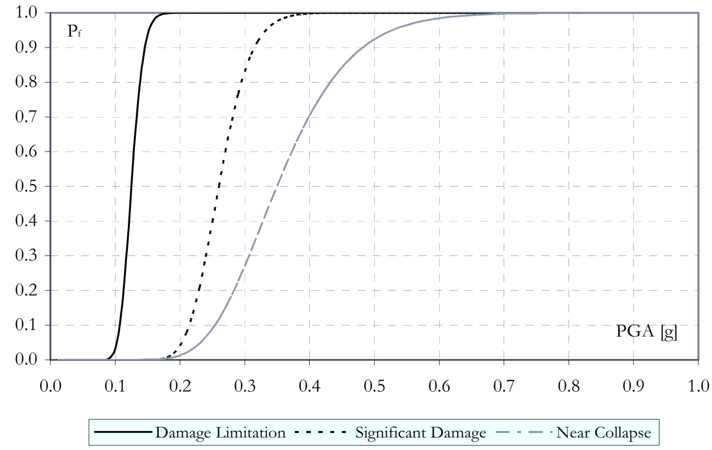


Figure 141. Fragility curves derived with HAZUS' formulation considering variability in materials in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

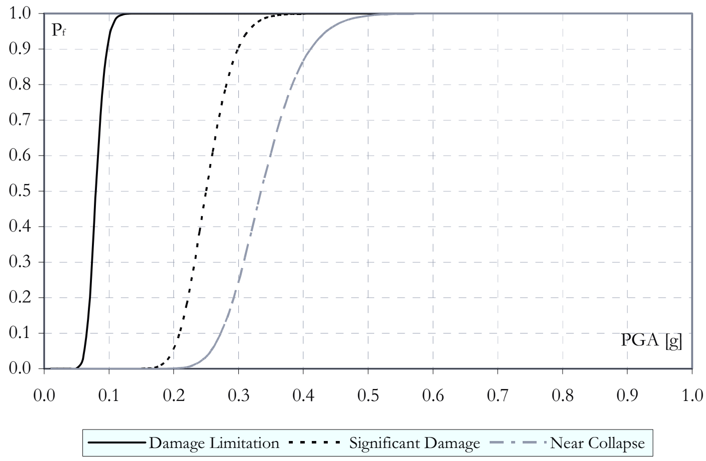


Figure 142. Fragility curves derived with HAZUS' formulation considering variability in materials in transversal direction – Hospital Dr. Luis Edmundo Vasquez

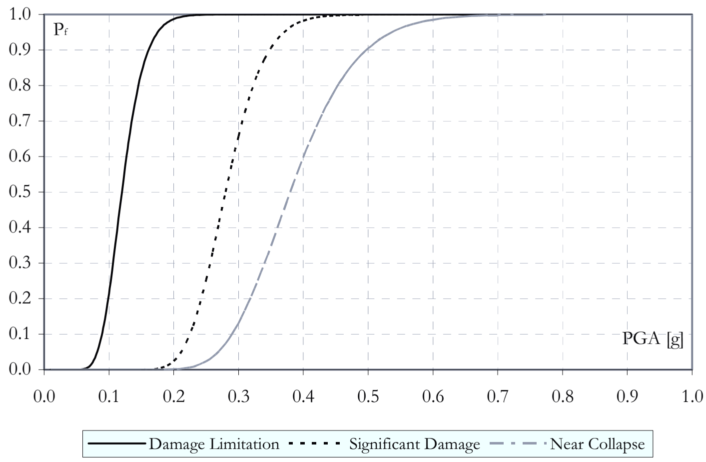


Figure 143. Fragility curves derived with HAZUS' formulation considering variability in thresholds in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

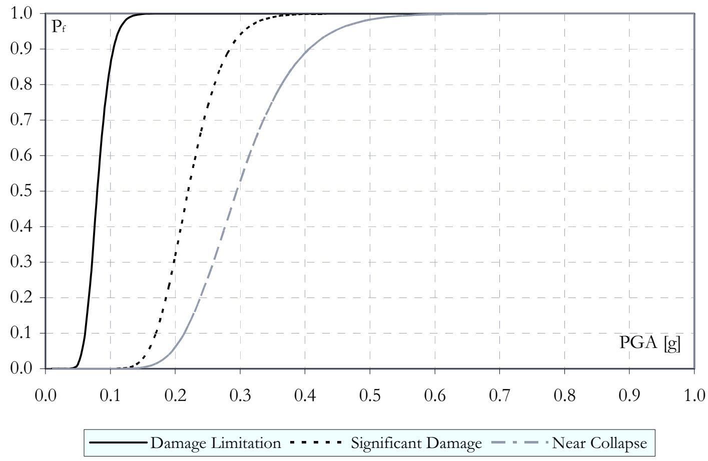


Figure 144. Fragility curves derived with HAZUS' formulation considering variability in thresholds in transversal direction – Hospital Dr. Luis Edmundo Vasquez

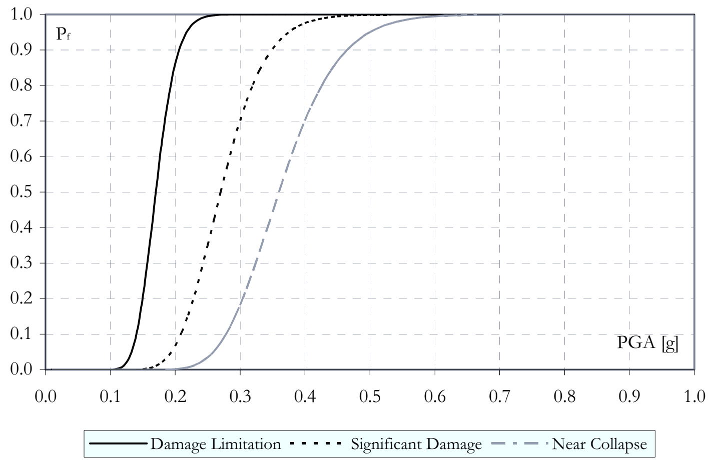


Figure 145. Fragility curves derived with HAZUS' formulation considering both variability in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

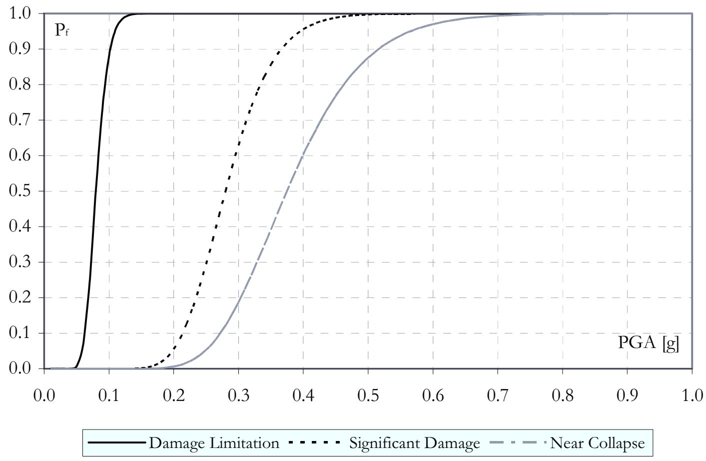


Figure 146. Fragility curves derived with HAZUS' formulation considering both variability in transversal direction – Hospital Dr. Luis Edmundo Vasquez

The seismic hazard map of the Americas produced as part of the Global Seismic Hazard Assessment Program (GSHAP) indicates almost Constant hazard throughout El Salvador, with 475-year PGA on the order of 0.5 g (Sheldock, 1999). The Central American hazard maps generated as a part of general earthquake loss estimation model by Chen et al. (2002) indicate a 475-year PGA level of 0.2-0.4 g in northern El Salvador and 0.4-0.8 g in the southern and western parts of the country.[47]

Considering data [48] about the earthquake happened in El Salvador in 2001 January 13, the epicentre was characterized by following coordinates: 13.049 N and 88.660 W, with depth of 60 Km, the proximate sensor was in Santa Ana and it reported a PGA of about 0.14 g in x direction and 0.09 g in y direction, so a verify of this value (the hospital survived to 2001 earthquake) on fragility curve is done in both building axis.

As it can be easily seen in Figure 147 and Figure 148, only damage limitation hit the structure, in both directions, as confirmation of building's survival during that earthquake.

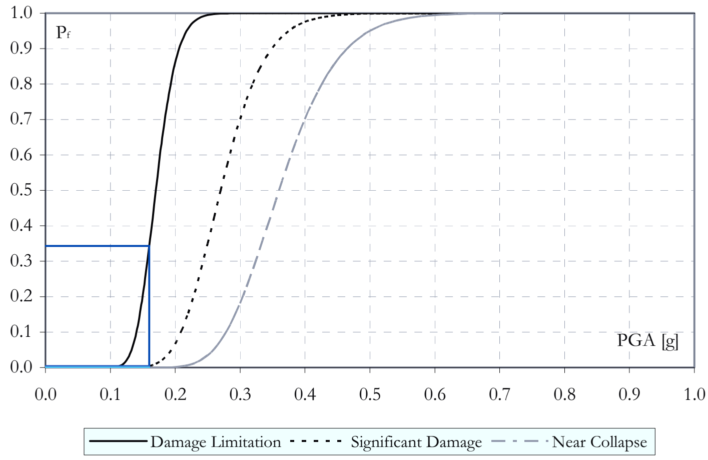


Figure 147. Pf in case of 2001 earthquake in San Salvador considering fragility for both variability in longitudinal direction – Hospital Dr. Luis Edmundo Vasquez

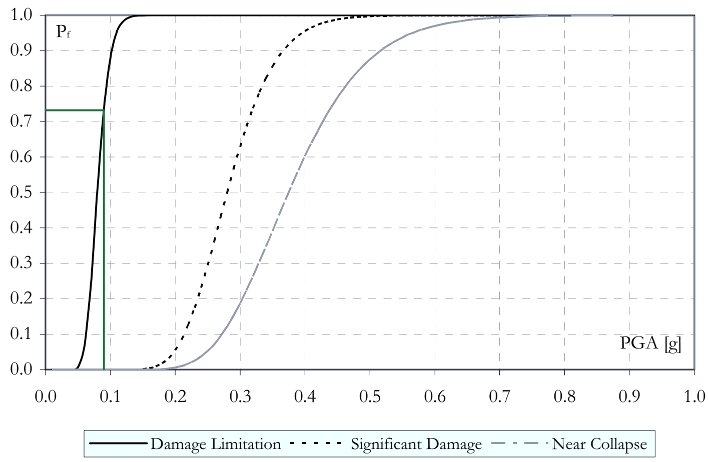


Figure 148. Pf in case of 2001 earthquake in San Salvador considering fragility for both variability in transversal direction – Hospital Dr. Luis Edmundo Vasquez

5.3. Final remarks

Done visual inspection ad hoc on schools and hospitals in Central American Countries, and applied the methodology based on questionnaires for structural and non-structural vulnerability index to have a list of priority of buildings in need of attention, two representative buildings are chosen (one for each category).

To compensate the lack in some case of information, a parametric approach was followed; in fact, the strength of concrete and steel was not computed by in situ destroyed tests, but a normal distribution of mean value, extrapolated by American Code at construction year and construction practise used in these countries, was adopted. The same think is done with models; a lognormal variability in thresholds definition is assumed to taken into account the dispersion in rotation values computed with regression formulas.

To compute structural vulnerability of reinforced concrete existing structures detailed analysis are used; structural models and the corresponding non-linear lumped plasticity models are generated. The seismic capacity is determined via pushover analysis and by the transformation of the equivalent SDOF capacity curve into bilinear form. The resulting lateral strength and displacement capacity are considered for selected limit states; also the effective period is retrieved.

Combining structural capacity with seismic demand through Capacity Spectrum Method, site dependent fragility curves, in terms of PGA (peak ground acceleration) are derived for three limit states: slight damage, severe damage and near collapse state. Uncertainties in the fragilities account for are those of the surveyed parameters; moreover variability of the limit state thresholds and of inelastic demand are also included.

Relationships existing between non-linear capacity parameters and the variability adopted in input are studied. In fact, the importance to consider thresholds variability in the evaluation of nonlinear parameters as the capacity displacement and the effective period is emphasized. The influence of thresholds variability on nonlinear structure capacity is evaluated.

Coherently, the variability of the displacement capacity at the ultimate limit state is directly imputable to the variability of the ultimate rotation characterizing the failing element. With θ_y - θ_u variability, C_s does not vary, fixed strength materials, instead a variability is reported when a variability in materials strength f_c - f_y is adopted. On the other hand, varying θ_y - θ_u, C_d and T change: C_d is influenced more by θ_u, especially for SD and NC limit states, T, instead, by θ_y, in fact T increases proportionally with increase of θ_y.

Obtained fragility, results reached with European approach are compared with HAZUS' one; satisfactory outcomes are obtained.

A comparison between fragility considering thresholds variability for both school and hospital structures is done. As it can be seen, the probability of failure for school is higher than hospital's one, except for damage limitation. The result reflects the relationship between structural vulnerability index evaluated for both structures with the questionnaires' methodology, as testified in Appendix A.

Full of promise the comparison strategy between analytical approach and empirical one based on questionnaires to check the survey card formulated ad hoc to evaluation of structural vulnerability index.

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Appendix A: Visual Inspections

Following, the results of questionnaire applications to school Republica de Colombia and hospital Dr. Luis Edmundo Vasquez are reported.

Hospital Dr. Luis Edmundo Vasquez, Chalatenango – El Salvador

SEISMIC VULNERABILITY ASSESSMENT OF HOSPITALS BASED ON QUESTIONNAIRE SURVEY

NORSAR

General Information

INDEX: H-01

Name (ID):	**Dr. Luis Ermundo Vasquez, Chalatenango**		
Address:	Barrio San Antonio Contiguo a los Pinares Chalatenango		
Contact person:	José Trinidad Palma, Marco Antonio Aguilar		
Coordinates:	Latitude	14.0385	
	Longitude	-88.9362	
Structural characteristics:	Typology of the primary structure:		
	RC frames with masonry infill walls		
	no. of stories	5	
	basements:	1	
	interstory height:	4.0	*m*
	no. of cores:	1	
	plan shape:	◦ L U T	
	max length *L*:	42.0	*m*
	max width *W*:	13.0	*m*
Photo ID's:	DSC 5307 - 5342		
Screener/date:	DHL, DWD, MG / Feb 16, 2009		

Occupancy:	Hospital	Health center	Laboratory
	other: ______________________________		
No. of:	occupants:	>100	
	beds:	100	
	patients:	100	
	medical staff:	24+282=306	
Occupancy period:	24 h	12 h	8h
	from: __________	to: __________	
Age:	< 10 years (AF: 1.00)	10-20 years (AF: 1.025)	
	20-40 years (AF: 1.05)	> 40 years (AF: 1.10)	
	year of construction: 1971		Age factor: 1.050
Actual state:	good (new)	(ASF : 1.00)	
	recently renovated	(ASF : 1.05)	
	in need of renovation	(ASF : 1.10)	
	bad (decayed)	(ASF : 1.20)	Actual state factor: 1.100
Topography:	plane, flat	foothill (base of slope)	other:
	sediment basin (valley)	slope situation	
	close to river	ridge (top of slope; hilltop)	
Maintenance program:	exists	does not exist	
	if yes, in which period:	annual	
Comments:	suffered structural damage during earthquake in 2001		

Note: In case that a hospital consists of several buildings physically separated from each other, separate questionnaire forms should be filled in for each building (block).

Figure 149. General Information – Hospital Dr. Luis Edmundo Vasquez

SEISMIC VULNERABILITY ASSESSMENT OF HOSPITALS BASED ON QUESTIONNAIRE SURVEY

Structural Vulnerability Index *SVI*

SVI = **3.7 (4.3)**

ITEM		Reinforced Concrete Building: YES	NO	NA	Score	Masonry Building: YES	NO	NA	Score
1	Is the building irregular in plan?	0	1		0	0	0		0
2	Are the columns regularly distributed?	1	0		0				
3	Are both building directions adequately braced (RC frames or shear walls, masonry walls)?	1	0		0	0	0		0
4	Does the ratio between the building's length and width is > 2.5 ?	1	0		4	0	0		0
5	Does the building possess eccentric cores (staircases or elevators)?	1	0		8	0	0		0
6	Does the building have a soft storey?	1	0	0	16				
7	Is the building irregular in elevation caused by setbacks of upper stories?	0	1	0	0	0	0	0	0
8	Does the building have cantilevering upper stories?	0	1	0	0	0	0	0	0
9	Does the building possess a heavy mass at the top or at roof level?	0	1		0	0	0		0
10	Are pounding effects possible?	1	0		4	0	0		0
11	Does the building have short columns?	1	0		8				
12	Are strong beams–weak columns available?	0	1		0				
13	Does the building possess shear walls ?	0	1		4				
14	Did the building suffer any significant structural damage in the past?	1	0		4	0	0		0
15	Does the building possess seismic retrofitting or strengthening measures?	0	1		8	0	0		0

	Reinforced Concrete Building		Masonry Building
SUM:	**56**	**SUM:**	**0**
Answered questions:	**15**	**Answered questions:**	**10**
SVI:	**3.7**	**SVI:**	**0**
Age factor:	**1.050**	**Age factor:**	**1.050**
Actual state factor:	**1.100**	**Actual state factor:**	**1.100**
(SVI):	**(4.3)**	**(SVI):**	**0**

Note: (1) Answering a question is done by inserting integer '1' **either** at YES, NO or NA.
(2) Gray-shaded fields cannot be filled.
(3) For single-story buildings, the questions no. (6), 7 and 8 cannot be answered.

Figure 150. Structural Vulnerability Index SVI – Hospital Dr. Luis Edmundo Vasquez

SEISMIC VULNERABILITY ASSESSMENT OF HOSPITALS BASED ON QUESTIONNAIRE SURVEY

Non-Structural Vulnerability Index *NVI*

NVI = 6.4

	ITEM	YES	NO	NA	Score
I. Electrical Facilities					
1	Is there an emergency generator and fuel tank available?	1	0		0
2	If yes, are both located outside the building? (if Q1 = NO → NA)	1	0	0	0
3	If outside, in a certain distance such that e.g. parts of the building can not fall on them? (if Q1 = NO → NA)	0	1	0	8
4	Are they adequately secured? (if Q1 = NO → NA)	1	0	0	0
5	Are service lines and other pipes attached with flexible connections?	0	1		16
6	Are they able to accommodate relative movement across joints?	0	1	0	16
7	Are bus ducts and cables able to distort at their connections to equipment without rupture?	0	1		8
8	Are they able to accommodate relative movement across joints?	0	1		8
II. Fire Fighting					
9	Are there smoke detectors and alarms available?	0	1		4
10	Are there enough fire extinguishers and hose-reel cabinets available?	0	1		16
11	Are they easily accessible? (if Q10 = NO → NA)	0	1	0	16
12	Is the emergency water tank located outside the building?	1	0		0
13	If located outside, can it be damaged during an earthquake by falling parts? (if Q12 = NO → NA)	0	1	0	0
III. Propane pipes or any other gas pipes (e.g., oxygen)					
14	Does the system have an automatic, earthquake-triggered shut-off valve?	0	1		16
15	If not, can it be easily closed manually e.g. by a wrench tool stored close by? (if Q14 = YES → NA)	1	0	0	0
16	Are supply pipes able to accommodate relative movement across joints and at the tank?	0	1		16
17	Are supply pipes able to distort at their connections to equipment without rupture?	0	1		16
IV. Elevators					
18	Are elevators available?	1	0		4
19	Are elevators maintained and are they regularly (every 2 months) controlled? (if Q18 = NO → NA)	0	1	0	4
20	Are motors and control cabinets anchored to the floor? (if Q18 = NO → NA)	1	0	0	0
V. Non-structural Infill Walls and Partitions					
21	Are (infill) brick walls protected against out-of-plane failure by e.g. internal reinforcement or surface meshes?	0	1		8
22	Do movement joints between infill walls and RC frames exist to allow damage-free movement? (for masonry buildings → NA)	1	0	0	0
VI. Ceilings					
23	Are suspended ceilings available?	1	0		4
24	Are the suspended ceilings adequately secured against failure? (if Q23 = NO → NA)	0	1	0	4
VII. Emergency Exits and Escape Routes					
25	If exit fire doors jam in an earthquake, is there a crowbar or sledge hammer readily available to facilitate emergency opening?	0	1		16
26	Do all exit doors open outwards?	1	0		0
27	Are all doors unlocked from the inside and also unblocked?	0	1		16
28	Are automatic doors available?	0	1		0
29	Do automatic doors have manual overrides? (if Q28 = NO → NA)	0	0	1	0
30	Has the glazing of windows been designed to accommodate lateral movement?	0	1		4
31	Do large windows, door transoms and skylights have safety glass?	0	1		4
32	Are emergency exits and escape routes adequately designated, e.g. by fluorescent signs?	0	1		8
33	Are emergency exits and escape routes adequately illuminated?	0	1		8
VIII. Appendages					
34	Can nonstructural elements (e.g. parapets, facade cladding, roof tiles, chimneys, external AC machines) fall from the building and harm people running outside?	0	1		0
IX. Movable Equipment					
35	Are gas cylinders tightly secured with chains at top and bottom (or otherwise)?	0	1		8
36	Are chemicals stored in accordance with manufacturers recommendations?	0	1		4
37	Are cabinets for hazardous materials given special attention with respect to anchoring?	0	1		8
X. Appurtenant structures					
38	Are enough open spaces around the building to be used as escape routes and where people are safe from falling objects?	1	0		0
39	Can neighboring structures (also walls, electricity lines) block escape routes or harm people running/gathering outside?	0	1		0
40	Can road access to and from the hospital be blocked due to collapse of buildings or geotechnical effects (landslides etc.)?	1	0		8

Note: (1) Answering a question is done by inserting integer '1' **either** at YES, NO or NA.
(2) Gray-shaded fields cannot be filled.

SUM: 248
Answered questions: 39
NVI: 6.4

Figure 151. Non-Structural Vulnerability Index NVI – Hospital Dr. Luis Edmundo Vasquez

School Republica de Colombia – Guatemala

SEISMIC VULNERABILITY ASSESSMENT OF SCHOOLS BASED ON QUESTIONNAIRE SURVEY

General Information

INDEX: S-05

Name (ID):	**C.E. Republica de Colombia**
Address:	Zona 11 Guatemala City
Contact person:	Omar Flores
Coordinates:	Latitude 14.6097 Longitude -90.5493
Structural characteristics:	Typology of the primary structure: RC frames with masonry infills no. of individual buildings: 1 no. of stories (b.): 2 interstory height: 3.0 m no. of cores: - plan shape: ☒ ☐ ☐ L ☐ U ☐ T max length L: 57.0 m max width W: 20.6 m
Photo ID's:	DSC 4167-4224 P1000586 - P1000609
Screener/date:	MIV, DHL / June 12, 2008 DHL / Feb 26, 2007

Occupancy:	☒ School ☐ Kindergarten ☐ University ☐ other: ____________
No. of:	☐ pupils/students: among disabled: ☐ teachers/employees: ☐ classrooms: ☐ total classroom area: m^2
Occupancy period:	☐ 24 h ☐ 12 h ☐ other: from: ________ to: ________
Age:	☐ < 10 years (AF: 1.00) ☐ 10-20 years (AF: 1.025) ☐ 20-40 years (AF: 1.05) ☒ > 40 years (AF: 1.10) year of construction: 1965 Age factor: 1.100
Actual state:	☐ good (new) (ASF : 1.00) ☐ recently renovated (ASF : 1.05) ☐ in need of renovation (ASF : 1.10) ☒ bad (decayed) (ASF : 1.20) Actual state factor: 1.200
Topography:	☒ plane, flat ☐ foothill (base of slope) ☐ other: ☐ sediment basin (valley) ☐ slope situation ☐ close to river ☐ ridge (top of slope; hilltop)
Maintenance program:	☐ exists ☒ does not exist if yes, in which period: ________
Comments:	

Note: In case that a school consists of several buildings physically separated from each other, separate questionnaire forms should be filled in for each building (block).

Figure 152. General Information – School Republica de Colombia

SEISMIC VULNERABILITY ASSESSMENT OF SCHOOLS BASED ON QUESTIONNAIRE SURVEY

Structural Vulnerability Index *SVI*

SVI = 4.5 (6.0)

	ITEM	Reinforced Concrete Building:				Masonry Building:			
		YES	NO	NA	Score	YES	NO	NA	Score
1	Is the building irregular in plan?	0	1		0	0	0		0
2	Are the columns regularly distributed?	1	0		0				
3	Are both building directions adequately braced (RC frames/shear walls, masonry walls)?	0	1		16	0	0		0
4	Does the ratio between the building's length and width is > 2.5 ?	1	0		4	0	0		0
5	Does the building possess eccentric cores (staircases or elevators)?	1	0		8	0	0		0
6	Does the building have a soft storey?	0	1	0	0				
7	Is the building irregular in elevation caused by setbacks of upper stories?	0	1	0	0	0	0	0	0
8	Does the building have cantilevering upper stories?	0	1	0	0	0	0	0	0
9	Does the building possess a heavy mass at the top or at roof level?	0	1		0	0	0		0
10	Are pounding effects possible?	0	1		0	0	0		0
11	Does the building have short columns?	1	0		8				
12	Are strong beams–weak columns available?	1	0		16				
13	Does the building possess shear walls ?	0	1		4				
14	Did the building suffer any significant structural damage in the past?	1	0		4	0	0		0
15	Does the building possess seismic retrofitting or strengthening measures?	0	1		8	0	0		0

	Reinforced Concrete Building	Masonry Building
SUM:	68	0
Answered questions:	15	10
SVI:	4.5	0
Age factor:	1.10	1.10
Actual state factor:	1.20	1.20
(SVI):	(6.0)	0

Note: (1) Answering a question is done by inserting integer '1' **either** at YES, NO or NA.
(2) Gray-shaded fields cannot be filled.
(3) For single-story buildings, the questions no. (6), 7 and 8 cannot be answered.

Figure 153. Structural Vulnerability Index SVI – School Republica de Colombia

SEISMIC VULNERABILITY ASSESSMENT OF SCHOOLS BASED ON QUESTIONNAIRE SURVEY

Non-Structural Vulnerability Index *NVI*

NVI = 5.7

	ITEM	YES	NO	NA	Score
I. Fire Fighting					
1	Are there smoke detectors and alarms available?	0	1		4
2	Are there enough fire extinguishers and hose-reel cabinets available?	0	1		8
3	Are they easily accessible? (if Q2 = NO → NA)	0	0	1	0
II. Elevators					
4	Are elevators available?	0	1		0
5	Are elevators maintained and are they regularly (every 2 months) controlled? (if Q4 = NO → NA)	0	0	1	0
6	Are motors and control cabinets anchored to the floor? (if Q4 = NO → NA)	0	0	1	0
III. Non-structural Infill Walls and Partitions					
7	Are (infill) brick walls protected against out-of-plane failure by e.g. internal reinforcement or surface meshes?	0	1		8
8	Do movement joints between brick infill walls and RC frames exist to allow damage-free movement? (for masonry → NA)	0	1	0	8
IV. Ceilings					
9	Are suspended ceilings available?	0	1		0
10	Are the suspended ceilings adequately secured against failure? (if Q9 = NO → NA)	0	0	1	0
V. Emergency Exits and Escape Routes					
11	If exit fire doors jam in an earthquake, is there a crowbar or sledge hammer readily available to facilitate emergency opening?	0	1		16
12	Do all exit doors open outwards?	0	1		16
13	Are all doors unlocked from the inside and also unblocked?	0	1		16
14	Are the windows of ground floor barred/trellised?	1	0		8
15	Are glazed windows available?	1	0		8
16	Has the glazing of windows been designed to accommodate lateral movement? (if Q15 = NO → NA)	0	1	0	8
17	Do large windows, door transoms and skylights have safety glass? (if Q15 = NO → NA)	0	1	0	8
18	Are emergency exits and escape routes adequately designated, e.g. by fluorescent signs?	0	1		4
19	Are emergency exits and escape routes adequately illuminated?	1	0		0

	ITEM	YES	NO	NA	Score
VI. Appendages					
20	Can nonstructural elements (e.g. parapets, facade cladding, roof tiles, chimneys) fall from the building and harm children or teachers running outside?	0	1		0
VII. Movable Equipment					
21	Are wardrobes/lockers/bookshelves/blackboards adequately anchored to the walls?	0	1		8
22	Are tables stable enough to protect children from falling objects (e.g. suspended ceilings)?	1	0		0
VIII. Appurtenant structures					
23	Are enough open spaces around the building to be used as escape routes and where people are safe from falling objects?	1	0		0
24	Can neighboring structures (also walls, electricity lines) block escape routes or harm people running/gathering outside?	0	1		0
25	Can road access to and from the school be blocked due to collapse of buildings or geotechnical effects (landslides etc.)?	0	1		0

Note: (1) Answering a question is done by inserting integer '1' **either** at YES, NO or NA.
(2) Gray-shaded fields cannot be filled.

SUM: 120
Answered questions: 21
NVI: 5.7

Figure 154. Non-Structural Vulnerability Index NVI – School Republica de Colombia

Appendix B: Non-linear parameters for Hospital Dr. Luis Edmundo Vasquez

	$\varsigma\, y_{percentile}$	$\varsigma\, pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
301	0.16	0.16	19	345	1.5294	0.1639	0.0500	0.1528	0.1689
302	0.16	0.34	19	345	1.5323	0.1643	0.0500	0.1653	0.1835
303	0.16	0.5	19	345	1.5347	0.1646	0.0500	0.1728	0.1962
304	0.16	0.66	19	345	1.5376	0.1649	0.0500	0.1842	0.2091
305	0.16	0.84	19	345	1.5409	0.1652	0.0500	0.2033	0.2284
306	0.34	0.16	19	345	1.6348	0.1636	0.0649	0.1680	0.1848
307	0.34	0.34	19	345	1.6372	0.1640	0.0649	0.1779	0.1982
308	0.34	0.5	19	345	1.6394	0.1642	0.0649	0.1889	0.2108
309	0.34	0.66	19	345	1.6423	0.1645	0.0649	0.2001	0.2254
310	0.34	0.84	19	345	1.6457	0.1648	0.0649	0.2172	0.2442
311	0.5	0.16	19	345	1.7200	0.1636	0.0809	0.1825	0.1992
312	0.5	0.34	19	345	1.7224	0.1639	0.0809	0.1925	0.2129
313	0.5	0.5	19	345	1.7243	0.1642	0.0809	0.2010	0.2249
314	0.5	0.66	19	345	1.7270	0.1645	0.0809	0.2138	0.2397
315	0.5	0.84	19	345	1.7307	0.1648	0.0809	0.2328	0.2604
316	0.66	0.16	19	345	1.8138	0.1639	0.0971	0.1968	0.2152
317	0.66	0.34	19	345	1.8160	0.1642	0.0971	0.2090	0.2294
318	0.66	0.5	19	345	1.8178	0.1644	0.0971	0.2178	0.2415
319	0.66	0.66	19	345	1.8202	0.1647	0.0971	0.2275	0.2555
320	0.66	0.84	19	345	1.8238	0.1650	0.0971	0.2473	0.2781
321	0.84	0.16	19	345	1.9622	0.1639	0.1281	0.2253	0.2430
322	0.84	0.34	19	345	1.9642	0.1641	0.1281	0.2359	0.2572
323	0.84	0.5	19	345	1.9658	0.1643	0.1281	0.2434	0.2694
324	0.84	0.66	19	345	1.9679	0.1646	0.1281	0.2562	0.2837
325	0.84	0.84	19	345	1.9713	0.1649	0.1281	0.2723	0.3067

Table 47. Non-linear parameters in longitudinal building axis, considering variability in thresholds, for fixed materials – Hospital Dr. Luis Edmundo Vasquez

	$\varsigma y_{percentile}$	$\varsigma pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
301	0.16	0.16	19	345	1.3826	0.2081	0.0363	0.1003	0.1367
302	0.16	0.34	19	345	1.4131	0.2185	0.0363	0.1226	0.1653
303	0.16	0.5	19	345	1.4355	0.2245	0.0363	0.1448	0.1901
304	0.16	0.66	19	345	1.4518	0.2281	0.0363	0.1682	0.2110
305	0.16	0.84	19	345	1.4693	0.2314	0.0363	0.2030	0.2380
306	0.34	0.16	19	345	1.4431	0.1987	0.0435	0.1048	0.1346
307	0.34	0.34	19	345	1.4840	0.2147	0.0435	0.1223	0.1695
308	0.34	0.5	19	345	1.5087	0.2221	0.0435	0.1460	0.1955
309	0.34	0.66	19	345	1.4828	0.2143	0.0435	0.1690	0.1683
310	0.34	0.84	19	345	1.5496	0.2312	0.0435	0.2077	0.2491
311	0.5	0.16	19	345	1.4884	0.1901	0.0461	0.1083	0.1361
312	0.5	0.34	19	345	1.5390	0.2109	0.0461	0.1283	0.1717
313	0.5	0.5	19	345	1.5633	0.2188	0.0461	0.1436	0.1963
314	0.5	0.66	19	345	1.5868	0.2251	0.0461	0.1717	0.2237
315	0.5	0.84	19	345	1.6129	0.2308	0.0461	0.2089	0.2584
316	0.66	0.16	19	345	1.5402	0.1828	0.0517	0.1127	0.1416
317	0.66	0.34	19	345	1.5907	0.2030	0.0517	0.1300	0.1683
318	0.66	0.5	19	345	1.6239	0.2151	0.0517	0.1469	0.1991
319	0.66	0.66	19	345	1.6477	0.2221	0.0517	0.1685	0.2255
320	0.66	0.84	19	345	1.6806	0.2299	0.0517	0.2094	0.2674
321	0.84	0.16	19	345	1.6127	0.1710	0.0590	0.1204	0.1486
322	0.84	0.34	19	345	1.6686	0.1897	0.0590	0.1395	0.1726
323	0.84	0.5	19	345	1.7119	0.2060	0.0590	0.1523	0.1987
324	0.84	0.66	19	345	1.7442	0.2170	0.0590	0.1706	0.2311
325	0.84	0.84	19	345	1.7842	0.2276	0.0590	0.2119	0.2805

Table 48. Non-linear parameters in transversal building axis, considering variability in thresholds, for fixed materials – Hospital Dr. Luis Edmundo Vasquez

	$By_{percentile}$	$Bpl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
13	0.5	0.5	9.5	290	1.8790	0.1095	0.0803	0.1409	0.1512
38	0.5	0.5	14.25	290	1.7554	0.1384	0.0699	0.1682	0.1889
63	0.5	0.5	19	290	1.6980	0.1781	0.0796	0.2206	0.2473
88	0.5	0.5	23.75	290	1.6081	0.1585	0.0625	0.1948	0.2362
113	0.5	0.5	28.5	290	1.7840	0.1450	0.0596	0.1665	0.1981
138	0.5	0.5	9.5	317	1.9147	0.1165	0.0889	0.1502	0.1658
163	0.5	0.5	14.25	317	1.7840	0.1450	0.0784	0.1783	0.1981
188	0.5	0.5	19	317	1.6975	0.1576	0.0722	0.1924	0.2170
213	0.5	0.5	23.75	317	1.6479	0.1647	0.0670	0.2052	0.2315
238	0.5	0.5	28.5	317	1.5817	0.1696	0.0629	0.2042	0.2341
263	0.5	0.5	9.5	345	1.9492	0.1238	0.0957	0.1629	0.1785
288	0.5	0.5	14.25	345	1.8126	0.1522	0.0753	0.1894	0.2082
313	0.5	0.5	19	345	1.7243	0.1642	0.0809	0.2010	0.2249
338	0.5	0.5	23.75	345	1.6401	0.1611	0.0735	0.2095	0.2296
363	0.5	0.5	28.5	345	1.6077	0.1761	0.0713	0.2121	0.2422
388	0.5	0.5	9.5	373	1.9834	0.1307	0.1084	0.1743	0.1916
413	0.5	0.5	14.25	373	1.8431	0.1593	0.0949	0.1985	0.2191
438	0.5	0.5	19	373	1.7243	0.1642	0.0802	0.1994	0.2249
463	0.5	0.5	23.75	373	1.6980	0.1781	0.0796	0.2206	0.2473
488	0.5	0.5	28.5	373	1.6315	0.1825	0.0766	0.2182	0.2486
513	0.5	0.5	9.5	400	2.0144	0.1377	0.1156	0.1856	0.2037
538	0.5	0.5	14.25	400	1.8701	0.1662	0.1021	0.2104	0.2298
563	0.5	0.5	19	400	1.7760	0.1777	0.0948	0.2203	0.2436
588	0.5	0.5	23.75	400	1.7224	0.1846	0.0853	0.2302	0.2569
613	0.5	0.5	28.5	400	1.1008	0.1892	0.0491	0.1577	0.1829

Table 49. Non-linear parameters in longitudinal building axis, considering variability in materials, for fixed rotations – Hospital Dr. Luis Edmundo Vasquez

	$By_{percentile}$	$Bpl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
13	0.5	0.5	9.5	290	1.7556	0.1553	0.0519	0.1396	0.1827
38	0.5	0.5	14.25	290	1.6182	0.1866	0.0464	0.1372	0.1807
63	0.5	0.5	19	290	1.5328	0.2352	0.0490	0.1476	0.2041
88	0.5	0.5	23.75	290	1.4911	0.2114	0.0402	0.1409	0.1876
113	0.5	0.5	28.5	290	1.6270	0.1943	0.0383	0.1415	0.1846
138	0.5	0.5	9.5	317	1.7542	0.1610	0.0561	0.1453	0.1785
163	0.5	0.5	14.25	317	1.6270	0.1943	0.0466	0.1404	0.1846
188	0.5	0.5	19	317	1.5536	0.2104	0.0439	0.1396	0.1907
213	0.5	0.5	23.75	317	1.5153	0.2189	0.0418	0.1414	0.1948
238	0.5	0.5	28.5	317	1.4275	0.2174	0.0381	0.1232	0.1621
263	0.5	0.5	9.5	345	1.7203	0.1682	0.0700	0.1346	0.1672
288	0.5	0.5	14.25	345	1.6379	0.2030	0.0485	0.1434	0.1922
313	0.5	0.5	19	345	1.5633	0.2188	0.0461	0.1436	0.1963
338	0.5	0.5	23.75	345	1.5214	0.2222	0.0456	0.1491	0.1921
363	0.5	0.5	28.5	345	1.4696	0.2353	0.0421	0.1448	0.2005
388	0.5	0.5	9.5	373	1.7354	0.1729	0.0765	0.1360	0.1683
413	0.5	0.5	14.25	373	1.6512	0.2104	0.0548	0.1451	0.1965
438	0.5	0.5	19	373	1.5633	0.2188	0.0459	0.1430	0.1963
463	0.5	0.5	23.75	373	1.5328	0.2352	0.0490	0.1476	0.2041
488	0.5	0.5	28.5	373	1.4761	0.2427	0.0461	0.1476	0.2015
513	0.5	0.5	9.5	400	1.7505	0.1791	0.0815	0.1429	0.1739
538	0.5	0.5	14.25	400	1.6609	0.2170	0.0579	0.1507	0.1995
563	0.5	0.5	19	400	1.5829	0.2342	0.0516	0.1504	0.2051
588	0.5	0.5	23.75	400	1.5393	0.2420	0.0503	0.1504	0.2061
613	0.5	0.5	28.5	400	1.4774	0.2482	0.0475	0.1479	0.1983

Table 50. Non-linear parameters in transversal building axis, considering variability in materials, for fixed rotations – Hospital Dr. Luis Edmundo Vasquez

	$By_{percentile}$	$Bpl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
51	0.16	0.16	19	290	1.5091	0.1776	0.0506	0.1665	0.1861
52	0.16	0.34	19	290	1.5120	0.1779	0.0506	0.1783	0.2018
53	0.16	0.5	19	290	1.5150	0.1783	0.0506	0.1895	0.2176
54	0.16	0.66	19	290	1.5180	0.1786	0.0506	0.2029	0.2319
55	0.16	0.84	19	290	1.5221	0.1789	0.0506	0.2245	0.2540
56	0.34	0.16	19	290	1.6114	0.1774	0.0668	0.1836	0.2034
57	0.34	0.34	19	290	1.6140	0.1778	0.0668	0.1938	0.2184
58	0.34	0.5	19	290	1.6164	0.1781	0.0668	0.2058	0.2328
59	0.34	0.66	19	290	1.6195	0.1784	0.0668	0.2175	0.2493
60	0.34	0.84	19	290	1.6237	0.1788	0.0668	0.2398	0.2723
61	0.5	0.16	19	290	1.6933	0.1775	0.0796	0.1967	0.2180
62	0.5	0.34	19	290	1.6958	0.1778	0.0796	0.2100	0.2335
63	0.5	0.5	19	290	1.6980	0.1781	0.0796	0.2206	0.2473
64	0.5	0.66	19	290	1.7008	0.1784	0.0796	0.2321	0.2639
65	0.5	0.84	19	290	1.7050	0.1787	0.0796	0.2516	0.2880
66	0.66	0.16	19	290	1.7836	0.1773	0.0961	0.2131	0.2340
67	0.66	0.34	19	290	1.7860	0.1776	0.0961	0.2236	0.2499
68	0.66	0.5	19	290	1.7879	0.1778	0.0961	0.2351	0.2636
69	0.66	0.66	19	290	1.7906	0.1781	0.0961	0.2476	0.2803
70	0.66	0.84	19	290	1.7947	0.1785	0.0961	0.2699	0.3055
71	0.84	0.16	19	290	1.9287	0.1776	0.1310	0.2418	0.2642
72	0.84	0.34	19	290	1.9308	0.1779	0.1310	0.2530	0.2802
73	0.84	0.5	19	290	1.9326	0.1782	0.1310	0.2628	0.2940
74	0.84	0.66	19	290	1.9347	0.1784	0.1310	0.2746	0.3098
75	0.84	0.84	19	290	1.4365	0.1578	0.1310	0.2981	0.1677
76	0.16	0.16	23.75	290	1.4404	0.1581	0.0405	0.1486	0.1834
77	0.16	0.34	23.75	290	1.4427	0.1583	0.0405	0.1611	0.1975
78	0.16	0.5	23.75	290	1.4436	0.1584	0.0405	0.1711	0.2099
79	0.16	0.66	23.75	290	1.4444	0.1584	0.0405	0.1860	0.2305
80	0.16	0.84	23.75	290	1.5272	0.1578	0.0405	0.2034	0.1793
81	0.34	0.16	23.75	290	1.5310	0.1582	0.0512	0.1610	0.1959
82	0.34	0.34	23.75	290	1.5335	0.1584	0.0512	0.1737	0.2095
83	0.34	0.5	23.75	290	1.5342	0.1585	0.0512	0.1826	0.2226
84	0.34	0.66	23.75	290	1.5351	0.1585	0.0512	0.1956	0.2438
85	0.34	0.84	23.75	290	1.6009	0.1578	0.0512	0.2175	0.1910
86	0.5	0.16	23.75	290	1.6045	0.1581	0.0625	0.1714	0.2076
87	0.5	0.34	23.75	290	1.6072	0.1584	0.0625	0.1848	0.2215
88	0.5	0.5	23.75	290	1.6081	0.1585	0.0625	0.1948	0.2362
89	0.5	0.66	23.75	290	1.6088	0.1585	0.0625	0.2076	0.2568
90	0.5	0.84	23.75	290	1.6827	0.1580	0.0625	0.2275	0.2052
91	0.66	0.16	23.75	290	1.6859	0.1583	0.0735	0.1840	0.2209
92	0.66	0.34	23.75	290	1.6885	0.1585	0.0735	0.1965	0.2349
93	0.66	0.5	23.75	290	1.6897	0.1586	0.0735	0.2063	0.2507
94	0.66	0.66	23.75	290	1.6907	0.1587	0.0735	0.2199	0.2712
95	0.66	0.84	23.75	290	1.8126	0.1580	0.0735	0.2425	0.2270
96	0.84	0.16	23.75	290	1.8157	0.1584	0.1010	0.2049	0.2440
97	0.84	0.34	23.75	290	1.8181	0.1586	0.1010	0.2193	0.2582
98	0.84	0.5	23.75	290	1.8201	0.1588	0.1010	0.2291	0.2744
99	0.84	0.66	23.75	290	1.8207	0.1588	0.1010	0.2418	0.2972
100	0.84	0.84	23.75	290	1.6676	0.1594	0.1010	0.2636	0.3124

Table 51. Non-linear parameters in the longitudinal building axis (part b) – Hospital Dr. Luis Edmundo Vasquez

	$By_{percentile}$	$Bpl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
101	0.16	0.16	28.5	290	1.5795	0.1447	0.0366	0.1269	0.1473
102	0.16	0.34	28.5	290	1.5818	0.1450	0.0366	0.1401	0.1587
103	0.16	0.5	28.5	290	1.5841	0.1452	0.0366	0.1507	0.1685
104	0.16	0.66	28.5	290	1.5864	0.1455	0.0366	0.1610	0.1796
105	0.16	0.84	28.5	290	1.5892	0.1457	0.0366	0.1794	0.1949
106	0.34	0.16	28.5	290	1.6913	0.1448	0.0494	0.1367	0.1638
107	0.34	0.34	28.5	290	1.6934	0.1451	0.0494	0.1492	0.1750
108	0.34	0.5	28.5	290	1.6954	0.1454	0.0494	0.1593	0.1848
109	0.34	0.66	28.5	290	1.6977	0.1456	0.0494	0.1713	0.1964
110	0.34	0.84	28.5	290	1.7008	0.1459	0.0494	0.1875	0.2127
111	0.5	0.16	28.5	290	1.7801	0.1445	0.0596	0.1444	0.1767
112	0.5	0.34	28.5	290	1.7821	0.1448	0.0596	0.1559	0.1882
113	0.5	0.5	28.5	290	1.7840	0.1450	0.0596	0.1665	0.1981
114	0.5	0.66	28.5	290	1.7861	0.1452	0.0596	0.1781	0.2097
115	0.5	0.84	28.5	290	1.7895	0.1456	0.0596	0.1966	0.2266
116	0.66	0.16	28.5	290	1.8784	0.1447	0.0720	0.1562	0.1931
117	0.66	0.34	28.5	290	1.8802	0.1449	0.0720	0.1665	0.2041
118	0.66	0.5	28.5	290	1.8820	0.1451	0.0720	0.1772	0.2140
119	0.66	0.66	28.5	290	1.8839	0.1453	0.0720	0.1897	0.2252
120	0.66	0.84	28.5	290	1.8873	0.1456	0.0720	0.2067	0.2438
121	0.84	0.16	28.5	290	2.0332	0.1445	0.0988	0.1733	0.2197
122	0.84	0.34	28.5	290	2.0349	0.1447	0.0988	0.1850	0.2311
123	0.84	0.5	28.5	290	2.0365	0.1449	0.0988	0.1943	0.2408
124	0.84	0.66	28.5	290	2.0384	0.1451	0.0988	0.2054	0.2525
125	0.84	0.84	28.5	290	2.0414	0.1454	0.0988	0.2232	0.2714
126	0.16	0.16	9.5	317	1.6865	0.1163	0.0553	0.1136	0.1226
127	0.16	0.34	9.5	317	1.6877	0.1164	0.0553	0.1187	0.1299
128	0.16	0.5	9.5	317	1.6880	0.1165	0.0553	0.1239	0.1358
129	0.16	0.66	9.5	317	1.6882	0.1165	0.0553	0.1277	0.1393
130	0.16	0.84	9.5	317	1.6888	0.1166	0.0553	0.1386	0.1461
131	0.34	0.16	9.5	317	1.8135	0.1165	0.0734	0.1271	0.1410
132	0.34	0.34	9.5	317	1.8143	0.1166	0.0734	0.1344	0.1464
133	0.34	0.5	9.5	317	1.8147	0.1167	0.0734	0.1394	0.1521
134	0.34	0.66	9.5	317	1.8149	0.1167	0.0734	0.1450	0.1558
135	0.34	0.84	9.5	317	1.8154	0.1167	0.0734	0.1542	0.1626
136	0.5	0.16	9.5	317	1.9133	0.1163	0.0889	0.1405	0.1527
137	0.5	0.34	9.5	317	1.9144	0.1165	0.0889	0.1454	0.1599
138	0.5	0.5	9.5	317	1.9147	0.1165	0.0889	0.1502	0.1658
139	0.5	0.66	9.5	317	1.9149	0.1165	0.0889	0.1577	0.1693
140	0.5	0.84	9.5	317	1.9153	0.1166	0.0889	0.1660	0.1760
141	0.66	0.16	9.5	317	2.0231	0.1161	0.1089	0.1541	0.1689
142	0.66	0.34	9.5	317	2.0242	0.1163	0.1089	0.1609	0.1755
143	0.66	0.5	9.5	317	2.0247	0.1163	0.1089	0.1647	0.1814
144	0.66	0.66	9.5	317	2.0248	0.1163	0.1089	0.1703	0.1852
145	0.66	0.84	9.5	317	2.0252	0.1164	0.1089	0.1811	0.1916
146	0.84	0.16	9.5	317	2.1970	0.1161	0.1409	0.1779	0.1948
147	0.84	0.34	9.5	317	2.1980	0.1162	0.1409	0.1860	0.2020
148	0.84	0.5	9.5	317	2.1985	0.1163	0.1409	0.1907	0.2075
149	0.84	0.66	9.5	317	2.1986	0.1163	0.1409	0.1951	0.2111
150	0.84	0.84	9.5	317	2.1989	0.1163	0.1409	0.2046	0.2174

Table 52. Non-linear parameters in the longitudinal building axis (part c) – Hospital Dr. Luis Edmundo Vasquez

	$\varsigma y_{percentile}$	$\varsigma pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
151	0.16	0.16	14.25	317	1.5795	0.1447	0.0497	0.1331	0.1473
152	0.16	0.34	14.25	317	1.5818	0.1450	0.0497	0.1423	0.1587
153	0.16	0.5	14.25	317	1.5841	0.1452	0.0497	0.1502	0.1685
154	0.16	0.66	14.25	317	1.5864	0.1455	0.0497	0.1573	0.1796
155	0.16	0.84	14.25	317	1.5892	0.1457	0.0497	0.1759	0.1949
156	0.34	0.16	14.25	317	1.6913	0.1448	0.0622	0.1494	0.1638
157	0.34	0.34	14.25	317	1.6934	0.1451	0.0622	0.1581	0.1750
158	0.34	0.5	14.25	317	1.6954	0.1454	0.0622	0.1656	0.1848
159	0.34	0.66	14.25	317	1.6977	0.1456	0.0622	0.1744	0.1964
160	0.34	0.84	14.25	317	1.7008	0.1459	0.0622	0.1896	0.2127
161	0.5	0.16	14.25	317	1.7801	0.1445	0.0784	0.1628	0.1767
162	0.5	0.34	14.25	317	1.7821	0.1448	0.0784	0.1712	0.1882
163	0.5	0.5	14.25	317	1.7840	0.1450	0.0784	0.1783	0.1981
164	0.5	0.66	14.25	317	1.7861	0.1452	0.0784	0.1857	0.2097
165	0.5	0.84	14.25	317	1.7895	0.1456	0.0784	0.2023	0.2266
166	0.66	0.16	14.25	317	1.8784	0.1447	0.0951	0.1760	0.1931
167	0.66	0.34	14.25	317	1.8802	0.1449	0.0951	0.1860	0.2041
168	0.66	0.5	14.25	317	1.8820	0.1451	0.0951	0.1934	0.2140
169	0.66	0.66	14.25	317	1.8839	0.1453	0.0951	0.2003	0.2252
170	0.66	0.84	14.25	317	1.8873	0.1456	0.0951	0.2175	0.2438
171	0.84	0.16	14.25	317	2.0332	0.1445	0.1252	0.2022	0.2197
172	0.84	0.34	14.25	317	2.0349	0.1447	0.1252	0.2109	0.2311
173	0.84	0.5	14.25	317	2.0365	0.1449	0.1252	0.2173	0.2408
174	0.84	0.66	14.25	317	2.0384	0.1451	0.1252	0.2274	0.2525
175	0.84	0.84	14.25	317	2.0414	0.1454	0.1252	0.2426	0.2714
176	0.16	0.16	19	317	1.5080	0.1569	0.0445	0.1444	0.1622
177	0.16	0.34	19	317	1.5112	0.1572	0.0445	0.1553	0.1766
178	0.16	0.5	19	317	1.5137	0.1575	0.0445	0.1646	0.1891
179	0.16	0.66	19	317	1.5164	0.1577	0.0445	0.1774	0.2005
180	0.16	0.84	19	317	1.5178	0.1578	0.0445	0.1955	0.2195
181	0.34	0.16	19	317	1.6102	0.1570	0.0586	0.1601	0.1776
182	0.34	0.34	19	317	1.6130	0.1573	0.0586	0.1710	0.1913
183	0.34	0.5	19	317	1.6153	0.1575	0.0586	0.1804	0.2035
184	0.34	0.66	19	317	1.6183	0.1578	0.0586	0.1915	0.2179
185	0.34	0.84	19	317	1.6198	0.1580	0.0586	0.2088	0.2366
186	0.5	0.16	19	317	1.6926	0.1571	0.0722	0.1737	0.1903
187	0.5	0.34	19	317	1.6954	0.1574	0.0722	0.1823	0.2053
188	0.5	0.5	19	317	1.6975	0.1576	0.0722	0.1924	0.2170
189	0.5	0.66	19	317	1.7004	0.1579	0.0722	0.2046	0.2315
190	0.5	0.84	19	317	1.7024	0.1581	0.0722	0.2209	0.2509
191	0.66	0.16	19	317	1.7828	0.1569	0.0889	0.1860	0.2054
192	0.66	0.34	19	317	1.7852	0.1572	0.0889	0.1967	0.2197
193	0.66	0.5	19	317	1.7872	0.1574	0.0889	0.2071	0.2315
194	0.66	0.66	19	317	1.7898	0.1576	0.0889	0.2181	0.2459
195	0.66	0.84	19	317	1.7924	0.1579	0.0889	0.2363	0.2669
196	0.84	0.16	19	317	1.9258	0.1570	0.1172	0.2115	0.2309
197	0.84	0.34	19	317	1.9280	0.1573	0.1172	0.2222	0.2451
198	0.84	0.5	19	317	1.9299	0.1575	0.1172	0.2307	0.2573
199	0.84	0.66	19	317	1.9321	0.1578	0.1172	0.2425	0.2714
200	0.84	0.84	19	317	1.9350	0.1580	0.1172	0.2591	0.2941

Table 53. Non-linear parameters in the longitudinal building axis (part d) – Hospital Dr. Luis Edmundo Vasquez

	$By_{percentile}$	$Bpl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
201	0.16	0.16	23.75	317	1.4720	0.1642	0.0425	0.1564	0.1747
202	0.16	0.34	23.75	317	1.4757	0.1646	0.0425	0.1691	0.1913
203	0.16	0.5	23.75	317	1.4787	0.1649	0.0425	0.1802	0.2056
204	0.16	0.66	23.75	317	1.4816	0.1651	0.0425	0.1914	0.2203
205	0.16	0.84	23.75	317	1.4827	0.1652	0.0425	0.2131	0.2412
206	0.34	0.16	23.75	317	1.5666	0.1642	0.0558	0.1698	0.1899
207	0.34	0.34	23.75	317	1.5700	0.1645	0.0558	0.1822	0.2053
208	0.34	0.5	23.75	317	1.5726	0.1648	0.0558	0.1916	0.2190
209	0.34	0.66	23.75	317	1.5760	0.1651	0.0558	0.2069	0.2353
210	0.34	0.84	23.75	317	1.5769	0.1651	0.0558	0.2258	0.2559
211	0.5	0.16	23.75	317	1.6423	0.1641	0.0670	0.1801	0.2019
212	0.5	0.34	23.75	317	1.6455	0.1645	0.0670	0.1927	0.2183
213	0.5	0.5	23.75	317	1.6479	0.1647	0.0670	0.2052	0.2315
214	0.5	0.66	23.75	317	1.6512	0.1650	0.0670	0.2158	0.2480
215	0.5	0.84	23.75	317	1.6524	0.1651	0.0670	0.2367	0.2694
216	0.66	0.16	23.75	317	1.7272	0.1642	0.0795	0.1939	0.2165
217	0.66	0.34	23.75	317	1.7302	0.1646	0.0795	0.2072	0.2329
218	0.66	0.5	23.75	317	1.7324	0.1648	0.0795	0.2191	0.2457
219	0.66	0.66	23.75	317	1.7355	0.1651	0.0795	0.2317	0.2624
220	0.66	0.84	23.75	317	1.7372	0.1652	0.0795	0.2501	0.2857
221	0.84	0.16	23.75	317	1.9093	0.1640	0.1129	0.2214	0.2440
222	0.84	0.34	23.75	317	1.8639	0.1646	0.1129	0.2307	0.2564
223	0.84	0.5	23.75	317	1.8661	0.1648	0.1129	0.2413	0.2702
224	0.84	0.66	23.75	317	1.8688	0.1651	0.1129	0.2525	0.2867
225	0.84	0.84	23.75	317	1.8711	0.1653	0.1129	0.2750	0.3121
226	0.16	0.16	28.5	317	1.4146	0.1693	0.0400	0.1588	0.1789
227	0.16	0.34	28.5	317	1.4180	0.1697	0.0400	0.1729	0.1960
228	0.16	0.5	28.5	317	1.4209	0.1699	0.0400	0.1837	0.2115
229	0.16	0.66	28.5	317	1.4232	0.1701	0.0400	0.1960	0.2259
230	0.16	0.84	28.5	317	1.4236	0.1701	0.0400	0.2178	0.2474
231	0.34	0.16	28.5	317	1.5042	0.1691	0.0515	0.1721	0.1908
232	0.34	0.34	28.5	317	1.5075	0.1694	0.0515	0.1855	0.2086
233	0.34	0.5	28.5	317	1.5102	0.1696	0.0515	0.1940	0.2234
234	0.34	0.66	28.5	317	1.5130	0.1699	0.0515	0.2075	0.2394
235	0.34	0.84	28.5	317	1.5135	0.1699	0.0515	0.2305	0.2611
236	0.5	0.16	28.5	317	1.5761	0.1690	0.0629	0.1813	0.2016
237	0.5	0.34	28.5	317	1.5790	0.1693	0.0629	0.1957	0.2192
238	0.5	0.5	28.5	317	1.5817	0.1696	0.0629	0.2042	0.2341
239	0.5	0.66	28.5	317	1.5847	0.1698	0.0629	0.2178	0.2506
240	0.5	0.84	28.5	317	1.5849	0.1699	0.0629	0.2409	0.2727
241	0.66	0.16	28.5	317	1.6567	0.1689	0.0780	0.1934	0.2157
242	0.66	0.34	28.5	317	1.6594	0.1692	0.0780	0.2082	0.2329
243	0.66	0.5	28.5	317	1.6619	0.1694	0.0780	0.2172	0.2477
244	0.66	0.66	28.5	317	1.6649	0.1697	0.0780	0.2307	0.2654
245	0.66	0.84	28.5	317	1.6657	0.1698	0.0780	0.2563	0.2879
246	0.84	0.16	28.5	317	1.7840	0.1689	0.1048	0.2161	0.2384
247	0.84	0.34	28.5	317	1.7864	0.1691	0.1048	0.2290	0.2555
248	0.84	0.5	28.5	317	1.7885	0.1693	0.1048	0.2383	0.2698
249	0.84	0.66	28.5	317	1.7914	0.1696	0.1048	0.2519	0.2881
250	0.84	0.84	28.5	317	1.7931	0.1697	0.1048	0.2762	0.3122

Table 54. Non-linear parameters in the longitudinal building axis (part e) – Hospital Dr. Luis Edmundo Vasquez

	$By_{percentile}$	$Bpl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
251	0.16	0.16	9.5	345	1.7140	0.1236	0.0594	0.1223	0.1325
252	0.16	0.34	9.5	345	1.7155	0.1238	0.0594	0.1265	0.1397
253	0.16	0.5	9.5	345	1.7162	0.1239	0.0594	0.1326	0.1460
254	0.16	0.66	9.5	345	1.7169	0.1240	0.0594	0.1371	0.1518
255	0.16	0.84	9.5	345	1.7173	0.1240	0.0594	0.1467	0.1582
256	0.34	0.16	9.5	345	1.8443	0.1236	0.0805	0.1375	0.1507
257	0.34	0.34	9.5	345	1.8455	0.1238	0.0805	0.1445	0.1575
258	0.34	0.5	9.5	345	1.8464	0.1239	0.0805	0.1496	0.1637
259	0.34	0.66	9.5	345	1.8469	0.1239	0.0805	0.1551	0.1697
260	0.34	0.84	9.5	345	1.8473	0.1240	0.0805	0.1653	0.1762
261	0.5	0.16	9.5	345	1.9471	0.1235	0.0957	0.1522	0.1649
262	0.5	0.34	9.5	345	1.9483	0.1237	0.0957	0.1583	0.1724
263	0.5	0.5	9.5	345	1.9492	0.1238	0.0957	0.1629	0.1785
264	0.5	0.66	9.5	345	1.9497	0.1238	0.0957	0.1692	0.1845
265	0.5	0.84	9.5	345	1.9502	0.1239	0.0957	0.1772	0.1912
266	0.66	0.16	9.5	345	2.0610	0.1236	0.1201	0.1678	0.1820
267	0.66	0.34	9.5	345	2.0621	0.1238	0.1201	0.1743	0.1900
268	0.66	0.5	9.5	345	2.0632	0.1239	0.1201	0.1788	0.1974
269	0.66	0.66	9.5	345	2.0636	0.1240	0.1201	0.1853	0.2028
270	0.66	0.84	9.5	345	2.0639	0.1240	0.1201	0.1959	0.2091
271	0.84	0.16	9.5	345	2.2401	0.1237	0.1576	0.1928	0.2110
272	0.84	0.34	9.5	345	2.2413	0.1239	0.1576	0.2007	0.2193
273	0.84	0.5	9.5	345	2.2423	0.1240	0.1576	0.2059	0.2263
274	0.84	0.66	9.5	345	2.2431	0.1241	0.1576	0.2140	0.2334
275	0.84	0.84	9.5	345	2.2435	0.1241	0.1576	0.2237	0.2407
276	0.16	0.16	14.25	345	1.6012	0.1519	0.0499	0.1425	0.1550
277	0.16	0.34	14.25	345	1.6033	0.1522	0.0499	0.1501	0.1660
278	0.16	0.5	14.25	345	1.6054	0.1524	0.0499	0.1588	0.1763
279	0.16	0.66	14.25	345	1.6076	0.1527	0.0499	0.1677	0.1878
280	0.16	0.84	14.25	345	1.6112	0.1530	0.0499	0.1829	0.2038
281	0.34	0.16	14.25	345	1.7165	0.1518	0.0644	0.1589	0.1722
282	0.34	0.34	14.25	345	1.7184	0.1521	0.0644	0.1682	0.1836
283	0.34	0.5	14.25	345	1.7203	0.1523	0.0644	0.1752	0.1934
284	0.34	0.66	14.25	345	1.7222	0.1525	0.0644	0.1826	0.2047
285	0.34	0.84	14.25	345	1.7256	0.1529	0.0644	0.1994	0.2218
286	0.5	0.16	14.25	345	1.8091	0.1517	0.0753	0.1732	0.1869
287	0.5	0.34	14.25	345	1.8109	0.1520	0.0753	0.1819	0.1983
288	0.5	0.5	14.25	345	1.8126	0.1522	0.0753	0.1894	0.2082
289	0.5	0.66	14.25	345	1.8145	0.1524	0.0753	0.1989	0.2198
290	0.5	0.84	14.25	345	1.8177	0.1527	0.0753	0.2120	0.2375
291	0.66	0.16	14.25	345	1.9106	0.1516	0.0917	0.1886	0.2043
292	0.66	0.34	14.25	345	1.9122	0.1518	0.0917	0.1972	0.2157
293	0.66	0.5	14.25	345	1.9138	0.1521	0.0917	0.2058	0.2255
294	0.66	0.66	14.25	345	1.9156	0.1523	0.0917	0.2135	0.2371
295	0.66	0.84	14.25	345	1.9246	0.1526	0.0917	0.2283	0.2559
296	0.84	0.16	14.25	345	2.0719	0.1518	0.1254	0.2179	0.2343
297	0.84	0.34	14.25	345	2.0734	0.1520	0.1254	0.2268	0.2456
298	0.84	0.5	14.25	345	2.0748	0.1522	0.1254	0.2343	0.2554
299	0.84	0.66	14.25	345	2.0764	0.1524	0.1254	0.2432	0.2670
300	0.84	0.84	14.25	345	2.0792	0.1527	0.1254	0.2585	0.2868

Table 55. Non-linear parameters in the longitudinal building axis (part f) – Hospital Dr. Luis Edmundo Vasquez

	$By_{percentile}$	$Bpl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
301	0.16	0.16	19	345	1.5294	0.1639	0.0500	0.1528	0.1689
302	0.16	0.34	19	345	1.5323	0.1643	0.0500	0.1653	0.1835
303	0.16	0.5	19	345	1.5347	0.1646	0.0500	0.1728	0.1962
304	0.16	0.66	19	345	1.5376	0.1649	0.0500	0.1842	0.2091
305	0.16	0.84	19	345	1.5409	0.1652	0.0500	0.2033	0.2284
306	0.34	0.16	19	345	1.6348	0.1636	0.0649	0.1680	0.1848
307	0.34	0.34	19	345	1.6372	0.1640	0.0649	0.1779	0.1982
308	0.34	0.5	19	345	1.6394	0.1642	0.0649	0.1889	0.2108
309	0.34	0.66	19	345	1.6423	0.1645	0.0649	0.2001	0.2254
310	0.34	0.84	19	345	1.6457	0.1648	0.0649	0.2172	0.2442
311	0.5	0.16	19	345	1.7200	0.1636	0.0809	0.1825	0.1992
312	0.5	0.34	19	345	1.7224	0.1639	0.0809	0.1925	0.2129
313	0.5	0.5	19	345	1.7243	0.1642	0.0809	0.2010	0.2249
314	0.5	0.66	19	345	1.7270	0.1645	0.0809	0.2138	0.2397
315	0.5	0.84	19	345	1.7307	0.1648	0.0809	0.2328	0.2604
316	0.66	0.16	19	345	1.8138	0.1639	0.0971	0.1968	0.2152
317	0.66	0.34	19	345	1.8160	0.1642	0.0971	0.2090	0.2294
318	0.66	0.5	19	345	1.8178	0.1644	0.0971	0.2178	0.2415
319	0.66	0.66	19	345	1.8202	0.1647	0.0971	0.2275	0.2555
320	0.66	0.84	19	345	1.8238	0.1650	0.0971	0.2473	0.2781
321	0.84	0.16	19	345	1.9622	0.1639	0.1281	0.2253	0.2430
322	0.84	0.34	19	345	1.9642	0.1641	0.1281	0.2359	0.2572
323	0.84	0.5	19	345	1.9658	0.1643	0.1281	0.2434	0.2694
324	0.84	0.66	19	345	1.9679	0.1646	0.1281	0.2562	0.2837
325	0.84	0.84	19	345	1.9713	0.1649	0.1281	0.2723	0.3067
326	0.16	0.16	23.75	345	1.4821	0.1667	0.0467	0.1592	0.1725
327	0.16	0.34	23.75	345	1.4821	0.1667	0.0467	0.1723	0.1874
328	0.16	0.5	23.75	345	1.4863	0.1673	0.0467	0.1801	0.2004
329	0.16	0.66	23.75	345	1.4887	0.1675	0.0467	0.1919	0.2135
330	0.16	0.84	23.75	345	1.4917	0.1678	0.0467	0.2118	0.2332
331	0.34	0.16	23.75	345	1.5599	0.1606	0.0617	0.1750	0.1887
332	0.34	0.34	23.75	345	1.5599	0.1606	0.0617	0.1854	0.2024
333	0.34	0.5	23.75	345	1.5599	0.1606	0.0617	0.1968	0.2153
334	0.34	0.66	23.75	345	1.5599	0.1606	0.0617	0.2085	0.2301
335	0.34	0.84	23.75	345	1.5599	0.1606	0.0617	0.2264	0.2494
336	0.5	0.16	23.75	345	1.6401	0.1610	0.0735	0.1902	0.2034
337	0.5	0.34	23.75	345	1.6401	0.1611	0.0735	0.2006	0.2174
338	0.5	0.5	23.75	345	1.6401	0.1611	0.0735	0.2095	0.2296
339	0.5	0.66	23.75	345	1.6401	0.1610	0.0735	0.2228	0.2448
340	0.5	0.84	23.75	345	1.6401	0.1610	0.0735	0.2426	0.2660
341	0.66	0.16	23.75	345	1.7268	0.1612	0.0896	0.2051	0.2197
342	0.66	0.34	23.75	345	1.7268	0.1612	0.0896	0.2178	0.2343
343	0.66	0.5	23.75	345	1.7268	0.1612	0.0896	0.2270	0.2467
344	0.66	0.66	23.75	345	1.7268	0.1612	0.0896	0.2371	0.2609
345	0.66	0.84	23.75	345	1.7268	0.1612	0.0896	0.2577	0.2840
346	0.84	0.16	23.75	345	1.8662	0.1616	0.1205	0.2348	0.2482
347	0.84	0.34	23.75	345	1.8662	0.1616	0.1205	0.2458	0.2626
348	0.84	0.5	23.75	345	1.8662	0.1616	0.1205	0.2536	0.2751
349	0.84	0.66	23.75	345	1.8662	0.1616	0.1205	0.2670	0.2897
350	0.84	0.84	23.75	345	1.8662	0.1616	0.1205	0.2838	0.3132

Table 56. Non-linear parameters in the longitudinal building axis (part g) – Hospital Dr. Luis Edmundo Vasquez

	$By_{percentile}$	$Bpl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
351	0.16	0.16	28.5	345	1.4322	0.1756	0.0442	0.1617	0.1816
352	0.16	0.34	28.5	345	1.4358	0.1760	0.0442	0.1765	0.1993
353	0.16	0.5	28.5	345	1.4390	0.1763	0.0442	0.1871	0.2146
354	0.16	0.66	28.5	345	1.4417	0.1765	0.0442	0.1994	0.2293
355	0.16	0.84	28.5	345	1.4431	0.1767	0.0442	0.2207	0.2515
356	0.34	0.16	28.5	345	1.5260	0.1753	0.0572	0.1760	0.1969
357	0.34	0.34	28.5	345	1.5292	0.1757	0.0572	0.1880	0.2137
358	0.34	0.5	28.5	345	1.5321	0.1760	0.0572	0.1989	0.2283
359	0.34	0.66	28.5	345	1.5350	0.1762	0.0572	0.2139	0.2436
360	0.34	0.84	28.5	345	1.5365	0.1763	0.0572	0.2351	0.2661
361	0.5	0.16	28.5	345	1.6020	0.1755	0.0713	0.1872	0.2104
362	0.5	0.34	28.5	345	1.6050	0.1758	0.0713	0.2021	0.2271
363	0.5	0.5	28.5	345	1.6077	0.1761	0.0713	0.2121	0.2422
364	0.5	0.66	28.5	345	1.6107	0.1764	0.0713	0.2255	0.2589
365	0.5	0.84	28.5	345	1.6124	0.1765	0.0713	0.2486	0.2815
366	0.66	0.16	28.5	345	1.6853	0.1753	0.0872	0.2026	0.2236
367	0.66	0.34	28.5	345	1.6882	0.1757	0.0872	0.2147	0.2413
368	0.66	0.5	28.5	345	1.6908	0.1760	0.0872	0.2261	0.2563
369	0.66	0.66	28.5	345	1.6937	0.1762	0.0872	0.2381	0.2733
370	0.66	0.84	28.5	345	1.6958	0.1764	0.0872	0.2637	0.2973
371	0.84	0.16	28.5	345	1.8186	0.1754	0.1161	0.2255	0.2501
372	0.84	0.34	28.5	345	1.8212	0.1757	0.1161	0.2407	0.2676
373	0.84	0.5	28.5	345	1.8235	0.1760	0.1161	0.2521	0.2824
374	0.84	0.66	28.5	345	1.8261	0.1763	0.1161	0.2654	0.3001
375	0.84	0.84	28.5	345	1.8288	0.1765	0.1161	0.2856	0.3249
376	0.16	0.16	9.5	373	1.7401	0.1305	0.0667	0.1297	0.1420
377	0.16	0.34	9.5	373	1.7414	0.1307	0.0667	0.1353	0.1504
378	0.16	0.5	9.5	373	1.7424	0.1309	0.0667	0.1402	0.1557
379	0.16	0.66	9.5	373	1.7434	0.1310	0.0667	0.1474	0.1626
380	0.16	0.84	9.5	373	1.7438	0.1310	0.0667	0.1571	0.1699
381	0.34	0.16	9.5	373	1.8748	0.1306	0.0877	0.1487	0.1608
382	0.34	0.34	9.5	373	1.8759	0.1307	0.0877	0.1543	0.1686
383	0.34	0.5	9.5	373	1.8769	0.1309	0.0877	0.1600	0.1746
384	0.34	0.66	9.5	373	1.8778	0.1310	0.0877	0.1638	0.1818
385	0.34	0.84	9.5	373	1.8783	0.1310	0.0877	0.1756	0.1892
386	0.5	0.16	9.5	373	1.9813	0.1304	0.1084	0.1622	0.1768
387	0.5	0.34	9.5	373	1.9824	0.1306	0.1084	0.1693	0.1847
388	0.5	0.5	9.5	373	1.9834	0.1307	0.1084	0.1743	0.1916
389	0.5	0.66	9.5	373	1.9843	0.1309	0.1084	0.1803	0.1982
390	0.5	0.84	9.5	373	1.9846	0.1309	0.1084	0.1890	0.2051
391	0.66	0.16	9.5	373	2.0986	0.1305	0.1310	0.1797	0.1952
392	0.66	0.34	9.5	373	2.0996	0.1307	0.1310	0.1860	0.2036
393	0.66	0.5	9.5	373	2.1006	0.1308	0.1310	0.1909	0.2104
394	0.66	0.66	9.5	373	2.1017	0.1310	0.1310	0.1992	0.2176
395	0.66	0.84	9.5	373	2.1022	0.1310	0.1310	0.2087	0.2256
396	0.84	0.16	9.5	373	2.2826	0.1303	0.1689	0.2076	0.2262
397	0.84	0.34	9.5	373	2.2836	0.1304	0.1689	0.2156	0.2346
398	0.84	0.5	9.5	373	2.2846	0.1306	0.1689	0.2204	0.2418
399	0.84	0.66	9.5	373	2.2857	0.1307	0.1689	0.2293	0.2496
400	0.84	0.84	9.5	373	2.2862	0.1308	0.1689	0.2390	0.2575

Table 57. Non-linear parameters in the longitudinal building axis (part h) – Hospital Dr. Luis Edmundo Vasquez

	$\Xi y_{percentile}$	$\Xi pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
401	0.16	0.16	14.25	373	1.6256	0.1589	0.0614	0.1479	0.1628
402	0.16	0.34	14.25	373	1.6275	0.1592	0.0614	0.1578	0.1742
403	0.16	0.5	14.25	373	1.6293	0.1594	0.0614	0.1637	0.1836
404	0.16	0.66	14.25	373	1.6313	0.1597	0.0614	0.1724	0.1954
405	0.16	0.84	14.25	373	1.6345	0.1600	0.0614	0.1875	0.2117
406	0.34	0.16	14.25	373	1.7449	0.1590	0.0789	0.1661	0.1820
407	0.34	0.34	14.25	373	1.7466	0.1592	0.0789	0.1747	0.1934
408	0.34	0.5	14.25	373	1.7483	0.1595	0.0789	0.1833	0.2033
409	0.34	0.66	14.25	373	1.7500	0.1597	0.0789	0.1907	0.2147
410	0.34	0.84	14.25	373	1.7531	0.1600	0.0789	0.2058	0.2321
411	0.5	0.16	14.25	373	1.8400	0.1588	0.0949	0.1811	0.1979
412	0.5	0.34	14.25	373	1.8416	0.1591	0.0949	0.1910	0.2093
413	0.5	0.5	14.25	373	1.8431	0.1593	0.0949	0.1985	0.2191
414	0.5	0.66	14.25	373	1.8447	0.1595	0.0949	0.2055	0.2304
415	0.5	0.84	14.25	373	1.8477	0.1598	0.0949	0.2211	0.2489
416	0.66	0.16	14.25	373	1.9446	0.1587	0.1163	0.1992	0.2166
417	0.66	0.34	14.25	373	1.9461	0.1590	0.1163	0.2073	0.2280
418	0.66	0.5	14.25	373	1.9475	0.1592	0.1163	0.2164	0.2379
419	0.66	0.66	14.25	373	1.9491	0.1594	0.1163	0.2252	0.2494
420	0.66	0.84	14.25	373	1.9518	0.1597	0.1163	0.2379	0.2685
421	0.84	0.16	14.25	373	2.1102	0.1589	0.1568	0.2291	0.2492
422	0.84	0.34	14.25	373	2.1115	0.1591	0.1568	0.2397	0.2606
423	0.84	0.5	14.25	373	2.1128	0.1593	0.1568	0.2471	0.2704
424	0.84	0.66	14.25	373	2.1142	0.1595	0.1568	0.2559	0.2820
425	0.84	0.84	14.25	373	2.1167	0.1598	0.1568	0.2690	0.3017
426	0.16	0.16	19	373	1.5294	0.1639	0.0496	0.1516	0.1689
427	0.16	0.34	19	373	1.5323	0.1643	0.0496	0.1640	0.1835
428	0.16	0.5	19	373	1.5347	0.1646	0.0496	0.1714	0.1962
429	0.16	0.66	19	373	1.5376	0.1649	0.0496	0.1827	0.2091
430	0.16	0.84	19	373	1.5409	0.1652	0.0496	0.2016	0.2284
431	0.34	0.16	19	373	1.6348	0.1636	0.0644	0.1666	0.1848
432	0.34	0.34	19	373	1.6372	0.1640	0.0644	0.1764	0.1982
433	0.34	0.5	19	373	1.6394	0.1642	0.0644	0.1874	0.2108
434	0.34	0.66	19	373	1.6423	0.1645	0.0644	0.1984	0.2254
435	0.34	0.84	19	373	1.6457	0.1648	0.0644	0.2155	0.2442
436	0.5	0.16	19	373	1.7200	0.1636	0.0802	0.1810	0.1992
437	0.5	0.34	19	373	1.7224	0.1639	0.0802	0.1909	0.2129
438	0.5	0.5	19	373	1.7243	0.1642	0.0802	0.1994	0.2249
439	0.5	0.66	19	373	1.7270	0.1645	0.0802	0.2120	0.2397
440	0.5	0.84	19	373	1.7307	0.1648	0.0802	0.2309	0.2604
441	0.66	0.16	19	373	1.8138	0.1639	0.0963	0.1952	0.2152
442	0.66	0.34	19	373	1.8160	0.1642	0.0963	0.2073	0.2294
443	0.66	0.5	19	373	1.8178	0.1644	0.0963	0.2161	0.2415
444	0.66	0.66	19	373	1.8202	0.1647	0.0963	0.2257	0.2555
445	0.66	0.84	19	373	1.8238	0.1650	0.0963	0.2453	0.2781
446	0.84	0.16	19	373	1.9622	0.1639	0.1270	0.2234	0.2430
447	0.84	0.34	19	373	1.9642	0.1641	0.1270	0.2340	0.2572
448	0.84	0.5	19	373	1.9658	0.1643	0.1270	0.2414	0.2694
449	0.84	0.66	19	373	1.9679	0.1646	0.1270	0.2541	0.2837
450	0.84	0.84	19	373	1.9713	0.1649	0.1270	0.2701	0.3067

Table 58. Non-linear parameters in the longitudinal building axis (part i) – Hospital Dr. Luis Edmundo Vasquez

	$\Xi y_{percentile}$	$\Xi pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
451	0.16	0.16	23.75	373	1.5091	0.1776	0.0506	0.1665	0.1861
452	0.16	0.34	23.75	373	1.5120	0.1779	0.0506	0.1783	0.2018
453	0.16	0.5	23.75	373	1.5150	0.1783	0.0506	0.1895	0.2176
454	0.16	0.66	23.75	373	1.5180	0.1786	0.0506	0.2029	0.2319
455	0.16	0.84	23.75	373	1.5221	0.1789	0.0506	0.2245	0.2540
456	0.34	0.16	23.75	373	1.6114	0.1774	0.0668	0.1836	0.2034
457	0.34	0.34	23.75	373	1.6140	0.1778	0.0668	0.1938	0.2184
458	0.34	0.5	23.75	373	1.6164	0.1781	0.0668	0.2058	0.2328
459	0.34	0.66	23.75	373	1.6195	0.1784	0.0668	0.2175	0.2493
460	0.34	0.84	23.75	373	1.6237	0.1788	0.0668	0.2398	0.2723
461	0.5	0.16	23.75	373	1.6933	0.1775	0.0796	0.1967	0.2180
462	0.5	0.34	23.75	373	1.6958	0.1778	0.0796	0.2100	0.2335
463	0.5	0.5	23.75	373	1.6980	0.1781	0.0796	0.2206	0.2473
464	0.5	0.66	23.75	373	1.7008	0.1784	0.0796	0.2321	0.2639
465	0.5	0.84	23.75	373	1.7050	0.1787	0.0796	0.2516	0.2880
466	0.66	0.16	23.75	373	1.7836	0.1773	0.0961	0.2131	0.2340
467	0.66	0.34	23.75	373	1.7860	0.1776	0.0961	0.2236	0.2499
468	0.66	0.5	23.75	373	1.7879	0.1778	0.0961	0.2351	0.2636
469	0.66	0.66	23.75	373	1.7905	0.1781	0.0961	0.2476	0.2802
470	0.66	0.84	23.75	373	1.7947	0.1785	0.0961	0.2698	0.3055
471	0.84	0.16	23.75	373	1.9287	0.1776	0.1310	0.2418	0.2642
472	0.84	0.34	23.75	373	1.9308	0.1779	0.1310	0.2530	0.2802
473	0.84	0.5	23.75	373	1.9326	0.1782	0.1310	0.2628	0.2940
474	0.84	0.66	23.75	373	1.9347	0.1784	0.1310	0.2746	0.3098
475	0.84	0.84	23.75	373	1.9375	0.1787	0.1310	0.2891	0.3289
476	0.16	0.16	28.5	373	1.4493	0.1818	0.0486	0.1647	0.1864
477	0.16	0.34	28.5	373	1.4526	0.1822	0.0486	0.1800	0.2040
478	0.16	0.5	28.5	373	1.4556	0.1825	0.0486	0.1914	0.2179
479	0.16	0.66	28.5	373	1.4583	0.1827	0.0486	0.2039	0.2325
480	0.16	0.84	28.5	373	1.4615	0.1830	0.0486	0.2260	0.2559
481	0.34	0.16	28.5	373	1.5477	0.1818	0.0626	0.1821	0.2033
482	0.34	0.34	28.5	373	1.5505	0.1821	0.0626	0.1956	0.2196
483	0.34	0.5	28.5	373	1.5532	0.1824	0.0626	0.2046	0.2341
484	0.34	0.66	28.5	373	1.5562	0.1827	0.0626	0.2198	0.2509
485	0.34	0.84	28.5	373	1.5598	0.1830	0.0626	0.2425	0.2747
486	0.5	0.16	28.5	373	1.6262	0.1819	0.0766	0.1938	0.2173
487	0.5	0.34	28.5	373	1.6289	0.1822	0.0766	0.2070	0.2338
488	0.5	0.5	28.5	373	1.6315	0.1825	0.0766	0.2182	0.2486
489	0.5	0.66	28.5	373	1.6343	0.1828	0.0766	0.2329	0.2655
490	0.5	0.84	28.5	373	1.6382	0.1831	0.0766	0.2559	0.2898
491	0.66	0.16	28.5	373	1.7129	0.1819	0.0919	0.2106	0.2333
492	0.66	0.34	28.5	373	1.7155	0.1823	0.0919	0.2230	0.2499
493	0.66	0.5	28.5	373	1.7178	0.1825	0.0919	0.2354	0.2648
494	0.66	0.66	28.5	373	1.7206	0.1828	0.0919	0.2470	0.2825
495	0.66	0.84	28.5	373	1.7247	0.1832	0.0919	0.2694	0.3074
496	0.84	0.16	28.5	373	1.8507	0.1819	0.1246	0.2378	0.2606
497	0.84	0.34	28.5	373	1.8531	0.1822	0.1246	0.2488	0.2778
498	0.84	0.5	28.5	373	1.8551	0.1825	0.1246	0.2597	0.2924
499	0.84	0.66	28.5	373	1.8576	0.1827	0.1246	0.2742	0.3100
500	0.84	0.84	28.5	373	1.8617	0.1831	0.1246	0.2968	0.3360

Table 59. Non-linear parameters in the longitudinal building axis (part l) – Hospital Dr. Luis Edmundo Vasquez

	$\varsigma\, y_{percentile}$	$\varsigma\, pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
501	0.16	0.16	9.5	400	1.7647	0.1373	0.0744	0.1387	0.1507
502	0.16	0.34	9.5	400	1.7659	0.1376	0.0744	0.1442	0.1590
503	0.16	0.5	9.5	400	1.7669	0.1377	0.0744	0.1499	0.1646
504	0.16	0.66	9.5	400	1.7683	0.1379	0.0744	0.1561	0.1726
505	0.16	0.84	9.5	400	1.7690	0.1380	0.0744	0.1658	0.1814
506	0.34	0.16	9.5	400	1.9035	0.1375	0.0942	0.1576	0.1712
507	0.34	0.34	9.5	400	1.9046	0.1377	0.0942	0.1663	0.1796
508	0.34	0.5	9.5	400	1.9056	0.1379	0.0942	0.1692	0.1859
509	0.34	0.66	9.5	400	1.9068	0.1381	0.0942	0.1757	0.1937
510	0.34	0.84	9.5	400	1.9077	0.1382	0.0942	0.1849	0.2036
511	0.5	0.16	9.5	400	2.0124	0.1374	0.1156	0.1728	0.1879
512	0.5	0.34	9.5	400	2.0134	0.1375	0.1156	0.1793	0.1964
513	0.5	0.5	9.5	400	2.0144	0.1377	0.1156	0.1856	0.2037
514	0.5	0.66	9.5	400	2.0157	0.1379	0.1156	0.1913	0.2121
515	0.5	0.84	9.5	400	2.0166	0.1380	0.1156	0.2030	0.2216
516	0.66	0.16	9.5	400	2.1328	0.1372	0.1426	0.1919	0.2080
517	0.66	0.34	9.5	400	2.1337	0.1374	0.1426	0.1987	0.2164
518	0.66	0.5	9.5	400	2.1347	0.1375	0.1426	0.2034	0.2235
519	0.66	0.66	9.5	400	2.1358	0.1377	0.1426	0.2116	0.2319
520	0.66	0.84	9.5	400	2.1370	0.1378	0.1426	0.2213	0.2427
521	0.84	0.16	9.5	400	2.3220	0.1374	0.1835	0.2221	0.2424
522	0.84	0.34	9.5	400	2.3228	0.1375	0.1835	0.2312	0.2508
523	0.84	0.5	9.5	400	2.3236	0.1377	0.1835	0.2373	0.2581
524	0.84	0.66	9.5	400	2.3247	0.1378	0.1835	0.2422	0.2666
525	0.84	0.84	9.5	400	2.3261	0.1380	0.1835	0.2542	0.2784
526	0.16	0.16	14.25	400	1.6466	0.1659	0.0662	0.1603	0.1707
527	0.16	0.34	14.25	400	1.6483	0.1662	0.0662	0.1689	0.1821
528	0.16	0.5	14.25	400	1.6500	0.1664	0.0662	0.1741	0.1918
529	0.16	0.66	14.25	400	1.6519	0.1667	0.0662	0.1831	0.2037
530	0.16	0.84	14.25	400	1.6550	0.1670	0.0662	0.1981	0.2210
531	0.34	0.16	14.25	400	1.7695	0.1658	0.0841	0.1763	0.1910
532	0.34	0.34	14.25	400	1.7711	0.1661	0.0841	0.1850	0.2026
533	0.34	0.5	14.25	400	1.7726	0.1663	0.0841	0.1936	0.2125
534	0.34	0.66	14.25	400	1.7742	0.1665	0.0841	0.2026	0.2240
535	0.34	0.84	14.25	400	1.7771	0.1669	0.0841	0.2180	0.2423
536	0.5	0.16	14.25	400	1.8672	0.1657	0.1021	0.1940	0.2084
537	0.5	0.34	14.25	400	1.8687	0.1660	0.1021	0.2015	0.2199
538	0.5	0.5	14.25	400	1.8701	0.1662	0.1021	0.2104	0.2298
539	0.5	0.66	14.25	400	1.8716	0.1664	0.1021	0.2193	0.2414
540	0.5	0.84	14.25	400	1.8744	0.1667	0.1021	0.2330	0.2606
541	0.66	0.16	14.25	400	1.9750	0.1657	0.1247	0.2124	0.2289
542	0.66	0.34	14.25	400	1.9765	0.1660	0.1247	0.2222	0.2403
543	0.66	0.5	14.25	400	1.9778	0.1662	0.1247	0.2281	0.2499
544	0.66	0.66	14.25	400	1.9792	0.1664	0.1247	0.2388	0.2619
545	0.66	0.84	14.25	400	1.9817	0.1667	0.1247	0.2524	0.2814
546	0.84	0.16	14.25	400	2.1441	0.1655	0.1673	0.2439	0.2635
547	0.84	0.34	14.25	400	2.1453	0.1658	0.1673	0.2554	0.2745
548	0.84	0.5	14.25	400	2.1465	0.1659	0.1673	0.2629	0.2845
549	0.84	0.66	14.25	400	2.1477	0.1661	0.1673	0.2718	0.2960
550	0.84	0.84	14.25	400	2.1500	0.1664	0.1673	0.2862	0.3160

Table 60. Non-linear parameters in the longitudinal building axis (part m) – Hospital Dr. Luis Edmundo Vasquez

	$\varsigma\, y_{percentile}$	$\varsigma\, pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
551	0.16	0.16	19	400	1.5686	0.1775	0.0618	0.1664	0.1821
552	0.16	0.34	19	400	1.5710	0.1779	0.0618	0.1764	0.1964
553	0.16	0.5	19	400	1.5731	0.1781	0.0618	0.1858	0.2094
554	0.16	0.66	19	400	1.5756	0.1784	0.0618	0.1986	0.2228
555	0.16	0.84	19	400	1.5794	0.1788	0.0618	0.2156	0.2430
556	0.34	0.16	19	400	1.6812	0.1769	0.0790	0.1837	0.2001
557	0.34	0.34	19	400	1.6833	0.1773	0.0790	0.1935	0.2142
558	0.34	0.5	19	400	1.6851	0.1775	0.0790	0.2046	0.2264
559	0.34	0.66	19	400	1.6874	0.1778	0.0790	0.2147	0.2408
560	0.34	0.84	19	400	1.6911	0.1782	0.0790	0.2322	0.2619
561	0.5	0.16	19	400	1.7724	0.1772	0.0948	0.1988	0.2177
562	0.5	0.34	19	400	1.7744	0.1775	0.0948	0.2096	0.2313
563	0.5	0.5	19	400	1.7760	0.1777	0.0948	0.2203	0.2436
564	0.5	0.66	19	400	1.7782	0.1780	0.0948	0.2314	0.2580
565	0.5	0.84	19	400	1.7817	0.1784	0.0948	0.2504	0.2806
566	0.66	0.16	19	400	1.8724	0.1774	0.1164	0.2174	0.2370
567	0.66	0.34	19	400	1.8742	0.1777	0.1164	0.2280	0.2506
568	0.66	0.5	19	400	1.8757	0.1780	0.1164	0.2393	0.2631
569	0.66	0.66	19	400	1.8776	0.1782	0.1164	0.2483	0.2772
570	0.66	0.84	19	400	1.8810	0.1786	0.1164	0.2668	0.3006
571	0.84	0.16	19	400	2.0298	0.1773	0.1512	0.2501	0.2696
572	0.84	0.34	19	400	2.0315	0.1776	0.1512	0.2619	0.2833
573	0.84	0.5	19	400	2.0327	0.1778	0.1512	0.2684	0.2948
574	0.84	0.66	19	400	2.0344	0.1780	0.1512	0.2803	0.3094
575	0.84	0.84	19	400	2.0375	0.1784	0.1512	0.2971	0.3332
576	0.16	0.16	23.75	400	1.5272	0.1838	0.0542	0.1732	0.1933
577	0.16	0.34	23.75	400	1.5299	0.1842	0.0542	0.1846	0.2084
578	0.16	0.5	23.75	400	1.5325	0.1845	0.0542	0.1965	0.2224
579	0.16	0.66	23.75	400	1.5355	0.1848	0.0542	0.2075	0.2383
580	0.16	0.84	23.75	400	1.5396	0.1852	0.0542	0.2283	0.2606
581	0.34	0.16	23.75	400	1.6323	0.1836	0.0703	0.1880	0.2098
582	0.34	0.34	23.75	400	1.6347	0.1840	0.0703	0.2012	0.2251
583	0.34	0.5	23.75	400	1.6369	0.1843	0.0703	0.2118	0.2391
584	0.34	0.66	23.75	400	1.6397	0.1846	0.0703	0.2254	0.2555
585	0.34	0.84	23.75	400	1.6438	0.1850	0.0703	0.2464	0.2789
586	0.5	0.16	23.75	400	1.7181	0.1840	0.0853	0.2061	0.2275
587	0.5	0.34	23.75	400	1.7203	0.1843	0.0853	0.2168	0.2425
588	0.5	0.5	23.75	400	1.7224	0.1846	0.0853	0.2302	0.2569
589	0.5	0.66	23.75	400	1.7250	0.1849	0.0853	0.2401	0.2733
590	0.5	0.84	23.75	400	1.7291	0.1853	0.0853	0.2612	0.2979
591	0.66	0.16	23.75	400	1.8118	0.1839	0.1053	0.2215	0.2444
592	0.66	0.34	23.75	400	1.8140	0.1842	0.1053	0.2340	0.2608
593	0.66	0.5	23.75	400	1.8159	0.1845	0.1053	0.2464	0.2747
594	0.66	0.66	23.75	400	1.8181	0.1847	0.1053	0.2577	0.2903
595	0.66	0.84	23.75	400	1.8223	0.1852	0.1053	0.2794	0.3175
596	0.84	0.16	23.75	400	1.9606	0.1840	0.1390	0.2541	0.2770
597	0.84	0.34	23.75	400	1.9626	0.1843	0.1390	0.2651	0.2929
598	0.84	0.5	23.75	400	1.9642	0.1845	0.1390	0.2744	0.3062
599	0.84	0.66	23.75	400	1.9662	0.1848	0.1390	0.2890	0.3229
600	0.84	0.84	23.75	400	1.9701	0.1852	0.1390	0.3089	0.3504

Table 61. Non-linear parameters in the longitudinal building axis (part n) – Hospital Dr. Luis Edmundo Vasquez

	$\Phi y_{percentile}$	$\Phi pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	m* [ton]	Φ	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
601	0.16	0.16	28.5	400	1.0607	2253	1.294	0.1885	0.0459	0.1282	0.1469
602	0.16	0.34	28.5	400	1.0650	2253	1.294	0.1889	0.0459	0.1453	0.1649
603	0.16	0.5	28.5	400	1.0691	2253	1.294	0.1892	0.0459	0.1545	0.1799
604	0.16	0.66	28.5	400	1.0736	2253	1.294	0.1896	0.0459	0.1708	0.1984
605	0.16	0.84	28.5	400	1.0778	2253	1.294	0.1898	0.0459	0.1918	0.2270
606	0.34	0.16	28.5	400	1.0795	2253	1.294	0.1886	0.0480	0.1320	0.1488
607	0.34	0.34	28.5	400	1.0834	2253	1.294	0.1890	0.0480	0.1467	0.1663
608	0.34	0.5	28.5	400	1.0875	2253	1.294	0.1893	0.0480	0.1590	0.1824
609	0.34	0.66	28.5	400	1.0920	2253	1.294	0.1896	0.0480	0.1727	0.2003
610	0.34	0.84	28.5	400	1.0967	2253	1.294	0.1899	0.0480	0.1936	0.2290
611	0.5	0.16	28.5	400	1.0933	2253	1.294	0.1886	0.0832	0.1319	0.1502
612	0.5	0.34	28.5	400	1.0972	2253	1.294	0.1889	0.0832	0.1481	0.1682
613	0.5	0.5	28.5	400	1.1008	2253	1.294	0.1892	0.0832	0.1577	0.1829
614	0.5	0.66	28.5	400	1.1056	2253	1.294	0.1896	0.0832	0.1739	0.2016
615	0.5	0.84	28.5	400	1.1107	2253	1.294	0.1899	0.0832	0.1949	0.2306
616	0.66	0.16	28.5	400	1.1076	2253	1.294	0.1885	0.1134	0.1325	0.1511
617	0.66	0.34	28.5	400	1.1114	2253	1.294	0.1888	0.1134	0.1491	0.1689
618	0.66	0.5	28.5	400	1.1149	2253	1.294	0.1891	0.1134	0.1611	0.1842
619	0.66	0.66	28.5	400	1.1192	2253	1.294	0.1894	0.1134	0.1718	0.2013
620	0.66	0.84	28.5	400	1.1249	2253	1.294	0.1898	0.1134	0.1980	0.2319
621	0.84	0.16	28.5	400	1.1292	2253	1.294	0.1884	0.1398	0.1351	0.1533
622	0.84	0.34	28.5	400	1.1330	2253	1.294	0.1888	0.1398	0.1519	0.1712
623	0.84	0.5	28.5	400	1.1363	2253	1.294	0.1890	0.1398	0.1630	0.1866
624	0.84	0.66	28.5	400	1.1404	2253	1.294	0.1893	0.1398	0.1771	0.2044
625	0.84	0.84	28.5	400	1.1472	2253	1.294	0.1898	0.1398	0.2002	0.2347

Table 62. Non-linear parameters in the longitudinal building axis (part o) – Hospital Dr. Luis Edmundo Vasquez

	$\varsigma y_{percentile}$	$\varsigma pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
1	0.16	0.16	9.5	290	1.5318	0.1469	0.0400	0.1008	0.1238
2	0.16	0.34	9.5	290	1.5684	0.1539	0.0400	0.1174	0.1498
3	0.16	0.5	9.5	290	1.5947	0.1579	0.0400	0.1307	0.1709
4	0.16	0.66	9.5	290	1.6218	0.1614	0.0400	0.1499	0.1930
5	0.16	0.84	9.5	290	1.6231	0.1616	0.0400	0.1756	0.1942
6	0.34	0.16	9.5	290	1.6195	0.1442	0.0465	0.1043	0.1307
7	0.34	0.34	9.5	290	1.6538	0.1513	0.0465	0.1206	0.1534
8	0.34	0.5	9.5	290	1.6862	0.1568	0.0465	0.1335	0.1789
9	0.34	0.66	9.5	290	1.6978	0.1585	0.0465	0.1554	0.1885
10	0.34	0.84	9.5	290	1.7001	0.1588	0.0465	0.1856	0.1904
11	0.5	0.16	9.5	290	1.6879	0.1418	0.0519	0.1083	0.1362
12	0.5	0.34	9.5	290	1.7178	0.1484	0.0519	0.1239	0.1554
13	0.5	0.5	9.5	290	1.7556	0.1553	0.0519	0.1396	0.1827
14	0.5	0.66	9.5	290	1.7838	0.1596	0.0519	0.1533	0.2065
15	0.5	0.84	9.5	290	1.7931	0.1609	0.0519	0.1916	0.2145
16	0.66	0.16	9.5	290	1.7564	0.1376	0.0584	0.1124	0.1393
17	0.66	0.34	9.5	290	1.7901	0.1454	0.0584	0.1291	0.1599
18	0.66	0.5	9.5	290	1.8202	0.1514	0.0584	0.1435	0.1812
19	0.66	0.66	9.5	290	1.8661	0.1590	0.0584	0.1604	0.2171
20	0.66	0.84	9.5	290	1.8828	0.1613	0.0584	0.1945	0.2319
21	0.84	0.16	9.5	290	1.8675	0.1324	0.0687	0.1225	0.1494
22	0.84	0.34	9.5	290	1.9098	0.1423	0.0687	0.1390	0.1727
23	0.84	0.5	9.5	290	1.9320	0.1469	0.0687	0.1526	0.1879
24	0.84	0.66	9.5	290	1.9640	0.1529	0.0687	0.1702	0.2121
25	0.84	0.84	9.5	290	2.0063	0.1598	0.0687	0.1940	0.2458
26	0.16	0.16	14.25	290	1.4314	0.1781	0.0354	0.0966	0.1278
27	0.16	0.34	14.25	290	1.4621	0.1864	0.0354	0.1177	0.1520
28	0.16	0.5	14.25	290	1.4838	0.1911	0.0354	0.1361	0.1731
29	0.16	0.66	14.25	290	1.4997	0.1941	0.0354	0.1541	0.1906
30	0.16	0.84	14.25	290	1.5035	0.1947	0.0354	0.1833	0.1954
31	0.34	0.16	14.25	290	1.4987	0.1721	0.0413	0.0994	0.1282
32	0.34	0.34	14.25	290	1.5343	0.1829	0.0413	0.1187	0.1554
33	0.34	0.5	14.25	290	1.5580	0.1886	0.0413	0.1381	0.1762
34	0.34	0.66	14.25	290	1.5784	0.1928	0.0413	0.1577	0.1973
35	0.34	0.84	14.25	290	1.6014	0.1967	0.0413	0.1896	0.2263
36	0.5	0.16	14.25	290	1.5514	0.1666	0.0464	0.1035	0.1299
37	0.5	0.34	14.25	290	1.5925	0.1799	0.0464	0.1196	0.1590
38	0.5	0.5	14.25	290	1.6182	0.1866	0.0464	0.1372	0.1807
39	0.5	0.66	14.25	290	1.6411	0.1916	0.0464	0.1586	0.2035
40	0.5	0.84	14.25	290	1.6675	0.1964	0.0464	0.1908	0.2343
41	0.66	0.16	14.25	290	1.6098	0.1612	0.0501	0.1088	0.1347
42	0.66	0.34	14.25	290	1.6547	0.1761	0.0501	0.1245	0.1616
43	0.66	0.5	14.25	290	1.6811	0.1835	0.0501	0.1393	0.1840
44	0.66	0.66	14.25	290	1.7071	0.1896	0.0501	0.1616	0.2080
45	0.66	0.84	14.25	290	1.7396	0.1959	0.0501	0.1965	0.2448
46	0.84	0.16	14.25	290	1.6904	0.1511	0.0582	0.1136	0.1414
47	0.84	0.34	14.25	290	1.7439	0.1673	0.0582	0.1323	0.1650
48	0.84	0.5	14.25	290	1.7791	0.1781	0.0582	0.1448	0.1892
49	0.84	0.66	14.25	290	1.8053	0.1850	0.0582	0.1620	0.2131
50	0.84	0.84	14.25	290	1.8462	0.1938	0.0582	0.1989	0.2557

Table 63. Non-linear parameters in the transversal building axis (part a) – Hospital Dr. Luis Edmundo Vasquez

	$\varsigma\, y_{percentile}$	$\varsigma\, pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
51	0.16	0.16	19	290	1.3552	0.2220	0.0376	0.1055	0.1390
52	0.16	0.34	19	290	1.3894	0.2350	0.0376	0.1254	0.1721
53	0.16	0.5	19	290	1.4120	0.2415	0.0376	0.1507	0.1984
54	0.16	0.66	19	290	1.4282	0.2454	0.0376	0.1739	0.2200
55	0.16	0.84	19	290	1.4465	0.2492	0.0376	0.2107	0.2503
56	0.34	0.16	19	290	1.4101	0.2112	0.0436	0.1082	0.1377
57	0.34	0.34	19	290	1.4540	0.2300	0.0436	0.1268	0.1739
58	0.34	0.5	19	290	1.4801	0.2384	0.0436	0.1471	0.2023
59	0.34	0.66	19	290	1.5039	0.2447	0.0436	0.1753	0.2327
60	0.34	0.84	19	290	1.5239	0.2490	0.0436	0.2174	0.2635
61	0.5	0.16	19	290	1.4545	0.2034	0.0490	0.1121	0.1413
62	0.5	0.34	19	290	1.5039	0.2249	0.0490	0.1312	0.1741
63	0.5	0.5	19	290	1.5328	0.2352	0.0490	0.1476	0.2041
64	0.5	0.66	19	290	1.5597	0.2428	0.0490	0.1765	0.2372
65	0.5	0.84	19	290	1.5837	0.2483	0.0490	0.2175	0.2713
66	0.66	0.16	19	290	1.4971	0.1937	0.0520	0.1161	0.1448
67	0.66	0.34	19	290	1.5526	0.2163	0.0520	0.1363	0.1726
68	0.66	0.5	19	290	1.5904	0.2311	0.0520	0.1511	0.2067
69	0.66	0.66	19	290	1.6169	0.2394	0.0520	0.1726	0.2375
70	0.66	0.84	19	290	1.6500	0.2476	0.0520	0.2191	0.2829
71	0.84	0.16	19	290	1.5652	0.1825	0.0607	0.1222	0.1528
72	0.84	0.34	19	290	1.6277	0.2037	0.0607	0.1433	0.1798
73	0.84	0.5	19	290	1.6708	0.2204	0.0607	0.1568	0.2049
74	0.84	0.66	19	290	1.7069	0.2334	0.0607	0.1757	0.2406
75	0.84	0.84	19	290	1.7456	0.2443	0.0607	0.2161	0.2903
76	0.16	0.16	23.75	290	1.3270	0.2022	0.0310	0.0963	0.1311
77	0.16	0.34	23.75	290	1.3585	0.2109	0.0310	0.1222	0.1603
78	0.16	0.5	23.75	290	1.3749	0.2145	0.0310	0.1383	0.1784
79	0.16	0.66	23.75	290	1.3915	0.2176	0.0310	0.1631	0.2009
80	0.16	0.84	23.75	290	1.4126	0.2209	0.0310	0.1901	0.2323
81	0.34	0.16	23.75	290	1.3850	0.1974	0.0356	0.0989	0.1329
82	0.34	0.34	23.75	290	1.4191	0.2078	0.0356	0.1207	0.1618
83	0.34	0.5	23.75	290	1.4425	0.2134	0.0356	0.1424	0.1864
84	0.34	0.66	23.75	290	1.4587	0.2167	0.0356	0.1618	0.2056
85	0.34	0.84	23.75	290	1.4823	0.2206	0.0356	0.1972	0.2389
86	0.5	0.16	23.75	290	1.4234	0.1907	0.0402	0.1043	0.1314
87	0.5	0.34	23.75	290	1.4659	0.2049	0.0402	0.1214	0.1628
88	0.5	0.5	23.75	290	1.4911	0.2114	0.0402	0.1409	0.1876
89	0.5	0.66	23.75	290	1.5131	0.2162	0.0402	0.1638	0.2131
90	0.5	0.84	23.75	290	1.5375	0.2205	0.0402	0.1997	0.2456
91	0.66	0.16	23.75	290	1.4650	0.1834	0.0425	0.1051	0.1338
92	0.66	0.34	23.75	290	1.5190	0.2016	0.0425	0.1244	0.1661
93	0.66	0.5	23.75	290	1.5442	0.2086	0.0425	0.1392	0.1893
94	0.66	0.66	23.75	290	1.5691	0.2144	0.0425	0.1651	0.2162
95	0.66	0.84	23.75	290	1.5985	0.2200	0.0425	0.2030	0.2538
96	0.84	0.16	23.75	290	1.5265	0.1731	0.0497	0.1120	0.1403
97	0.84	0.34	23.75	290	1.5887	0.1921	0.0497	0.1286	0.1660
98	0.84	0.5	23.75	290	1.6016	0.1962	0.0497	0.1434	0.1735
99	0.84	0.66	23.75	290	1.6016	0.1962	0.0497	0.1633	0.1735
100	0.84	0.84	23.75	290	1.6016	0.1962	0.0497	0.1735	0.2163

Table 64. Non-linear parameters in the transversal building axis (part b) – Hospital Dr. Luis Edmundo Vasquez

	$\Xi y_{percentile}$	$\Xi pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
101	0.16	0.16	28.5	290	1.4364	0.1848	0.0295	0.0990	0.1296
102	0.16	0.34	28.5	290	1.4697	0.1950	0.0295	0.1219	0.1569
103	0.16	0.5	28.5	290	1.4918	0.2004	0.0295	0.1422	0.1790
104	0.16	0.66	28.5	290	1.5083	0.2038	0.0295	0.1628	0.1977
105	0.16	0.84	28.5	290	1.5270	0.2070	0.0295	0.1936	0.2244
106	0.34	0.16	28.5	290	1.5089	0.1790	0.0349	0.0998	0.1326
107	0.34	0.34	28.5	290	1.5425	0.1905	0.0349	0.1218	0.1593
108	0.34	0.5	28.5	290	1.5680	0.1975	0.0349	0.1439	0.1823
109	0.34	0.66	28.5	290	1.5889	0.2022	0.0349	0.1648	0.2047
110	0.34	0.84	28.5	290	1.6499	0.1527	0.0349	0.1985	0.2095
111	0.5	0.16	28.5	290	1.5605	0.1718	0.0383	0.1035	0.1334
112	0.5	0.34	28.5	290	1.6031	0.1873	0.0383	0.1216	0.1634
113	0.5	0.5	28.5	290	1.6270	0.1943	0.0383	0.1415	0.1846
114	0.5	0.66	28.5	290	1.6512	0.2003	0.0383	0.1665	0.2091
115	0.5	0.84	28.5	290	1.6795	0.2060	0.0383	0.2020	0.2431
116	0.66	0.16	28.5	290	1.6152	0.1639	0.0415	0.1058	0.1369
117	0.66	0.34	28.5	290	1.6639	0.1816	0.0415	0.1257	0.1639
118	0.66	0.5	28.5	290	1.6926	0.1909	0.0415	0.1411	0.1887
119	0.66	0.66	28.5	290	1.7198	0.1981	0.0415	0.1657	0.2151
120	0.66	0.84	28.5	290	1.7511	0.2049	0.0415	0.2021	0.2511
121	0.84	0.16	28.5	290	1.7080	0.1565	0.0492	0.1118	0.1483
122	0.84	0.34	28.5	290	1.7596	0.1731	0.0492	0.1312	0.1717
123	0.84	0.5	28.5	290	1.7937	0.1847	0.0492	0.1455	0.1947
124	0.84	0.66	28.5	290	1.8222	0.1932	0.0492	0.1653	0.2217
125	0.84	0.84	28.5	290	1.8594	0.2023	0.0492	0.1700	0.2616
126	0.16	0.16	9.5	317	1.5399	0.1542	0.0428	0.1019	0.1268
127	0.16	0.34	9.5	317	1.5725	0.1615	0.0428	0.1194	0.1506
128	0.16	0.5	9.5	317	1.5972	0.1660	0.0428	0.1329	0.1721
129	0.16	0.66	9.5	317	1.6243	0.1701	0.0428	0.1515	0.1954
130	0.16	0.84	9.5	317	1.6382	0.1720	0.0428	0.1858	0.2078
131	0.34	0.16	9.5	317	1.6295	0.1508	0.0514	0.1082	0.1340
132	0.34	0.34	9.5	317	1.6615	0.1586	0.0514	0.1236	0.1563
133	0.34	0.5	9.5	317	1.6840	0.1632	0.0514	0.1374	0.1735
134	0.34	0.66	9.5	317	1.7064	0.1672	0.0514	0.1535	0.1935
135	0.34	0.84	9.5	317	1.7093	0.1677	0.0514	0.1868	0.1962
136	0.5	0.16	9.5	317	1.6956	0.1468	0.0561	0.1126	0.1376
137	0.5	0.34	9.5	317	1.7303	0.1557	0.0561	0.1277	0.1603
138	0.5	0.5	9.5	317	1.7542	0.1610	0.0561	0.1453	0.1785
139	0.5	0.66	9.5	317	1.7794	0.1658	0.0561	0.1583	0.1992
140	0.5	0.84	9.5	317	1.7852	0.1668	0.0561	0.1938	0.2043
141	0.66	0.16	9.5	317	1.7705	0.1432	0.0612	0.1176	0.1440
142	0.66	0.34	9.5	317	1.8060	0.1526	0.0612	0.1324	0.1661
143	0.66	0.5	9.5	317	1.8287	0.1579	0.0612	0.1463	0.1827
144	0.66	0.66	9.5	317	1.8626	0.1647	0.0612	0.1665	0.2102
145	0.66	0.84	9.5	317	1.8878	0.1691	0.0612	0.1918	0.2326
146	0.84	0.16	9.5	317	1.8809	0.1364	0.0717	0.1270	0.1539
147	0.84	0.34	9.5	317	1.9236	0.1475	0.0717	0.1417	0.1764
148	0.84	0.5	9.5	317	1.9518	0.1544	0.0717	0.1558	0.1966
149	0.84	0.66	9.5	317	1.9744	0.1594	0.0717	0.1715	0.2143
150	0.84	0.84	9.5	317	2.0213	0.1682	0.0717	0.2034	0.2551

Table 65. Non-linear parameters in the transversal building axis (part c) – Hospital Dr. Luis Edmundo Vasquez

	$\varsigma y_{percentile}$	$\varsigma pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
151	0.16	0.16	14.25	317	1.4364	0.1848	0.0393	0.0992	0.1296
152	0.16	0.34	14.25	317	1.4697	0.1950	0.0393	0.1199	0.1569
153	0.16	0.5	14.25	317	1.4918	0.2004	0.0393	0.1394	0.1790
154	0.16	0.66	14.25	317	1.5083	0.2038	0.0393	0.1598	0.1977
155	0.16	0.84	14.25	317	1.5270	0.2070	0.0393	0.1887	0.2244
156	0.34	0.16	14.25	317	1.5089	0.1790	0.0433	0.1031	0.1326
157	0.34	0.34	14.25	317	1.5425	0.1905	0.0433	0.1191	0.1593
158	0.34	0.5	14.25	317	1.5680	0.1975	0.0433	0.1383	0.1823
159	0.34	0.66	14.25	317	1.5889	0.2022	0.0433	0.1622	0.2047
160	0.34	0.84	14.25	317	1.6499	0.1527	0.0433	0.1933	0.2095
161	0.5	0.16	14.25	317	1.5605	0.1718	0.0466	0.1056	0.1334
162	0.5	0.34	14.25	317	1.6031	0.1873	0.0466	0.1232	0.1634
163	0.5	0.5	14.25	317	1.6270	0.1943	0.0466	0.1404	0.1846
164	0.5	0.66	14.25	317	1.6512	0.2003	0.0466	0.1634	0.2091
165	0.5	0.84	14.25	317	1.6795	0.2060	0.0466	0.1961	0.2431
166	0.66	0.16	14.25	317	1.6152	0.1639	0.0518	0.1099	0.1369
167	0.66	0.34	14.25	317	1.6639	0.1816	0.0518	0.1268	0.1639
168	0.66	0.5	14.25	317	1.6926	0.1909	0.0518	0.1424	0.1887
169	0.66	0.66	14.25	317	1.7198	0.1981	0.0518	0.1647	0.2151
170	0.66	0.84	14.25	317	1.7511	0.2049	0.0518	0.2005	0.2511
171	0.84	0.16	14.25	317	1.7080	0.1565	0.0604	0.1197	0.1483
172	0.84	0.34	14.25	317	1.7596	0.1731	0.0604	0.1368	0.1717
173	0.84	0.5	14.25	317	1.7937	0.1847	0.0604	0.1506	0.1947
174	0.84	0.66	14.25	317	1.8222	0.1932	0.0604	0.1681	0.2217
175	0.84	0.84	14.25	317	1.8594	0.2023	0.0604	0.2031	0.2616
176	0.16	0.16	19	317	1.3751	0.1999	0.0344	0.0989	0.1320
177	0.16	0.34	19	317	1.4081	0.2102	0.0344	0.1205	0.1620
178	0.16	0.5	19	317	1.4273	0.2148	0.0344	0.1406	0.1826
179	0.16	0.66	19	317	1.4447	0.2184	0.0344	0.1645	0.2048
180	0.16	0.84	19	317	1.4626	0.2215	0.0344	0.1940	0.2312
181	0.34	0.16	19	317	1.4397	0.1940	0.0398	0.1028	0.1338
182	0.34	0.34	19	317	1.4776	0.2071	0.0398	0.1194	0.1662
183	0.34	0.5	19	317	1.5008	0.2133	0.0398	0.1411	0.1903
184	0.34	0.66	19	317	1.5182	0.2171	0.0398	0.1652	0.2103
185	0.34	0.84	19	317	1.5410	0.2214	0.0398	0.2022	0.2419
186	0.5	0.16	19	317	1.4810	0.1851	0.0439	0.1052	0.1325
187	0.5	0.34	19	317	1.5280	0.2028	0.0439	0.1219	0.1658
188	0.5	0.5	19	317	1.5536	0.2104	0.0439	0.1396	0.1907
189	0.5	0.66	19	317	1.5770	0.2160	0.0439	0.1670	0.2172
190	0.5	0.84	19	317	1.6024	0.2210	0.0439	0.2040	0.2502
191	0.66	0.16	19	317	1.5280	0.1769	0.0474	0.1082	0.1359
192	0.66	0.34	19	317	1.5844	0.1979	0.0474	0.1258	0.1671
193	0.66	0.5	19	317	1.6118	0.2068	0.0474	0.1410	0.1925
194	0.66	0.66	19	317	1.6382	0.2138	0.0474	0.1640	0.2209
195	0.66	0.84	19	317	1.6695	0.2204	0.0474	0.2056	0.2603
196	0.84	0.16	19	317	1.6016	0.1669	0.0566	0.1152	0.1440
197	0.84	0.34	19	317	1.6597	0.1856	0.0566	0.1332	0.1682
198	0.84	0.5	19	317	1.7021	0.2004	0.0566	0.1476	0.1959
199	0.84	0.66	19	317	1.7310	0.2091	0.0566	0.1657	0.2248
200	0.84	0.84	19	317	1.7705	0.2185	0.0566	0.2101	0.2716

Table 66. Non-linear parameters in the transversal building axis (part d) – Hospital Dr. Luis Edmundo Vasquez

	$\Xi y_{percentile}$	$\Xi pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
201	0.16	0.16	23.75	317	1.3457	0.2080	0.0341	0.0999	0.1342
202	0.16	0.34	23.75	317	1.3774	0.2179	0.0341	0.1226	0.1636
203	0.16	0.5	23.75	317	1.3976	0.2228	0.0341	0.1441	0.1861
204	0.16	0.66	23.75	317	1.4151	0.2263	0.0341	0.1688	0.2100
205	0.16	0.84	23.75	317	1.4341	0.2296	0.0341	0.1983	0.2397
206	0.34	0.16	23.75	317	1.4010	0.2010	0.0385	0.1038	0.1337
207	0.34	0.34	23.75	317	1.4404	0.2146	0.0385	0.1221	0.1667
208	0.34	0.5	23.75	317	1.4661	0.2214	0.0385	0.1437	0.1939
209	0.34	0.66	23.75	317	1.4842	0.2254	0.0385	0.1679	0.2160
210	0.34	0.84	23.75	317	1.5062	0.2295	0.0385	0.2047	0.2480
211	0.5	0.16	23.75	317	1.4417	0.1940	0.0418	0.1058	0.1348
212	0.5	0.34	23.75	317	1.4871	0.2107	0.0418	0.1229	0.1668
213	0.5	0.5	23.75	317	1.5153	0.2189	0.0418	0.1414	0.1948
214	0.5	0.66	23.75	317	1.5390	0.2244	0.0418	0.1704	0.2227
215	0.5	0.84	23.75	317	1.5627	0.2290	0.0418	0.2075	0.2546
216	0.66	0.16	23.75	317	1.4843	0.1864	0.0469	0.1098	0.1384
217	0.66	0.34	23.75	317	1.5372	0.2056	0.0469	0.1269	0.1672
218	0.66	0.5	23.75	317	1.5700	0.2159	0.0469	0.1440	0.1975
219	0.66	0.66	23.75	317	1.5967	0.2227	0.0469	0.1674	0.2274
220	0.66	0.84	23.75	317	1.6255	0.2286	0.0469	0.2099	0.2649
221	0.84	0.16	23.75	317	1.5991	0.1730	0.0542	0.1191	0.1469
222	0.84	0.34	23.75	317	1.6088	0.1951	0.0542	0.1334	0.1705
223	0.84	0.5	23.75	317	1.6336	0.2035	0.0542	0.1484	0.1848
224	0.84	0.66	23.75	317	1.6336	0.2035	0.0542	0.1670	0.1848
225	0.84	0.84	23.75	317	1.6336	0.2035	0.0542	0.1709	0.1848
226	0.16	0.16	28.5	317	1.2645	0.2032	0.0296	0.0854	0.1107
227	0.16	0.34	28.5	317	1.3006	0.2173	0.0296	0.1020	0.1368
228	0.16	0.5	28.5	317	1.3196	0.2232	0.0296	0.1184	0.1545
229	0.16	0.66	28.5	317	1.3366	0.2276	0.0296	0.1362	0.1732
230	0.16	0.84	28.5	317	1.3569	0.2320	0.0296	0.1630	0.1999
231	0.34	0.16	28.5	317	1.3105	0.1939	0.0345	0.0897	0.1129
232	0.34	0.34	28.5	317	1.3566	0.2124	0.0345	0.1052	0.1386
233	0.34	0.5	28.5	317	1.3800	0.2202	0.0345	0.1187	0.1586
234	0.34	0.66	28.5	317	1.3992	0.2257	0.0345	0.1383	0.1780
235	0.34	0.84	28.5	317	1.4261	0.2319	0.0345	0.1699	0.2104
236	0.5	0.16	28.5	317	1.3493	0.1883	0.0381	0.0944	0.1177
237	0.5	0.34	28.5	317	1.3993	0.2074	0.0381	0.1083	0.1405
238	0.5	0.5	28.5	317	1.4275	0.2174	0.0381	0.1232	0.1621
239	0.5	0.66	28.5	317	1.4491	0.2238	0.0381	0.1405	0.1827
240	0.5	0.84	28.5	317	1.4782	0.2309	0.0381	0.1735	0.2164
241	0.66	0.16	28.5	317	1.3829	0.1804	0.0417	0.0979	0.1208
242	0.66	0.34	28.5	317	1.4386	0.1995	0.0417	0.1132	0.1417
243	0.66	0.5	28.5	317	1.4724	0.2118	0.0417	0.1240	0.1611
244	0.66	0.66	28.5	317	1.5021	0.2213	0.0417	0.1396	0.1872
245	0.66	0.84	28.5	317	1.5223	0.2268	0.0417	0.1739	0.2082
246	0.84	0.16	28.5	317	1.4511	0.1749	0.0510	0.1027	0.1319
247	0.84	0.34	28.5	317	1.4985	0.1886	0.0510	0.1180	0.1486
248	0.84	0.5	28.5	317	1.5417	0.2025	0.0510	0.1312	0.1666
249	0.84	0.66	28.5	317	1.5617	0.2093	0.0510	0.1477	0.1775
250	0.84	0.84	28.5	317	1.5615	0.2092	0.0510	0.1734	0.1774

Table 67. Non-linear parameters in the transversal building axis (part e) – Hospital Dr. Luis Edmundo Vasquez

	$\varsigma y_{percentile}$	$\varsigma pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
251	0.16	0.16	9.5	345	1.5257	0.1641	0.0493	0.1041	0.1274
252	0.16	0.34	9.5	345	1.5445	0.1696	0.0493	0.1156	0.1412
253	0.16	0.5	9.5	345	1.5570	0.1729	0.0493	0.1267	0.1515
254	0.16	0.66	9.5	345	1.5724	0.1764	0.0493	0.1387	0.1649
255	0.16	0.84	9.5	345	1.5935	0.1807	0.0493	0.1531	0.1830
256	0.34	0.16	9.5	345	1.6112	0.1590	0.0598	0.1101	0.1327
257	0.34	0.34	9.5	345	1.6350	0.1665	0.0598	0.1225	0.1492
258	0.34	0.5	9.5	345	1.6482	0.1702	0.0598	0.1300	0.1594
259	0.34	0.66	9.5	345	1.6624	0.1738	0.0598	0.1420	0.1714
260	0.34	0.84	9.5	345	1.6840	0.1786	0.0598	0.1611	0.1906
261	0.5	0.16	9.5	345	1.6796	0.1555	0.0700	0.1160	0.1389
262	0.5	0.34	9.5	345	1.7000	0.1621	0.0700	0.1258	0.1519
263	0.5	0.5	9.5	345	1.7203	0.1682	0.0700	0.1346	0.1672
264	0.5	0.66	9.5	345	1.7331	0.1716	0.0700	0.1472	0.1773
265	0.5	0.84	9.5	345	1.7575	0.1774	0.0700	0.1673	0.1995
266	0.66	0.16	9.5	345	1.7527	0.1510	0.0818	0.1237	0.1453
267	0.66	0.34	9.5	345	1.7737	0.1579	0.0818	0.1317	0.1582
268	0.66	0.5	9.5	345	1.7934	0.1640	0.0818	0.1395	0.1718
269	0.66	0.66	9.5	345	1.8073	0.1680	0.0818	0.1510	0.1822
270	0.66	0.84	9.5	345	1.8329	0.1745	0.0818	0.1689	0.2042
271	0.84	0.16	9.5	345	1.8688	0.1457	0.1018	0.1356	0.1589
272	0.84	0.34	9.5	345	1.8883	0.1517	0.1018	0.1437	0.1696
273	0.84	0.5	9.5	345	1.9044	0.1567	0.1018	0.1520	0.1796
274	0.84	0.66	9.5	345	1.9215	0.1618	0.1018	0.1629	0.1914
275	0.84	0.84	9.5	345	1.9517	0.1701	0.1018	0.1764	0.2153
276	0.16	0.16	14.25	345	1.4420	0.1922	0.0403	0.1007	0.1331
277	0.16	0.34	14.25	345	1.4762	0.2038	0.0403	0.1214	0.1622
278	0.16	0.5	14.25	345	1.4983	0.2097	0.0403	0.1425	0.1846
279	0.16	0.66	14.25	345	1.5160	0.2137	0.0403	0.1632	0.2052
280	0.16	0.84	14.25	345	1.5352	0.2174	0.0403	0.1968	0.2330
281	0.34	0.16	14.25	345	1.5124	0.1837	0.0443	0.1065	0.1340
282	0.34	0.34	14.25	345	1.5506	0.1984	0.0443	0.1232	0.1643
283	0.34	0.5	14.25	345	1.5761	0.2062	0.0443	0.1413	0.1881
284	0.34	0.66	14.25	345	1.5988	0.2119	0.0443	0.1651	0.2132
285	0.34	0.84	14.25	345	1.6235	0.2169	0.0443	0.2003	0.2462
286	0.5	0.16	14.25	345	1.5651	0.1757	0.0485	0.1113	0.1360
287	0.5	0.34	14.25	345	1.6118	0.1945	0.0485	0.1264	0.1676
288	0.5	0.5	14.25	345	1.6379	0.2030	0.0485	0.1434	0.1922
289	0.5	0.66	14.25	345	1.6613	0.2094	0.0485	0.1686	0.2162
290	0.5	0.84	14.25	345	1.6908	0.2160	0.0485	0.2028	0.2528
291	0.66	0.16	14.25	345	1.6272	0.1697	0.0564	0.1170	0.1430
292	0.66	0.34	14.25	345	1.6729	0.1875	0.0564	0.1329	0.1681
293	0.66	0.5	14.25	345	1.7030	0.1985	0.0564	0.1467	0.1940
294	0.66	0.66	14.25	345	1.7272	0.2058	0.0564	0.1661	0.2184
295	0.66	0.84	14.25	345	1.7652	0.2150	0.0564	0.2059	0.2631
296	0.84	0.16	14.25	345	1.7170	0.1600	0.0644	0.1218	0.1526
297	0.84	0.34	14.25	345	1.7660	0.1763	0.0644	0.1391	0.1750
298	0.84	0.5	14.25	345	1.8017	0.1894	0.0644	0.1562	0.1970
299	0.84	0.66	14.25	345	1.8332	0.2001	0.0644	0.1740	0.2259
300	0.84	0.84	14.25	345	1.8727	0.2110	0.0644	0.2083	0.2699

Table 68. Non-linear parameters in the transversal building axis (part f) – Hospital Dr. Luis Edmundo Vasquez

	$\varsigma\, y_{percentile}$	$\varsigma\, pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
301	0.16	0.16	19	345	1.3826	0.2081	0.0363	0.1003	0.1367
302	0.16	0.34	19	345	1.4131	0.2185	0.0363	0.1226	0.1653
303	0.16	0.5	19	345	1.4355	0.2245	0.0363	0.1448	0.1901
304	0.16	0.66	19	345	1.4518	0.2281	0.0363	0.1682	0.2110
305	0.16	0.84	19	345	1.4693	0.2314	0.0363	0.2030	0.2380
306	0.34	0.16	19	345	1.4431	0.1987	0.0435	0.1048	0.1346
307	0.34	0.34	19	345	1.4840	0.2147	0.0435	0.1223	0.1695
308	0.34	0.5	19	345	1.5087	0.2221	0.0435	0.1460	0.1955
309	0.34	0.66	19	345	1.4828	0.2143	0.0435	0.1690	0.1683
310	0.34	0.84	19	345	1.5496	0.2312	0.0435	0.2077	0.2491
311	0.5	0.16	19	345	1.4884	0.1901	0.0461	0.1083	0.1361
312	0.5	0.34	19	345	1.5390	0.2109	0.0461	0.1283	0.1717
313	0.5	0.5	19	345	1.5633	0.2188	0.0461	0.1436	0.1963
314	0.5	0.66	19	345	1.5868	0.2251	0.0461	0.1717	0.2237
315	0.5	0.84	19	345	1.6129	0.2308	0.0461	0.2089	0.2584
316	0.66	0.16	19	345	1.5402	0.1828	0.0517	0.1127	0.1416
317	0.66	0.34	19	345	1.5907	0.2030	0.0517	0.1300	0.1683
318	0.66	0.5	19	345	1.6239	0.2151	0.0517	0.1469	0.1991
319	0.66	0.66	19	345	1.6477	0.2221	0.0517	0.1685	0.2255
320	0.66	0.84	19	345	1.6806	0.2299	0.0517	0.2094	0.2674
321	0.84	0.16	19	345	1.6127	0.1710	0.0590	0.1204	0.1486
322	0.84	0.34	19	345	1.6686	0.1897	0.0590	0.1395	0.1726
323	0.84	0.5	19	345	1.7119	0.2060	0.0590	0.1523	0.1987
324	0.84	0.66	19	345	1.7442	0.2170	0.0590	0.1706	0.2311
325	0.84	0.84	19	345	1.7842	0.2276	0.0590	0.2119	0.2805
326	0.16	0.16	23.75	345	1.3539	0.2126	0.0364	0.1023	0.1369
327	0.16	0.34	23.75	345	1.3539	0.2126	0.0364	0.1266	0.1369
328	0.16	0.5	23.75	345	1.4004	0.2272	0.0364	0.1435	0.1814
329	0.16	0.66	23.75	345	1.4137	0.2304	0.0364	0.1632	0.1975
330	0.16	0.84	23.75	345	1.4342	0.2345	0.0364	0.1907	0.2273
331	0.34	0.16	23.75	345	1.4157	0.2067	0.0402	0.1057	0.1392
332	0.34	0.34	23.75	345	1.4474	0.2184	0.0402	0.1274	0.1671
333	0.34	0.5	23.75	345	1.4655	0.2239	0.0402	0.1472	0.1841
334	0.34	0.66	23.75	345	1.4841	0.2288	0.0402	0.1681	0.2047
335	0.34	0.84	23.75	345	1.5066	0.2337	0.0402	0.1951	0.2354
336	0.5	0.16	23.75	345	1.4585	0.1993	0.0456	0.1104	0.1404
337	0.5	0.34	23.75	345	1.5004	0.2156	0.0456	0.1312	0.1716
338	0.5	0.5	23.75	345	1.5214	0.2222	0.0456	0.1491	0.1921
339	0.5	0.66	23.75	345	1.5403	0.2274	0.0456	0.1700	0.2113
340	0.5	0.84	23.75	345	1.5640	0.2329	0.0456	0.2024	0.2426
341	0.66	0.16	23.75	345	1.4998	0.1899	0.0493	0.1130	0.1433
342	0.66	0.34	23.75	345	1.5562	0.2113	0.0493	0.1323	0.1743
343	0.66	0.5	23.75	345	1.5777	0.2187	0.0493	0.1496	0.1950
344	0.66	0.66	23.75	345	1.5991	0.2250	0.0493	0.1744	0.2170
345	0.66	0.84	23.75	345	1.6283	0.2322	0.0493	0.2052	0.2526
346	0.84	0.16	23.75	345	1.5717	0.1807	0.0568	0.1219	0.1531
347	0.84	0.34	23.75	345	1.6322	0.2006	0.0568	0.1409	0.1791
348	0.84	0.5	23.75	345	1.6706	0.2142	0.0568	0.1551	0.2052
349	0.84	0.66	23.75	345	1.6910	0.2207	0.0568	0.1763	0.2265
350	0.84	0.84	23.75	345	1.7256	0.2299	0.0568	0.2122	0.2650

Table 69. Non-linear parameters in the transversal building axis (part g) – Hospital Dr. Luis Edmundo Vasquez

	$\varsigma y_{percentile}$	$\varsigma pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
351	0.16	0.16	28.5	345	1.3039	0.2244	0.0341	0.1026	0.1386
352	0.16	0.34	28.5	345	1.3355	0.2347	0.0341	0.1262	0.1693
353	0.16	0.5	28.5	345	1.3548	0.2397	0.0341	0.1468	0.1922
354	0.16	0.66	28.5	345	1.3689	0.2427	0.0341	0.1742	0.2120
355	0.16	0.84	28.5	345	1.3867	0.2460	0.0341	0.2021	0.2424
356	0.34	0.16	28.5	345	1.3578	0.2170	0.0390	0.1045	0.1378
357	0.34	0.34	28.5	345	1.3962	0.2310	0.0390	0.1257	0.1715
358	0.34	0.5	28.5	345	1.4188	0.2374	0.0390	0.1459	0.1965
359	0.34	0.66	28.5	345	1.4368	0.2416	0.0390	0.1715	0.2189
360	0.34	0.84	28.5	345	1.4580	0.2459	0.0390	0.2089	0.2517
361	0.5	0.16	28.5	345	1.3963	0.2091	0.0421	0.1093	0.1381
362	0.5	0.34	28.5	345	1.4422	0.2269	0.0421	0.1265	0.1720
363	0.5	0.5	28.5	345	1.4696	0.2353	0.0421	0.1448	0.2005
364	0.5	0.66	28.5	345	1.4909	0.2406	0.0421	0.1725	0.2263
365	0.5	0.84	28.5	345	1.5144	0.2455	0.0421	0.2135	0.2601
366	0.66	0.16	28.5	345	1.4377	0.2011	0.0478	0.1118	0.1419
367	0.66	0.34	28.5	345	1.4894	0.2210	0.0478	0.1304	0.1708
368	0.66	0.5	28.5	345	1.5194	0.2312	0.0478	0.1455	0.1992
369	0.66	0.66	28.5	345	1.5470	0.2388	0.0478	0.1708	0.2305
370	0.66	0.84	28.5	345	1.5747	0.2450	0.0478	0.2140	0.2677
371	0.84	0.16	28.5	345	1.5015	0.1906	0.0534	0.1186	0.1505
372	0.84	0.34	28.5	345	1.5597	0.2101	0.0534	0.1381	0.1751
373	0.84	0.5	28.5	345	1.5802	0.2175	0.0534	0.1540	0.1867
374	0.84	0.66	28.5	345	1.5803	0.2175	0.0534	0.1735	0.1868
375	0.84	0.84	28.5	345	1.5802	0.2175	0.0534	0.1821	0.1867
376	0.16	0.16	9.5	373	1.5305	0.1670	0.0536	0.1021	0.1249
377	0.16	0.34	9.5	373	1.5512	0.1744	0.0536	0.1131	0.1397
378	0.16	0.5	9.5	373	1.5686	0.1798	0.0536	0.1248	0.1539
379	0.16	0.66	9.5	373	1.5826	0.1835	0.0536	0.1377	0.1664
380	0.16	0.84	9.5	373	1.6044	0.1886	0.0536	0.1546	0.1867
381	0.34	0.16	9.5	373	1.6230	0.1623	0.0660	0.1091	0.1337
382	0.34	0.34	9.5	373	1.6452	0.1706	0.0660	0.1191	0.1486
383	0.34	0.5	9.5	373	1.6597	0.1755	0.0660	0.1302	0.1597
384	0.34	0.66	9.5	373	1.6767	0.1806	0.0660	0.1419	0.1742
385	0.34	0.84	9.5	373	1.6999	0.1866	0.0660	0.1607	0.1963
386	0.5	0.16	9.5	373	1.6928	0.1574	0.0765	0.1159	0.1394
387	0.5	0.34	9.5	373	1.7151	0.1658	0.0765	0.1266	0.1535
388	0.5	0.5	9.5	373	1.7354	0.1729	0.0765	0.1360	0.1683
389	0.5	0.66	9.5	373	1.7514	0.1780	0.0765	0.1459	0.1816
390	0.5	0.84	9.5	373	1.7752	0.1846	0.0765	0.1648	0.2039
391	0.66	0.16	9.5	373	1.7695	0.1528	0.0914	0.1252	0.1471
392	0.66	0.34	9.5	373	1.7923	0.1611	0.0914	0.1324	0.1607
393	0.66	0.5	9.5	373	1.8101	0.1674	0.0914	0.1405	0.1729
394	0.66	0.66	9.5	373	1.8272	0.1732	0.0914	0.1516	0.1859
395	0.66	0.84	9.5	373	1.8559	0.1818	0.0914	0.1706	0.2115
396	0.84	0.16	9.5	373	1.8906	0.1474	0.1097	0.1387	0.1627
397	0.84	0.34	9.5	373	1.9136	0.1550	0.1097	0.1461	0.1751
398	0.84	0.5	9.5	373	1.9301	0.1606	0.1097	0.1555	0.1852
399	0.84	0.66	9.5	373	1.9481	0.1667	0.1097	0.1645	0.1979
400	0.84	0.84	9.5	373	1.9781	0.1762	0.1097	0.1808	0.2223

Table 70. Non-linear parameters in the transversal building axis (part h) – Hospital Dr. Luis Edmundo Vasquez

	$\Xi y_{percentile}$	$\Xi pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
401	0.16	0.16	14.25	373	1.4509	0.1981	0.0418	0.1038	0.1350
402	0.16	0.34	14.25	373	1.4852	0.2115	0.0418	0.1236	0.1655
403	0.16	0.5	14.25	373	1.5070	0.2182	0.0418	0.1427	0.1875
404	0.16	0.66	14.25	373	1.5264	0.2231	0.0418	0.1644	0.2106
405	0.16	0.84	14.25	373	1.5472	0.2275	0.0418	0.2008	0.2410
406	0.34	0.16	14.25	373	1.5253	0.1892	0.0491	0.1110	0.1381
407	0.34	0.34	14.25	373	1.5638	0.2059	0.0491	0.1254	0.1689
408	0.34	0.5	14.25	373	1.5863	0.2137	0.0491	0.1433	0.1909
409	0.34	0.66	14.25	373	1.6088	0.2202	0.0491	0.1680	0.2156
410	0.34	0.84	14.25	373	1.6364	0.2266	0.0491	0.2018	0.2523
411	0.5	0.16	14.25	373	1.5812	0.1811	0.0548	0.1145	0.1418
412	0.5	0.34	14.25	373	1.6224	0.1995	0.0548	0.1311	0.1686
413	0.5	0.5	14.25	373	1.6512	0.2104	0.0548	0.1451	0.1965
414	0.5	0.66	14.25	373	1.6738	0.2175	0.0548	0.1692	0.2203
415	0.5	0.84	14.25	373	1.7068	0.2258	0.0548	0.2055	0.2627
416	0.66	0.16	14.25	373	1.6422	0.1736	0.0602	0.1200	0.1479
417	0.66	0.34	14.25	373	1.6842	0.1911	0.0602	0.1365	0.1702
418	0.66	0.5	14.25	373	1.7185	0.2052	0.0602	0.1498	0.1986
419	0.66	0.66	14.25	373	1.7435	0.2138	0.0602	0.1681	0.2251
420	0.66	0.84	14.25	373	1.7808	0.2240	0.0602	0.2089	0.2700
421	0.84	0.16	14.25	373	1.7308	0.1623	0.0696	0.1282	0.1564
422	0.84	0.34	14.25	373	1.7787	0.1787	0.0696	0.1440	0.1789
423	0.84	0.5	14.25	373	1.8186	0.1943	0.0696	0.1584	0.2024
424	0.84	0.66	14.25	373	1.8528	0.2074	0.0696	0.1782	0.2331
425	0.84	0.84	14.25	373	1.8894	0.2189	0.0696	0.2122	0.2754
426	0.16	0.16	19	373	1.3826	0.2081	0.0361	0.0999	0.1367
427	0.16	0.34	19	373	1.4131	0.2185	0.0361	0.1221	0.1653
428	0.16	0.5	19	373	1.4355	0.2245	0.0361	0.1442	0.1901
429	0.16	0.66	19	373	1.4518	0.2281	0.0361	0.1676	0.2110
430	0.16	0.84	19	373	1.4693	0.2314	0.0361	0.2022	0.2380
431	0.34	0.16	19	373	1.4431	0.1987	0.0433	0.1043	0.1346
432	0.34	0.34	19	373	1.4840	0.2147	0.0433	0.1218	0.1695
433	0.34	0.5	19	373	1.5087	0.2221	0.0433	0.1454	0.1955
434	0.34	0.66	19	373	1.5287	0.2269	0.0433	0.1683	0.2194
435	0.34	0.84	19	373	1.5496	0.2312	0.0433	0.2069	0.2491
436	0.5	0.16	19	373	1.4884	0.1901	0.0459	0.1079	0.1361
437	0.5	0.34	19	373	1.5390	0.2109	0.0459	0.1278	0.1717
438	0.5	0.5	19	373	1.5633	0.2188	0.0459	0.1430	0.1963
439	0.5	0.66	19	373	1.5868	0.2251	0.0459	0.1710	0.2237
440	0.5	0.84	19	373	1.6129	0.2308	0.0459	0.2080	0.2584
441	0.66	0.16	19	373	1.5402	0.1828	0.0515	0.1122	0.1416
442	0.66	0.34	19	373	1.5907	0.2030	0.0515	0.1294	0.1683
443	0.66	0.5	19	373	1.6239	0.2151	0.0515	0.1463	0.1991
444	0.66	0.66	19	373	1.6477	0.2221	0.0515	0.1678	0.2255
445	0.66	0.84	19	373	1.6806	0.2299	0.0515	0.2086	0.2674
446	0.84	0.16	19	373	1.6127	0.1710	0.0587	0.1199	0.1486
447	0.84	0.34	19	373	1.6686	0.1897	0.0587	0.1389	0.1726
448	0.84	0.5	19	373	1.7119	0.2060	0.0587	0.1517	0.1987
449	0.84	0.66	19	373	1.7442	0.2170	0.0587	0.1699	0.2311
450	0.84	0.84	19	373	1.7842	0.2276	0.0587	0.2111	0.2805

Table 71. Non-linear parameters in the transversal building axis (part i) – Hospital Dr. Luis Edmundo Vasquez

	$\Xi y_{percentile}$	$\Xi pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
451	0.16	0.16	23.75	373	1.3552	0.2220	0.0376	0.1055	0.1390
452	0.16	0.34	23.75	373	1.3894	0.2350	0.0376	0.1254	0.1721
453	0.16	0.5	23.75	373	1.4120	0.2415	0.0376	0.1507	0.1984
454	0.16	0.66	23.75	373	1.4282	0.2454	0.0376	0.1739	0.2200
455	0.16	0.84	23.75	373	1.4465	0.2492	0.0376	0.2107	0.2503
456	0.34	0.16	23.75	373	1.4101	0.2112	0.0436	0.1082	0.1377
457	0.34	0.34	23.75	373	1.4540	0.2300	0.0436	0.1268	0.1739
458	0.34	0.5	23.75	373	1.4801	0.2384	0.0436	0.1471	0.2023
459	0.34	0.66	23.75	373	1.5039	0.2447	0.0436	0.1753	0.2327
460	0.34	0.84	23.75	373	1.5239	0.2490	0.0436	0.2174	0.2635
461	0.5	0.16	23.75	373	1.4545	0.2034	0.0490	0.1121	0.1413
462	0.5	0.34	23.75	373	1.5039	0.2249	0.0490	0.1312	0.1741
463	0.5	0.5	23.75	373	1.5328	0.2352	0.0490	0.1476	0.2041
464	0.5	0.66	23.75	373	1.5597	0.2428	0.0490	0.1765	0.2372
465	0.5	0.84	23.75	373	1.5837	0.2483	0.0490	0.2175	0.2713
466	0.66	0.16	23.75	373	1.4971	0.1937	0.0520	0.1161	0.1448
467	0.66	0.34	23.75	373	1.5526	0.2163	0.0520	0.1363	0.1726
468	0.66	0.5	23.75	373	1.5904	0.2311	0.0520	0.1511	0.2067
469	0.66	0.66	23.75	373	1.6169	0.2394	0.0520	0.1726	0.2375
470	0.66	0.84	23.75	373	1.6500	0.2476	0.0520	0.2191	0.2829
471	0.84	0.16	23.75	373	1.5652	0.1825	0.0607	0.1222	0.1528
472	0.84	0.34	23.75	373	1.6237	0.2023	0.0607	0.1433	0.1779
473	0.84	0.5	23.75	373	1.6708	0.2204	0.0607	0.1568	0.2049
474	0.84	0.66	23.75	373	1.7069	0.2334	0.0607	0.1757	0.2406
475	0.84	0.84	23.75	373	1.7368	0.2421	0.0607	0.2039	0.2782
476	0.16	0.16	28.5	373	1.3061	0.2302	0.0356	0.1059	0.1383
477	0.16	0.34	28.5	373	1.3420	0.2432	0.0356	0.1261	0.1735
478	0.16	0.5	28.5	373	1.3613	0.2486	0.0356	0.1495	0.1963
479	0.16	0.66	28.5	373	1.3763	0.2522	0.0356	0.1751	0.2182
480	0.16	0.84	28.5	373	1.3932	0.2556	0.0356	0.2076	0.2474
481	0.34	0.16	28.5	373	1.3607	0.2213	0.0403	0.1070	0.1380
482	0.34	0.34	28.5	373	1.4005	0.2377	0.0403	0.1254	0.1719
483	0.34	0.5	28.5	373	1.4274	0.2461	0.0403	0.1488	0.2016
484	0.34	0.66	28.5	373	1.4463	0.2510	0.0403	0.1731	0.2256
485	0.34	0.84	28.5	373	1.4672	0.2555	0.0403	0.2146	0.2584
486	0.5	0.16	28.5	373	1.4002	0.2126	0.0461	0.1116	0.1398
487	0.5	0.34	28.5	373	1.4519	0.2344	0.0461	0.1289	0.1761
488	0.5	0.5	28.5	373	1.4761	0.2427	0.0461	0.1476	0.2015
489	0.5	0.66	28.5	373	1.5014	0.2497	0.0461	0.1765	0.2323
490	0.5	0.84	28.5	373	1.5247	0.2551	0.0461	0.2176	0.2663
491	0.66	0.16	28.5	373	1.4428	0.2041	0.0488	0.1153	0.1445
492	0.66	0.34	28.5	373	1.4983	0.2267	0.0488	0.1328	0.1737
493	0.66	0.5	28.5	373	1.5307	0.2388	0.0488	0.1501	0.2041
494	0.66	0.66	28.5	373	1.5563	0.2466	0.0488	0.1715	0.2335
495	0.66	0.84	28.5	373	1.5874	0.2544	0.0488	0.2171	0.2756
496	0.84	0.16	28.5	373	1.5058	0.1926	0.0565	0.1224	0.1524
497	0.84	0.34	28.5	373	1.5610	0.2114	0.0565	0.1387	0.1755
498	0.84	0.5	28.5	373	1.6085	0.2294	0.0565	0.1569	0.2035
499	0.84	0.66	28.5	373	1.6407	0.2407	0.0565	0.1736	0.2355
500	0.84	0.84	28.5	373	1.6828	0.2523	0.0565	0.2186	0.2891

Table 72. Non-linear parameters in the transversal building axis (part l) – Hospital Dr. Luis Edmundo Vasquez

	$\varsigma y_{percentile}$	$\varsigma pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
501	0.16	0.16	9.5	400	1.5451	0.1741	0.0566	0.1068	0.1310
502	0.16	0.34	9.5	400	1.5711	0.1841	0.0566	0.1190	0.1509
503	0.16	0.5	9.5	400	1.5832	0.1881	0.0566	0.1285	0.1615
504	0.16	0.66	9.5	400	1.5985	0.1927	0.0566	0.1442	0.1759
505	0.16	0.84	9.5	400	1.6199	0.1981	0.0566	0.1642	0.1971
506	0.34	0.16	9.5	400	1.6376	0.1679	0.0699	0.1154	0.1388
507	0.34	0.34	9.5	400	1.6587	0.1766	0.0699	0.1251	0.1534
508	0.34	0.5	9.5	400	1.6769	0.1835	0.0699	0.1344	0.1679
509	0.34	0.66	9.5	400	1.6958	0.1897	0.0699	0.1475	0.1851
510	0.34	0.84	9.5	400	1.7188	0.1962	0.0699	0.1683	0.2087
511	0.5	0.16	9.5	400	1.7127	0.1640	0.0815	0.1227	0.1472
512	0.5	0.34	9.5	400	1.7320	0.1719	0.0815	0.1333	0.1600
513	0.5	0.5	9.5	400	1.7505	0.1791	0.0815	0.1429	0.1739
514	0.5	0.66	9.5	400	1.7694	0.1858	0.0815	0.1530	0.1901
515	0.5	0.84	9.5	400	1.7931	0.1932	0.0815	0.1743	0.2131
516	0.66	0.16	9.5	400	1.7938	0.1600	0.0957	0.1331	0.1570
517	0.66	0.34	9.5	400	1.8124	0.1673	0.0957	0.1401	0.1687
518	0.66	0.5	9.5	400	1.8311	0.1746	0.0957	0.1485	0.1822
519	0.66	0.66	9.5	400	1.8491	0.1812	0.0957	0.1580	0.1967
520	0.66	0.84	9.5	400	1.8754	0.1899	0.0957	0.1763	0.2212
521	0.84	0.16	9.5	400	1.9161	0.1535	0.1208	0.1477	0.1726
522	0.84	0.34	9.5	400	1.9386	0.1615	0.1208	0.1569	0.1855
523	0.84	0.5	9.5	400	1.9523	0.1665	0.1208	0.1648	0.1944
524	0.84	0.66	9.5	400	1.9733	0.1742	0.1208	0.1716	0.2100
525	0.84	0.84	9.5	400	1.9993	0.1833	0.1208	0.1905	0.2322
526	0.16	0.16	14.25	400	1.4545	0.2021	0.0446	0.1063	0.1350
527	0.16	0.34	14.25	400	1.4921	0.2188	0.0446	0.1251	0.1688
528	0.16	0.5	14.25	400	1.5139	0.2263	0.0446	0.1454	0.1911
529	0.16	0.66	14.25	400	1.5353	0.2323	0.0446	0.1697	0.2173
530	0.16	0.84	14.25	400	1.5564	0.2371	0.0446	0.2069	0.2484
531	0.34	0.16	14.25	400	1.5287	0.1910	0.0531	0.1138	0.1384
532	0.34	0.34	14.25	400	1.5721	0.2121	0.0531	0.1308	0.1714
533	0.34	0.5	14.25	400	1.5934	0.2204	0.0531	0.1468	0.1931
534	0.34	0.66	14.25	400	1.6176	0.2283	0.0531	0.1692	0.2198
535	0.34	0.84	14.25	400	1.6481	0.2361	0.0531	0.2066	0.2612
536	0.5	0.16	14.25	400	1.5901	0.1846	0.0579	0.1186	0.1454
537	0.5	0.34	14.25	400	1.6304	0.2038	0.0579	0.1347	0.1704
538	0.5	0.5	14.25	400	1.6609	0.2170	0.0579	0.1507	0.1995
539	0.5	0.66	14.25	400	1.6820	0.2244	0.0579	0.1689	0.2227
540	0.5	0.84	14.25	400	1.7182	0.2347	0.0579	0.2102	0.2692
541	0.66	0.16	14.25	400	1.6547	0.1776	0.0631	0.1232	0.1529
542	0.66	0.34	14.25	400	1.6987	0.1969	0.0631	0.1404	0.1769
543	0.66	0.5	14.25	400	1.7296	0.2108	0.0631	0.1546	0.2019
544	0.66	0.66	14.25	400	1.7543	0.2204	0.0631	0.1740	0.2286
545	0.66	0.84	14.25	400	1.7919	0.2320	0.0631	0.2127	0.2747
546	0.84	0.16	14.25	400	1.7469	0.1664	0.0735	0.1338	0.1624
547	0.84	0.34	14.25	400	1.7967	0.1843	0.0735	0.1511	0.1870
548	0.84	0.5	14.25	400	1.8321	0.1988	0.0735	0.1668	0.2082
549	0.84	0.66	14.25	400	1.8636	0.2121	0.0735	0.1816	0.2348
550	0.84	0.84	14.25	400	1.9015	0.2258	0.0735	0.2170	0.2794

Table 73. Non-linear parameters in the transversal building axis (part m) – Hospital Dr. Luis Edmundo Vasquez

	$\varsigma y_{percentile}$	$\varsigma pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
551	0.16	0.16	19	400	1.3910	0.2193	0.0430	0.1079	0.1381
552	0.16	0.34	19	400	1.4260	0.2345	0.0430	0.1269	0.1720
553	0.16	0.5	19	400	1.4510	0.2426	0.0430	0.1503	0.2011
554	0.16	0.66	19	400	1.4673	0.2470	0.0430	0.1747	0.2223
555	0.16	0.84	19	400	1.4861	0.2512	0.0430	0.2131	0.2520
556	0.34	0.16	19	400	1.4538	0.2066	0.0481	0.1128	0.1384
557	0.34	0.34	19	400	1.4999	0.2290	0.0481	0.1300	0.1756
558	0.34	0.5	19	400	1.5244	0.2380	0.0481	0.1465	0.2025
559	0.34	0.66	19	400	1.5490	0.2454	0.0481	0.1765	0.2337
560	0.34	0.84	19	400	1.5705	0.2506	0.0481	0.2183	0.2654
561	0.5	0.16	19	400	1.5026	0.1974	0.0516	0.1162	0.1425
562	0.5	0.34	19	400	1.5552	0.2228	0.0516	0.1343	0.1757
563	0.5	0.5	19	400	1.5829	0.2342	0.0516	0.1504	0.2051
564	0.5	0.66	19	400	1.6068	0.2421	0.0516	0.1740	0.2342
565	0.5	0.84	19	400	1.6373	0.2501	0.0516	0.2167	0.2775
566	0.66	0.16	19	400	1.5604	0.1908	0.0588	0.1220	0.1502
567	0.66	0.34	19	400	1.6068	0.2112	0.0588	0.1395	0.1738
568	0.66	0.5	19	400	1.6460	0.2289	0.0588	0.1528	0.2069
569	0.66	0.66	19	400	1.6708	0.2380	0.0588	0.1724	0.2361
570	0.66	0.84	19	400	1.7059	0.2482	0.0588	0.2180	0.2835
571	0.84	0.16	19	400	1.6445	0.1806	0.0687	0.1286	0.1610
572	0.84	0.34	19	400	1.6944	0.1988	0.0687	0.1468	0.1844
573	0.84	0.5	19	400	1.7298	0.2134	0.0687	0.1617	0.2042
574	0.84	0.66	19	400	1.7736	0.2317	0.0687	0.1828	0.2440
575	0.84	0.84	19	400	1.8097	0.2437	0.0687	0.2172	0.2908
576	0.16	0.16	23.75	400	1.3595	0.2280	0.0406	0.1060	0.1409
577	0.16	0.34	23.75	400	1.3929	0.2420	0.0406	0.1285	0.1735
578	0.16	0.5	23.75	400	1.4192	0.2503	0.0406	0.1520	0.2048
579	0.16	0.66	23.75	400	1.4351	0.2545	0.0406	0.1764	0.2267
580	0.16	0.84	23.75	400	1.4528	0.2584	0.0406	0.2183	0.2561
581	0.34	0.16	23.75	400	1.4175	0.2168	0.0451	0.1109	0.1414
582	0.34	0.34	23.75	400	1.4609	0.2370	0.0451	0.1303	0.1770
583	0.34	0.5	23.75	400	1.4861	0.2461	0.0451	0.1482	0.2051
584	0.34	0.66	23.75	400	1.5102	0.2531	0.0451	0.1779	0.2360
585	0.34	0.84	23.75	400	1.5314	0.2581	0.0451	0.2201	0.2683
586	0.5	0.16	23.75	400	1.4603	0.2072	0.0503	0.1160	0.1443
587	0.5	0.34	23.75	400	1.5109	0.2307	0.0503	0.1325	0.1765
588	0.5	0.5	23.75	400	1.5393	0.2420	0.0503	0.1504	0.2061
589	0.5	0.66	23.75	400	1.5671	0.2508	0.0503	0.1765	0.2408
590	0.5	0.84	23.75	400	1.5935	0.2574	0.0503	0.2203	0.2787
591	0.66	0.16	23.75	400	1.5093	0.1992	0.0549	0.1211	0.1503
592	0.66	0.34	23.75	400	1.5608	0.2212	0.0549	0.1374	0.1762
593	0.66	0.5	23.75	400	1.5990	0.2375	0.0549	0.1553	0.2095
594	0.66	0.66	23.75	400	1.6255	0.2467	0.0549	0.1731	0.2409
595	0.66	0.84	23.75	400	1.6604	0.2563	0.0549	0.2213	0.2894
596	0.84	0.16	23.75	400	1.5818	0.1881	0.0644	0.1278	0.1595
597	0.84	0.34	23.75	400	1.6365	0.2074	0.0644	0.1463	0.1841
598	0.84	0.5	23.75	400	1.6755	0.2230	0.0644	0.1621	0.2056
599	0.84	0.66	23.75	400	1.7192	0.2404	0.0644	0.1794	0.2461
600	0.84	0.84	23.75	400	1.7563	0.2521	0.0644	0.2172	0.2946

Table 74. Non-linear parameters in the transversal building axis (part n) – Hospital Dr. Luis Edmundo Vasquez

	$\varsigma\, y_{percentile}$	$\varsigma\, pl_{percentile}$	f_c [MPa]	f_y [MPa]	T* [sec]	Cs [g]	Cd_{DL} [m]	Cd_{SD} [m]	Cd_{CO} [m]
601	0.16	0.16	28.5	400	1.3083	0.2358	0.0368	0.1060	0.1386
602	0.16	0.34	28.5	400	1.3395	0.2487	0.0368	0.1282	0.1683
603	0.16	0.5	28.5	400	1.3608	0.2557	0.0368	0.1499	0.1922
604	0.16	0.66	28.5	400	1.3743	0.2595	0.0368	0.1702	0.2102
605	0.16	0.84	28.5	400	1.3920	0.2637	0.0368	0.2012	0.2391
606	0.34	0.16	28.5	400	1.3613	0.2244	0.0428	0.1085	0.1380
607	0.34	0.34	28.5	400	1.4022	0.2432	0.0428	0.1268	0.1701
608	0.34	0.5	28.5	400	1.4256	0.2518	0.0428	0.1465	0.1950
609	0.34	0.66	28.5	400	1.4470	0.2583	0.0428	0.1733	0.2215
610	0.34	0.84	28.5	400	1.4647	0.2628	0.0428	0.2087	0.2472
611	0.5	0.16	28.5	400	1.4064	0.2173	0.0475	0.1120	0.1433
612	0.5	0.34	28.5	400	1.4520	0.2382	0.0475	0.1317	0.1724
613	0.5	0.5	28.5	400	1.4774	0.2482	0.0475	0.1479	0.1983
614	0.5	0.66	28.5	400	1.4999	0.2556	0.0475	0.1724	0.2239
615	0.5	0.84	28.5	400	1.5228	0.2619	0.0475	0.2081	0.2556
616	0.66	0.16	28.5	400	1.4463	0.2070	0.0516	0.1169	0.1468
617	0.66	0.34	28.5	400	1.4985	0.2291	0.0516	0.1353	0.1722
618	0.66	0.5	28.5	400	1.5328	0.2436	0.0516	0.1520	0.2004
619	0.66	0.66	28.5	400	1.5551	0.2516	0.0516	0.1728	0.2256
620	0.66	0.84	28.5	400	1.5867	0.2609	0.0516	0.2103	0.2662
621	0.84	0.16	28.5	400	1.5109	0.1952	0.0601	0.1239	0.1551
622	0.84	0.34	28.5	400	1.5665	0.2146	0.0601	0.1410	0.1788
623	0.84	0.5	28.5	400	1.6130	0.2330	0.0601	0.1590	0.2043
624	0.84	0.66	28.5	400	1.6446	0.2454	0.0601	0.1785	0.2321
625	0.84	0.84	28.5	400	1.6812	0.2573	0.0601	0.2127	0.2769

Table 75. Non-linear parameters in the transversal building axis (part o) – Hospital Dr. Luis Edmundo Vasquez