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# Optimal Load Factors for Seismic Design of Buildings

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**Abstract:** This paper reviews the load combinations for the design of buildings established in the Federal District Building Regulations (RCDF-2004) and its Complementary Technical Standards (NTC-2004). New load factors are proposed to be specified in the next version of the RCDF. The combination of gravity load (dead load plus live load) and the combination of earthquake load (dead load, live load and earthquake load) are revised. A methodology is proposed to establish optimal load factors and combinations that guarantee the minimum expected total cost during the lifetime of the structure and that the probability of failure is at least equal to that implied in the RCDF-2004. Artificial Neural Networks are used to estimate structural reliability.

**Key words:** load combination; failure probability; total expected cost; seismic design; artificial neural networks

## 1. Introduction

The Complementary Technical Norms on Criteria and Actions for the Structural Design of Buildings of the Construction Regulations for the Federal District specify that the safety of a structure should be verified for the combined effect of permanent, variable and accidental actions. The NTCCA-2004 also recommends that the effects of all actions should be multiplied by a load factor equal to 1.1 applied to the effects of all actions involved in the combination; that is, to check the effects in structures where seismic movements are included, the combination 1.1Cm + 1.1Cv + 1.1S is considered, where Cm = dead load, Cv= live load, S = earthquake. The load factors 1.1 that multiply Cm, Cv and S are based on brief studies and on the criteria of recognized researchers and structural engineers. The present study intends to give a rationale to these values or to modify them based on an optimization criterion. Here, a criterion is proposed and applied to find the optimum combination of load factors that includes the effects of dead, live and earthquake loads (Cm + Cv + S), so that the total cost is minimum and the probability of structural failure is at least equal to that implicit in the RCDF-2004. The proposed optimization criterion is applied to both reinforced concrete and steel buildings, located in Zone IIIb of the Valley of Mexico. For this purpose, a general survey of reinforced concrete and steel frame buildings located in Zone IIIb of the valley was carried out.

Structural design has been continuously changing and improving. Nowadays, the vast majority of codes in the world have tried to establish design criteria based on reliability, in order to improve structural behavior, reduce damage and reduce costs to acceptable values (Sorensen et al., 1994; Wen, 2001; Ellingwood, 1994a, 1994b; Bojórquez et al., 2017). There are several ways to calibrate design codes. Gayton et al., (2004) described some commonly used methods, which focused on evaluating the expected total cost of a building during its service life. Evaluating the total expected lifetime cost of structures is currently a major challenge for civil engineers, because structures are designed to have a relatively long service life, approximately 50 to 75 years (American Society of Civil Engineers, ASCE, 2010). This topic has been extensively studied in the last decades by several authors such as, for example: Akta et al., 2001; Ang, 2011; Mitropoulou et al., 2011; Lagaros, 2007; Wen and Kang, 2001a, 2001b; Baron and Frangopol, 2015; Esteva et al., 2011; Montiel and Ruiz, 2007; however, these methodologies are generally limited to the application of some particular examples. The total cost, as treated here, includes the initial cost and the cost for damage caused by future earthquakes. The present value of the cost includes different costs, namely, initial cost, repair cost, cost of content damage, cost associated with loss of human life, cost of injuries and economic losses.

This paper proposes a methodology to establish optimal load factors and combinations that guarantee the minimum expected total cost during the lifetime of the structure considering thousands of structures, and to perform this task, artificial intelligence tools are used. Probabilistic analyses are used to evaluate the structural performance and the occurrence of earthquakes is considered to be a Poisson process (Wang et al., 2014). The integration method is applied to calculate the annual exceedance rate of the maximum deformation level between a certain layer under a given seismic intensity. Seismic demands are simulated from the maximum interlayer distortion demand hazard curve. The simulation is performed using the inverse simulation method (Rubinstein, 1981). For the analysis, 31 motions originated by intense events (M>6.0) occurring in the Mexican Pacific subduction zone and recorded in different stations located in Zone IIIb of the valley of Mexico are used. The methodology is applied to steel and R/C buildings from 4 to 20 levels. In order to obtain the response of a large number of buildings, the Artificial Neural Networks technique is used. As a final result, combinations of appropriate values of dead, live and seismic load factors that are applicable to designs corresponding to Zone IIIb of the valley of Mexico are proposed.

## 2. Methodology

The general steps followed in this study are the following:

(1) Different C/R buildings, and alternatively steel buildings, are designed according to the specifications of the Federal District Building Regulations. The buildings are designed using different load combinations.

(2) The maximum floor-to-floor distortion associated with the limit states of interest is obtained by incremental dynamic analysis (IDA), (Vamvatsikos and Cornell, 2002).

(3) For each combination of design loads, and applying Eq. 1 (Cornell, 1968; Esteva, 1968), the annual exceedance rates of a certain level of distortion are determined (maximum distortion demand hazard curve):

$$V_D(\mathrm{d}) = \int \left| \frac{dv(Sa)}{d(Sa)} \right| P(D \geq d \mid Sa)\, d(Sa) \quad (1)$$

where:

d: represents the maximum interlayer distortion

$V_D(d)$: is the number of times per year that d is exceeded

Sa: is the ordinate of the spectrum of pseudo-accelerations

$P(D \geq d \mid Sa)d(Sa)$: represents the vulnerability curve. It is the probability that the value

D: value in a structure exceeds the distortion d, given an intensity Sa

v(Sa): is the average number of times per year that an intensity equal to or greater than Sa occurs. This function represents the seismic hazard curve of the site of interest.

(4) Subsequently, the annual failure probability is calculated for each combination, using eq. 2 (Cornell, 2002; Montiel and Ruiz, 2007).

$$V_f(d) = \int \left| \frac{dvD(d)}{d(d)} \right| p(C \le D)d(d) \quad (2)$$

where:

$V_f$: is the number of times per year that the demand D exceeds the capacity C

p(C≤D): is the cumulative distribution function of capacity

(5) Load factor combinations used in building designs that present an average annual structural failure rate greater than that implied by the Mexico City Building Code (V0, MCBC-04) are discarded; that is, the following condition is established:

$$Vf(\gamma) \le (V0, MCBC\text{-}04) \quad (3)$$

(6) From the simulated seismic demands, the damage index of the structures is calculated. With this it is possible to calculate the total costs, which must be transported at present value with the following expression:

$$VP = \frac{VF}{(1+i)^n} \quad (4)$$

where:

VP: is the time value at time 0 (i.e. the present)

VF: is the time value at time (future)

i: is the rate at which money will be compounded over time

n: is the year to be evaluated

(7) For each load combination, the total expected cost (Cy ) associated with the useful life of the structure (in the present study it is assumed that the buildings have a useful life of 50 years) is estimated as follows:

$$CT(\gamma) = CI(\gamma) + Cd(\gamma) \quad (5)$$

where:

$C_T$: represents the total expected cost per square meter of building

$C_I$: is the initial cost

$C_d$: is the cost associated with structural damage

γ: corresponds to each of the design load combinations

The total cost caused by earthquakes ($C_{TDS}$ ) for a group of buildings located in a given region associated with a certain load combination is estimated assuming that the structures in the group will suffer similar structural demands. Therefore, the $C_{TDS}$ in a region is assumed to be equal to the sum of the total lifetime costs of each building ($M_i$) located in that area. The expression to estimate such cost is (Bojórquez et al., 2017):

$$Ctds(\gamma i) = \sum Ct(\gamma i)(Mi) \quad (6)$$

where $C_{tds}(\gamma_i)$: total cost due to earthquake damage; $C_t(\gamma_i)(M_i)$ : total cost during the useful life of the building $M_i$; and $\gamma_i$: load combination with which the building is designed. Finally, the optimal combination of dead load, live load and seismic load factors is derived so that the total expected cost of the building is the minimum:

$$Min\ [Ctds(\gamma i)] \quad (7)$$

## 3. Total Cost over the Life of the Structure

The total cost is composed of the initial cost (Ci), damage repair cost (Cd) and social costs (Cs) that occur during the useful life of the structure. Thus, the total cost associated with a combination of load factors (γ) is given by the following expression:

$$Ct(\gamma) = Ci(\gamma) + Cd(\gamma) + Cs(\gamma) \quad (8)$$

The cost of damage repair and social costs are estimated from a measure of physical damage, and are established by means of a damage index, ID, which takes values between 0 and 1. Thus, in the case of total damage, the ID is equal to 1, while when there is no damage, the ID is equal to 0. The ID is defined by Eq. 9 (Tolentino and Ruiz, 2013):

$$ID = \frac{\delta d - \delta y}{\delta u - \delta y} \quad (9)$$

where δd: maximum interlayer demand distortion in the structure; δy: maximum interlayer distortion associated with the serviceability limit state (undamaged structure); and δu: is the maximum interlayer distortion associated with the collapse limit state. The values δy and δu are obtained from the statistics (expected value) of the incremental dynamic analysis results, while the value δd is the one obtained from the simulation of the structural demands from the seismic demand hazard curve in a specific time period.

3.1 Initial cost

The initial cost (Ci) is composed of the direct cost, indirect cost and profit, which are paid to the builder. This cost is estimated from the cost of materials (concrete and steel) using eq. 10 (Velázquez, 2015). Where Cm is the cost of materials.

$$Ci = 1.93Cm \quad (10)$$

In this work, the unit cost of concrete (f'c=250 kg/cm$^2$) is 2000 \$/m$^3$, while the unit cost of steel is 13 \$/kg. These costs are obtained from an average of prices quoted in Mexico.

3.2 Damage repair cost

The damage cost (Cd) of a structure during its useful life can be considered as the sum of the following costs: for repair or reconstruction (Cpr) and for loss of contents (Cpc). The damage repair cost is then expressed as:

$$Cd = Cpr + Cpc \quad (11)$$

a) Cost for repair or reconstruction

In many cases, the damage is very severe and for safety reasons, it is necessary to demolish the structure. De León and Ang (1995) state that from an ID>0.7 in concrete structures the repair can no longer be carried out, so it is necessary to demolish. When the ID>0.7, the maximum cost for reconstruction (including demolition, cleaning and redesign) of the structure is estimated as an additional 20% of the initial cost Ci while, for lower damage indices, the cost for repair is a function of the initial cost and the ID raised to the second power. Thus, the cost per repair considering the jacketing of structural members or cost per reconstruction is defined by eq. 12 (Bojórquez et al., 2015):

$$C_{pr} = \begin{cases} C_i(ID^2); 0 < ID < 0.7 \\ 1.2(C_i); ID \geq 0.7 \end{cases} \quad (12)$$

b) Cost for loss of content

The cost for loss of contents due to seismic movements may have greater importance depending on the use of the structure, since the contents of a hospital do not have the same economic value as the contents of an office building or a hotel. In this study, the maximum cost per loss of contents (ID≥1) is a fraction of the initial cost of the building, adopting a fraction of 50% (Surahman and Rojaniani, 1983). For the case of ID<1, a variation of cost as a function of ID is considered in a linear way. Thus, the cost per content is defined by Eq. 13 (De León, 1991):

$$C_{pc} = \begin{cases} 0.5(C_i)(ID); 0 < ID < 1 \\ 0.5(C_i); ID \geq 1 \end{cases} \quad (13)$$

3.3 Social costs

Social costs include the cost of economic loss (Cpe), loss of life (Cpv) and injury (Cpl). Social costs are then defined as:

$$Cs=Cpe+Cpv+Cpl \quad (14)$$

a) Economic loss cost

The cost for economic losses (Cpe) depends on the type of the structure use. In this study, buildings with office use are analyzed, so the economic losses are associated with not generating money due to the concept of rent during the time the structure is repaired or reconstructed. This maximum cost for economic losses (ID≥1) is a function of the maximum reconstruction period (Pr), the area of the building in square meters (A) and the cost per square meter of rent per month (R). Thus, the economic loss cost is estimated using Eq. 15 (De León, 1991; see Table 1). For the case of ID<1, a variation of the cost as a function of the ID raised to the second power is assumed. Based on the above, for the estimation of the cost for economic losses, it is considered that the average rent (R) of an office in Mexico City is equal to $\$250/m^2$ per month (Granados, 2015) and that the maximum reconstruction period (Pr) of a building is equal to 24 months.

b) Cost of loss of life

Estimating the cost per loss of life is a difficult task because it is a subjective issue, i.e., different criteria can be used to try to estimate this cost. In this study, the cost per loss of life is estimated considering the annual income of each person. On the other hand, in order to determine the cost per loss of life, it is necessary to estimate the average number of people killed within a construction area during intense seismic events. For this purpose, a nonlinear regression is used to estimate the number of deaths (Nd ) as a function of the collapsed area. This regression was obtained based on the total area of collapsed buildings in Mexico City during the 1985 earthquake (Instituto de Ingeniería UNAM, 1985) and the number of deaths (Tokyo Metropolitan Government, 1985). Thus, Nd is defined by the following expression:

$$N_{\mathrm{d}} = \frac{995.3A^{2.34}}{188 + A^{2.34}} \quad (16)$$

where A is the area of the collapsed building in 1000 $m^2$.

Bojórquez et al., (2017) assume that in the case of incipient collapse, the number of deaths per unit of collapsed area (Nd), represents 75% of the people inside the building and that only 5% of the people inside the building die, i.e., Nd represents the number of deaths for an extreme case. In this study, the case of incipient collapse is considered. The number of deaths in the case of incipient collapse is given by the following expression:

$$N\mathrm{m} = \frac{0.05N_d}{0.75} \quad (17)$$

The maximum cost per loss of life (ID≥1) is equal to the number of deaths in case of incipient collapse multiplied by the expected value of their income during their working life. It is considered that the average annual income of a person in Mexico is equal to \$156,000 (INEGI), and that the useful working life per individual is equal to 25 years, so the cost per death (Cpf) of a person is equal to \$3,900,000. Therefore, the cost per loss of life (Cpv) is defined with eq. 18 (De León, 1991; see Table 1). For ID<1, a variation of cost as a function of ID raised to the fourth power is assumed.

c) Cost of injuries

The evaluation of the cost per injury (Cpl ) refers to the costs involved during hospital stays of people injured in an earthquake. De León, (1991) estimated the average number of injured persons per unit of collapsed area of buildings, resulting in a value equal to $0.0168/m^2$, which is the result of dividing the number of injured persons reported in the 1985 earthquake that affected Mexico City (Tokyo Metropolitan Government, 1985) by the total area of collapsed buildings (Instituto de Ingeniería UNAM, 1985). On the other hand, the cost for non-disabling injuries (Csi) is considered equal to \$23,000 (Tokyo Metropolitan Government, 1985). The cost for disabling injuries (Cci) is assumed to be \$3,900,000 (equal to the cost of death). Taking the above into account, the maximum injury cost (ID≥1) is given by Eq. 19 (De Leon, 1991;

see Table 1), where the number of people with disability is considered to be equal to 10% of the total number of injured, while the remaining 90% have non-disabling injuries. For ID<1, a variation of the cost is assumed as a function of the ID raised to the second power. Table 1 summarizes the eqs. for estimating the social costs.

**Table 1.** Social costs

| Cost | 0<ID<1 | ID≥1 | eq. |
|---|---|---|---|
| For economic loss | Cpe = (Pr)(A)($ID^2$) | Cpe = R(Pr)(A) | 15 |
| For loss of life | Cpv =Nm (Cpf)($ID^4$) | Cpv = Nm (Cpf ) | 18 |
| For injuries | Cpl = [0.1Cci + 0.9Csi](0.0168)(A)($ID^2$) | Cpl = [0.1Cci + 0.9Csi](0.0168)(A) | 19 |

The costs described above should be transposed to present value using eq. 20:

$$VP = \frac{VF}{(1+i)^n} \qquad (20)$$

where VP: value at time 0 (i.e., the present); VF: value at time n (future); i: discount rate; and n: number of years to be considered.

## 4. Characteristics of the Study Area

4.1 Seismic movements

To analyze the buildings, the 31 acceleration maps recorded at the stations in the valley of Mexico shown in Table 2 were selected. The stations are located in Zone IIIb according to the seismic zoning of the Complementary Technical Standards for Earthquake Design (NTCDS-2004) of the RCDF. The main characteristics of the seismic events are shown in Table 2. In order to have different seismic intensities, the records were multiplied by different scaling factors. These factors are the ratio between the pseudo-acceleration corresponding to the vibration period of the system under study and the pseudo-acceleration corresponding to the intensity to which the movement is to be scaled (Shome and Cornell, 1999; Chan et al., 2005). The factor is multiplied by the full acceleration history of the record under study. The seismic hazard curves used in this work correspond to the site of the Secretary of Communications and Transportation (SCT).

**Table 2.** Characteristics of seismic events

| Date of earthquake | Epicenter coordinate N/W | Magnitude | Station |
|---|---|---|---|
| 97-01-11 | 17.910/103.04 | 6.9 | Valle Gómez |
| 95-10-09 | 18.74/104.67 | 7.3 | Valle Gómez |
| 89-04-25 | 16.603/99.400 | 6.9 | Tlatelolco |
| 95-09-14 | 16.31/98.88 | 7.4 | Tlatelolco |
| 97-01-11 | 17.91/103.04 | 6.9 | Tlatelolco |
| 89-04-25 | 16.603/99.400 | 6.9 | Garibaldi |
| 95-09-14 | 16.31/98.88 | 7.2 | Garibaldi |
| 95-10-09 | 18.74/104.67 | 7.3 | Garibaldi |
| 97-01-11 | 17.9/103 | 6.9 | Garibaldi |
| 95-09-14 | 16.31/98.88 | 7.2 | Alameda |
| 89-04-25 | 16.603/99.4 | 6.9 | Alameda |

| Date of earthquake | Epicenter coordinate N/W | Magnitude | Station |
|---|---|---|---|
| 89-04-25 | 16.603/99.4 | 6.9 | Tlatelolco |
| 95-09-14 | 16.31/98.88 | 7.2 | Tlatelolco |
| 95-10-09 | 18.74/104.67 | 7.3 | Liverpool |
| 97-01-11 | 17.9/103 | 6.9 | Liverpool |
| 95-09-14 | 16.31/98.88 | 7.2 | Cordoba |
| 95-10-09 | 18.74/104.67 | 7.3 | Cordoba |
| 97-01-11 | 17.9/103 | 6.9 | Cordoba |
| 89-04-25 | 16.603/99.4 | 6.9 | C.U. Juarez |
| 95-09-14 | 16.31/98.88 | 7.2 | C.U. Juarez |
| 95-10-09 | 18.74/104.67 | 7.3 | C.U. Juarez |
| 97-01-11 | 17.91/103.04 | 6.9 | C.U. Juarez |
| 95-09-14 | 16.31/98.88 | 7.2 | CUJP |
| 95-10-09 | 18.74/104.67 | 7.3 | CUJP |
| 97-01-11 | 17.9/103 | 6.9 | CUJP |
| 85-09-19 | 18.08/102.942 | 8.1 | SCT B-1 |
| 89-04-25 | 16.603/99.4 | 6.9 | SCT B-2 |
| 89-04-25 | 16.603/99.4 | 6.9 | Sector Popular |
| 95-09-14 | 16.31/98.88 | 7.2 | Sector Popular |
| 95-10-09 | 18.74/104.67 | 7.3 | Sector Popular |
| 97-01-11 | 17.91/103.04 | 6.9 | Sector Popular |

4.2 Survey of existing buildings in the study area

In order to generalize the results with the proposed methodology, it was necessary to survey all the buildings located in Zone IIIb of the Valley of Mexico, and to perform a lifetime cost analysis for each building using eq. 5; nevertheless, this task requires considerable computational time. To simplify the analysis in this work, regular reinforced concrete or steel buildings are considered. It is also assumed that buildings with less than 3 levels will not suffer significant damage because the fundamental period of these buildings is far from the period with the maximum ordinate of the mean response spectrum. In addition, buildings with more than 18 levels are grouped together since their fundamental period exceeds the period with the maximum peak of the mean response spectrum. Figs. 1 and 2 show the survey of all buildings with the aforementioned characteristics, located in Zone IIIb with dominant soil periods ranging from 1.5 to 2.0s of the Valley of Mexico (Velázquez, 2015). The buildings were grouped according to the number of levels, due to the fact that generally buildings with similar height were located very close to each other, for example, in residential complexes. It was found that there are approximately 140 thousand buildings were situated in this area, of which more than 134 thousand have less than 3 levels. The estimated number of C/R and steel buildings is shown in Table 3.

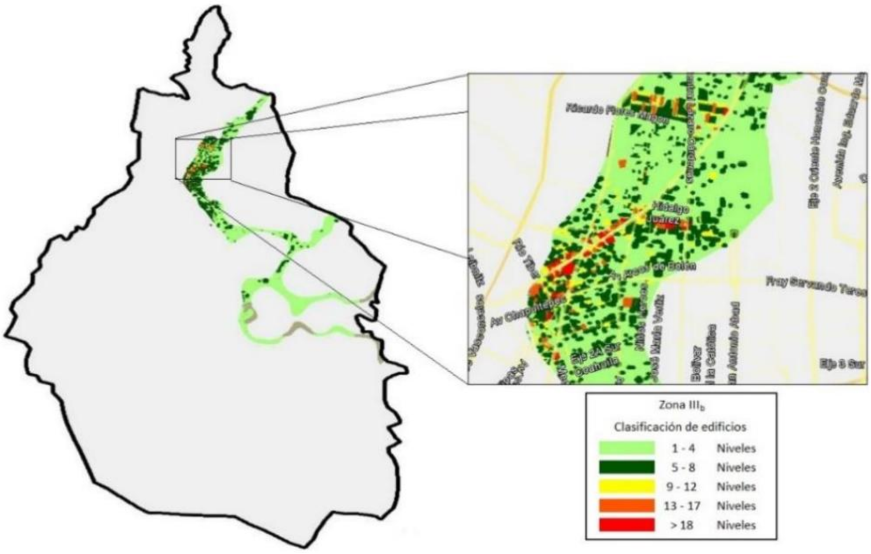


**Figure 1.** Map of the distribution of buildings in Zone IIIb.

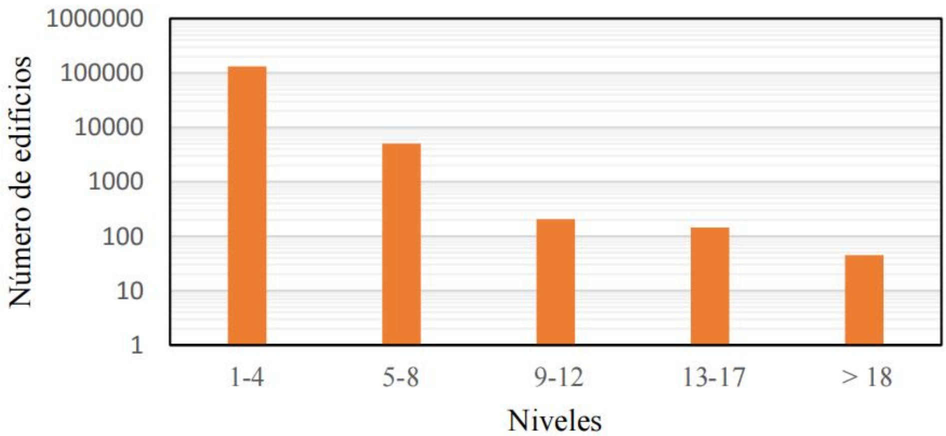


**Figure 2.** Histogram of the number of buildings located in Zone IIIb.

**Table 3.** Estimated number of C/R and steel buildings

| Group | Levels | Number of C/R buildings | Number of steel buildings |
|---|---|---|---|
| 1 | 1-4 | 109093 | 22273 |
| 2 | 5-8 | 4045 | 1011 |
| 3 | 9-12 | 152 | 53 |
| 4 | 13-17 | 84 | 60 |
| 5 | > 18 | 15 | 30 |

To evaluate eq. 7, considering all the buildings located in Zone IIIb, the theory of Artificial Neural Networks is used. Bojórquez et al., (2014) demonstrated that it is possible to obtain the design and response of regular C/R buildings using the ANN theory. In this study, such a model was used to obtain the designs of the whole set of buildings. In addition, another ANN model was developed to estimate the structural demand hazard curve, as well as the structural capacity curve for each of the buildings (Bojórquez et al., 2015). The database to train the buildings was obtained from previous studies, as well as from the designs performed in this study. The ANN models presented an error rate of no more than 15% in the buildings studied during training.

## 5. Characteristics of the Buildings Studied

5.1 Reinforced concrete buildings

Fourteen reinforced concrete buildings of 4, 6, 8, 10, 12, 15 and 20 levels are analyzed. For the building models, a seismic behavior factor Q = 3 was used. The C/R buildings were limited to δ/h = 0.03 for collapse, and δ /h = 0.004 for serviceability, where δ is the displacement and h is the height of the floor. The structure of the buildings is based on frames formed by three (or four) bays separated by distances of 6m for buildings of 4 and 6 levels, 8m for buildings of 8 and 10 levels, and 10m for buildings of 15 and 20 levels. In all cases, a mezzanine height of 4m is considered. The floor system is based on solid slabs. The structures are considered to be regular. The general geometric characteristics in plan and elevation are shown in Figs. 3a and 3b, respectively.

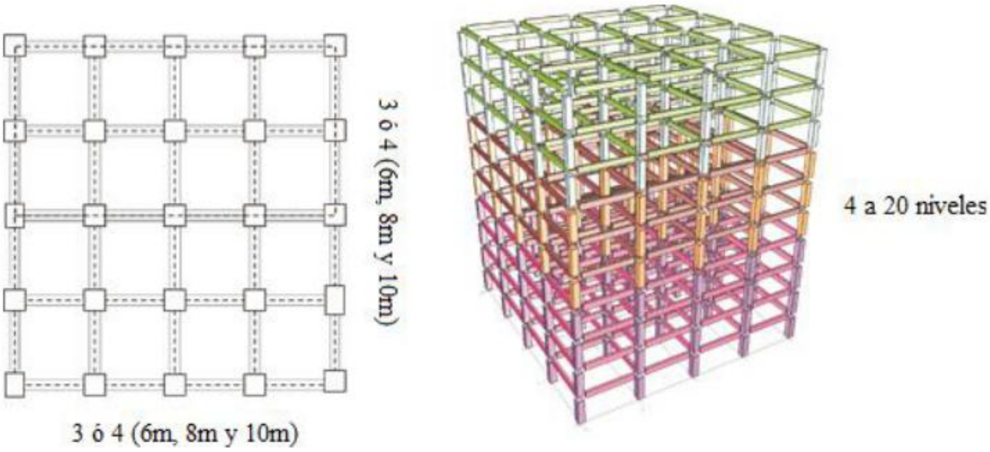


**Figure 3.** Plan and elevation of the C/R buildings analyzed.

5.2 Steel buildings

In parallel, seven steel buildings of 4, 6, 8, 10, 10, 12, 15 and 20 levels are analyzed. Buildings from 4 to 10 levels have 3 bays, and buildings from 12 to 20 levels have 4 bays. For the design of the buildings, a seismic performance factor Q = 3 was used. The floor-to-ceiling distortions were limited to δ/h = 0.015 for collapse, and δ/h = 0.004 for serviceability, where δ represents the displacement and h represents the height of the floor-to-ceiling.

The structure of the buildings is based on steel frames formed by three (or four) bays separated by distances of 6m for buildings of 4 and 6 levels, 8m for buildings of 8 and 10 levels, and 10m for buildings of 15 and 20 levels. In all cases, a mezzanine height of 4 m is considered. To stiffen the structure, concentric wind bracing was used in the central bay for the lower buildings (from 4 to 10 levels), and in the two end bays for the higher buildings (from 12 to 20 levels). The general geometric characteristics in plan and elevation of the models are shown in Figs. 4 and 5.

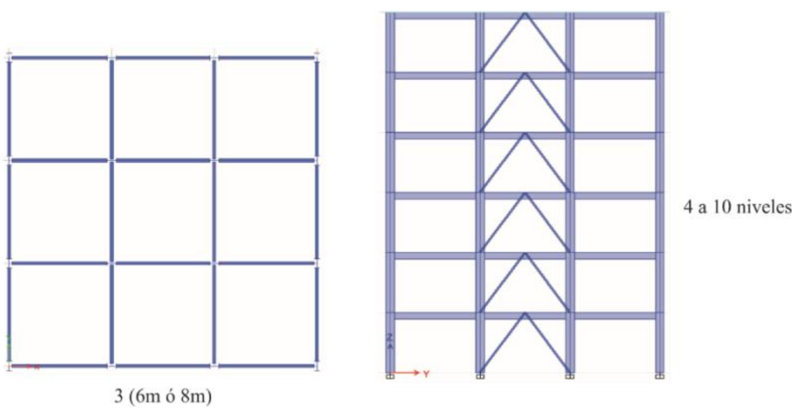


**Figure 4.** Floor plan and elevation of the steel buildings analyzed from 4 to 10 levels.

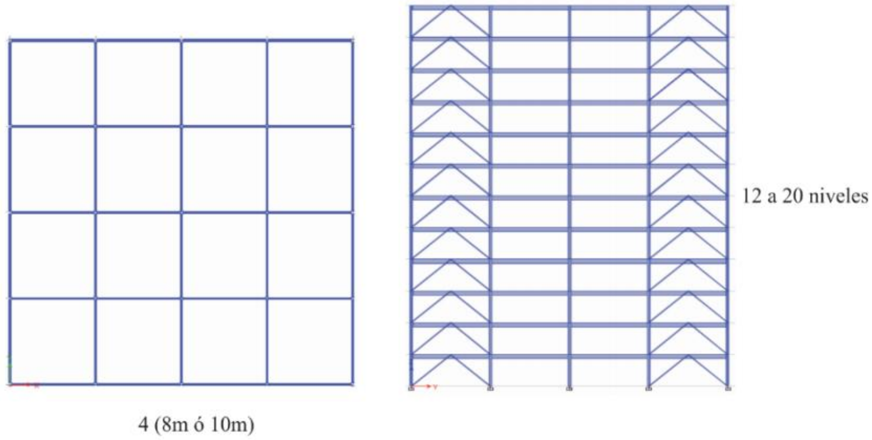


**Figure 5.** Plan and elevation of the analyzed steel buildings from 12 to 20 levels.

5.3 Structural analysis and design specifications

The design of the buildings is carried out in accordance with the criteria of the Mexico City Building Code (2004). The dead load ($F_{CM}$), instantaneous live load ($F_{CV}$) and seismic factors ($F_{CS}$) are used in the design combinations. Table 4 shows the different combinations used, noting that combination 1 is the one recommended by the RCDF-2004.

**Table 4.** Load combinations

| Combination | $F_{CM}$ | $F_{CV}$ | $F_{CS}$ |
|---|---|---|---|
| 1 | 1.1 | 1.1 | 1.1 |
| 2 | 1.0 | 1.0 | 1.0 |
| 3 | 1.0 | 1.0 | 1.1 |
| 4 | 1.0 | 1.0 | 1.2 |
| 5 | 1.1 | 1.0 | 1.2 |
| 6 | 1.2 | 1.0 | 1.0 |
| 7 | 1.2 | 1.0 | 1.1 |
| 8 | 1.2 | 1.0 | 1.2 |
| 9 | 1.1 | 1.0 | 1.0 |
| 10 | 1.1 | 1.0 | 1.1 |
| 11 | 1.1 | 1.1 | 1.2 |

5.4 Modeling of structures

In this study, incremental dynamic analyses are used to determine the behavior and structural capacity in the seismic environment of buildings designed for different combinations. For this purpose, representative three-dimensional models of the structure are developed in Ruaumoko3D software (Carr, 2007). In these models, it is assumed that the slabs provide a rigid diaphragm. In addition, stiffness and strength degradation of the reinforced concrete elements are considered in these analyses. For stiffness degradation, the Modified Takeda model (Otani, 1974) is used.

## 6. Application Example

In this section, the steps described in the methodology are applied to C/R buildings, and then applied to steel buildings. The results obtained for a 10-story C/R building are shown below.

6.1 Structural demand hazard curves

Knowing the seismic hazard curve for the fundamental period of vibration of the structure of interest and using Eq. 1, we can determine the maximum interlayer distortion demand hazard curve, representing the average annual exceedance rates of d. In Figure 6, the demand hazard curves are shown for the C/R building designed with the various load combinations listed in Table 4. These combinations are indicated on the graph using the term 'Combo'. The nomenclature 'M10-3' refers to a building with 10 stories in height and three bays in each direction.. Fig. 6 shows that for small maximum interlayer distortions, the difference in $V_D(d)$ between each design is small, which is due to the fact that for these intensity levels, the structure remains elastic; however, above a certain intensity level, the difference increases.

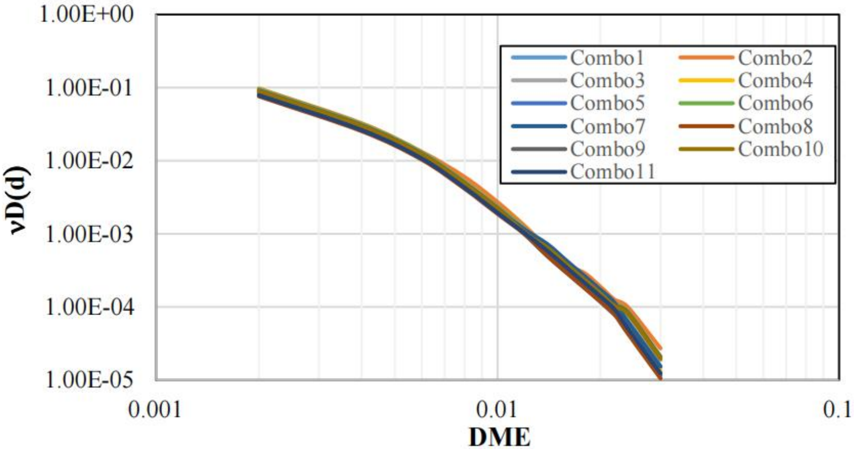


**Figure 6.** Demand hazard curves for frame M10-3.

6.2 Structural capacity curves

In the following, the steps described in Section 3.4 are applied first to a ten-story, three-bay building in both directions (M10-3), and then to all C/R buildings located in Zone IIIb. The building has a fundamental period of vibration of 1.25 sec. Based on the ADI's, the median and standard deviation of the logarithm of the maximum capacity are obtained, as well as the median and standard deviation of the logarithm of the creep capacity for each of the combinations studied. Fig. 7 shows the pseudo-acceleration vs. maximum interlayer distortions corresponding to the M10-3 frame, for each of the 31 records considered. Fig. 7 corresponds to the design obtained with combination 1. Table 5 presents the values of the logarithmic means of the curve fitting for the collapse and creep capacity as well as their respective logarithmic standard deviations, corresponding to the 11 combinations (combo) shown in Table 4.

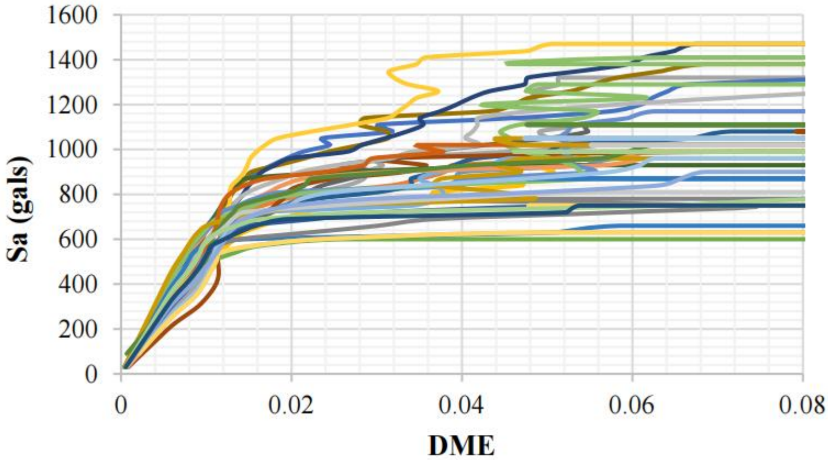


**Figure 7.** ADI's for C/R frame M10-3.

**Table 5.** DME for collapse and creep limit states for the M10-3 frame

| Combination (Combo) | Collapse | | Fluence | |
|---|---|---|---|---|
| | Median | σlnc | Median | σlnf |
| 1 | 0.0308 | 0.18 | 0.0082 | 0.080 |
| 2 | 0.0323 | 0.23 | 0.0077 | 0.082 |
| 3 | 0.0325 | 0.21 | 0.0079 | 0.089 |
| 4 | 0.0338 | 0.19 | 0.0083 | 0.091 |
| 5 | 0.0341 | 0.20 | 0.0083 | 0.078 |
| 6 | 0.0325 | 0.22 | 0.0077 | 0.083 |
| 7 | 0.0352 | 0.24 | 0.0082 | 0.085 |
| 8 | 0.0319 | 0.19 | 0.0084 | 0.090 |
| 9 | 0.0293 | 0.20 | 0.0077 | 0.083 |
| 10 | 0.0322 | 0.19 | 0.0082 | 0.086 |
| 11 | 0.034 | 0.22 | 0.0079 | 0.087 |

6.3 Simulation of structural demand

From the distortion demand hazard curve, the seismic demands are simulated. The simulation is performed using the inverse simulation method. It is assumed that the occurrence times are governed by Poisson processes and, therefore, the waiting time intervals of the structural demands follow an exponential distribution. A temporal analysis of demands is performed considering a structure lifetime of 50 years. The number of events per year is adjusted to the average number of events recorded in the Mexican Earthquake Database (SMIS-2015). On average, three events per year with a magnitude equal to or greater than 6.0 are expected. Fig. 8 shows the simulated structural DME demands corresponding to the M10-3 frame designed with combination 1.

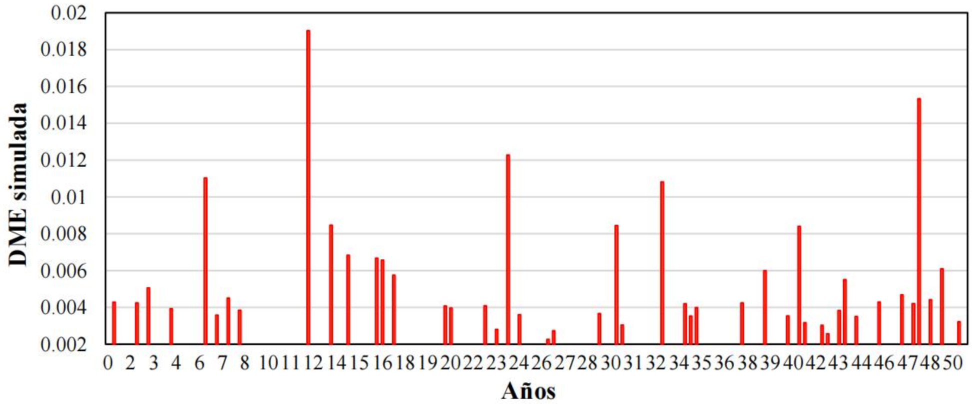


**Figure 8.** DME simulated from the demand hazard curve of the M10-3 framework.

6.4 Total cost associated with useful life

Based on the amount of material (concrete and steel) corresponding to the 12-story building designed with combination 1, Eq. 10 was applied to obtain the initial cost Ci = 29.60 million pesos (mdp).

On the other hand, the costs of damage repair and social costs occurring during the useful life of the structure is estimated from the maximum simulated interlayer distortions. Fig. 9 shows these costs due to the structural demands evaluated at each instant of time during the service life of the structure, duly transposed to the present value of money (eq. 20). An annual interest rate of 5% is assumed in this study.

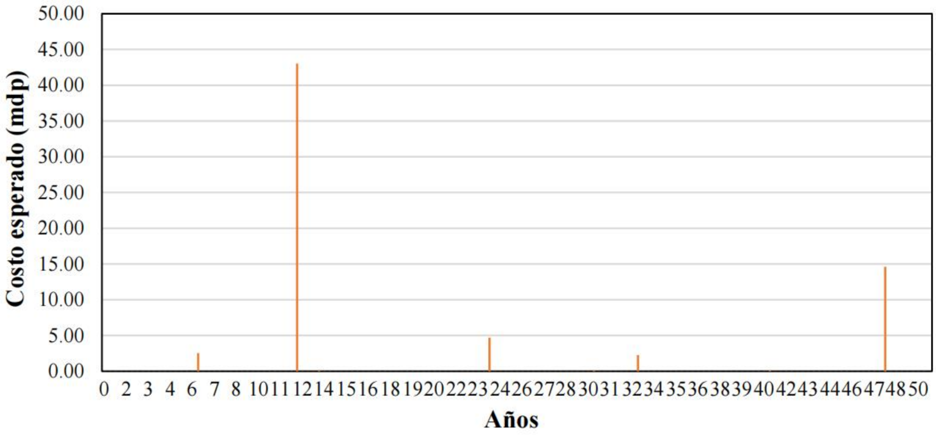


**Figure 9.** Cost generated during the useful life of the M10-3 building, Combo 1.

The damage repair cost and the social costs during the service life of the structure are obtained from the sum of the costs generated at each time instant (Fig. 9). The total cost (eq. 4) for the simulated scenario of structural demands shown in Fig. 9 is obtained by adding the initial cost (Ci), the damage repair cost (Cd) and the social cost (Cs), resulting in Ct = 88.18 million pesos.

The total cost previously obtained was only applicable when considering the simulation of a single scenario of seismic structural demands. However, as the final result, we took the average value of the total costs obtained from evaluating 100 simulations of scenarios of seismic structural demands for the same load combination ($\gamma$). The above procedure is applied to each of the designs obtained using the different load combinations. The summary of the total expected cost is shown in Table 6. Combinations 2, 3 and 9 do not meet the minimum failure probability constraint so they are not included in Table 6. Table 6 shows the structural failure rate (Vf) and its corresponding payback period. Table 6 shows that the minimum cost associated with structure M10-3 corresponds to Combination 8 (Combo 8), which is 21% lower than the cost obtained if the building is designed with the combination specified in the RCDF-2004 (Combo 1).

**Table 6.** Total expected lifetime cost of the M10-3 building for different combinations of load factors

| Combination | Total cost (mdp) | \$/m$^2$ (pesos) | Vf | Return period (years) |
|---|---|---|---|---|
| Combo 1 | 88.18 | 15,308.74 | 0.001305 | 766.3 |
| Combo 4 | 76.53 | 13,286.19 | 0.00124 | 806.5 |
| Combo 5 | 73.83 | 12,817.09 | 0.00123 | 813.0 |
| Combo 6 | 92.00 | 15,973.08 | 0.00129 | 775.2 |
| Combo 7 | 83.75 | 14,539.60 | 0.00128 | 781.3 |
| Combo 8 | 70.22 | 12,190.31 | 0.0012 | 833.3 |
| Combo 10 | 90.38 | 15,690.36 | 0.001296 | 771.6 |
| Combo 11 | 70.77 | 12,287.25 | 0.00122 | 819.7 |

## 7. Building Simulation Using Artificial Neural Networks

In order to evaluate eq. 7 (considering all the buildings located in the seismic zone of interest), the theory of Artificial Neural Networks is used.

With the database of the designed buildings, two ANN models and their respective computer programs were developed to perform the seismic design of buildings located in Zone IIIb of the Valley of Mexico. The first program provides the dimensions and reinforcement amounts for buildings from 3 to 20 levels in such a way that it is possible to simulate buildings located in this zone in a very short time interval (Bojórquez et al., 2014, 2016). In addition, an ANN model has been developed that allows estimating both the structural capacity and the demand curve of the buildings simulated with the first program. With these tools, the simulation of the whole seismic zone is done obtaining a total cost map of buildings located in Zone IIIb. The optimal load combination is obtained by minimizing the total cost during the useful life of the buildings. For this case, the buildings shown in Fig. 2 were randomly simulated.

7.1 Results for all buildings

In order to analyze the influence of the fundamental period of the structures on the factors corresponding to the optimum load combination, the following separates the structural models by intervals according to their fundamental period of vibration (T). The intervals considered are shown in Table 7:

**Table 7.** Fundamental vibration period intervals

| Interval (seconds) |
|---|
| $0.3 < T \leq 0.8$; $0.8 < T \leqslant 1.3$; $1.3 < T \leqslant 1.8$; $1.8 \leqslant T$ |

The results of the total expected costs and failure rates of all the buildings located in Zone IIIb of Mexico City are summarized in Table 8, which shows the summary results of the average of 100 simulations of structural demands. It is observed that combination 10 exhibits the lowest total expected cost, which is 2% smaller than that of combination 1. However, the failure rate of combination 1 is slightly higher than that of combination 10. From the results not shown, it was observed that the costs are similar for high-rise buildings when the dead load factor ($F_{CM}$) is increased without modifying the seismic factor ($F_{CS}$). However, buildings with smaller basic vibration periods are different. Increasing the dead load factor ($F_{CM}$) without modifying the $F_{CS}$ will result in greater cost changes, as gravity loads have a greater impact on the design in these cases. The buildings most vulnerable to earthquakes in Zone IIIb are in the second and third intervals, i.e. buildings with fundamental periods between 0.8 and 1.8 secs.

**Table 8.** Total costs for the whole building assembly (C/R and steel)

| Combination | Total cost (mdp) | $/m$^2$ (pesos) | $V_f$ | Return period |
|---|---|---|---|---|
| Combo 1 | 2,296.49 | 7,997.41 | 0.0005184 | 1,947.82 |
| Combo 4 | 2,486.57 | 8,546.38 | 0.000486504 | 2,055.48 |
| Combo 5 | 2,453.23 | 8,456.85 | 0.000467102 | 2,140.86 |
| Combo 6 | 2,407.77 | 8,265.76 | 0.00050758 | 1,970.13 |
| Combo 7 | 2,376.41 | 8,236.15 | 0.000505905 | 1,976.66 |
| Combo 8 | 2,484.90 | 8,522.55 | 0.000494394 | 2,022.68 |
| Combo 10 | 2,254.75 | 7,812.27 | 0.0005134 | 1,929.03 |
| Combo 11 | 2,408.11 | 8,320.89 | 0.000479823 | 2,084.10 |

## 8. Conclusion

A methodology was proposed to optimize the load factors for the seismic design of reinforced concrete and steel buildings located in Zone IIIb of Mexico City and designed according to the Mexico City Building Code 2004. The optimal combination of load factors was obtained using structural reliability analysis and artificial intelligence techniques. The procedure was to minimize the total expected cost over the useful life of the buildings, and to have an annual probability of failure equal to or less than that implied by designs made in accordance with the Building Regulations for the Federal District in force. It was found that the dead, live and seismic load factors are dependent on the fundamental period of vibration of the buildings. The results are summarized as follows:

| Interval (seconds) | Load factor |
|---|---|
| 0.3 < T < 0.8 | FCm = 1.1, FCv = 1.0, FCs = 1.0 |
| 0.8 < T < 1.3 | FCm = 1.1, FCv = 1.1, FCs = 1.2 |
| 1.3 < T < 1.8 | FCm = 1.1, FCv = 1.1, FCs = 1.1 |
| T > 1.8 | FCm = 1.1, FCv = 1.0, FCs = 1.1 |
| Full set | FCm = 1.1, FCv = 1.0, FCs = 1.1 |

For the parameters studied in this work, it was found that the load factors are independent of the cost of loss of human life. The return periods of failure associated with all the ranges of vibration periods are around 2000 years or more, except for the second period interval, which is in the order of 1000 years. If only one load combination were to be implemented in the Mexico City Building Code, the following combination of load factors is recommended for the design of buildings located in Zone IIIb.

FCm = 1.1, FCv = 1.1, FCs = 1.2


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## Conflicts of Interest

The author declares no conflicts of interest regarding the publication of this paper.

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