UNIVERSIDAD CATÓLICA SANTO TORIBIO DE MOGROVEJO

FACULTAD DE INGENIERÍA

ESCUELA DE INGENIERÍA CIVIL AMBIENTAL

Optimización de cuantía de acero en muros estructurales mediante verificación no lineal de diseño basado en desempeño en un edificio de 7 niveles

TRABAJO DE INVESTIGACIÓN PARA OPTAR EL GRADO ACADÉMICO DE BACHILLER EN INGENIERÍA CIVIL AMBIENTAL

AUTOR

Jorge Olivos Vasquez

ASESOR

Angel Antonio Ruiz Pico

https://orcid.org/0000-0003-2638-0593

Chiclayo, 2023

TIB

INFORME DE ORIGINALIDAD

12%	10%	8%	5%
INDICE DE SIMILITUD	FUENTES DE INTERNET	PUBLICACIONES	TRABAJOS DEL ESTUDIANTE

FUENTES PRIMARIAS

1	revistas.udea.edu.co Fuente de Internet	1%
2	tesis.usat.edu.pe Fuente de Internet	1%
3	revistadelaconstruccion.uc.cl Fuente de Internet	1%
4	store.atcouncil.org Fuente de Internet	1%
5	Submitted to Cebu Institute of Technology Trabajo del estudiante	1%
6	Ricardo Fitas, Gonçalo das Neves Carneiro, Carlos Conceição António. "An elitist multi-objective particle swarm optimization algorithm for composite structures design", Composite Structures, 2022 Publicación	1%
7	opencivilengineeringjournal.com Fuente de Internet	<1%

Índice

Resumen

Este artículo muestra los resultados de la optimización de la cuantía de acero en muros de concreto armado de un edificio de 7 niveles. Para ello se realizó un análisis lineal estático siguiendo lo descrito en la norma peruana E060, también se realizó un análisis no lineal Pushover, siguiendo lo estipulado en la norma ACI.318.19. en cuanto al análisis lineal se evaluaron aspectos de irregularidades, derivas máximas, fuerzas cortantes en l base de la estructura y los modos de participación, para así poder realizar el diseño de los muros estructurales de la edificación.

Por otra parte, para el análisis no lineal se realizó un análisis global de la estructura, evaluándola para sismo de servicio, de diseño y máximo. Para estos tres se evaluaron las derivas, además también se obtuvieron las curvas de capacidad de la estructura y el nivel de desempeño sísmico.

Finalmente se pudo optimizar la cuantía de acero, por lo cual se pudo cumplir con el objetivo principal y se comprobó la eficacia de un análisis no lineal.

Palabras clave: Análisis no lineal, optimización, análisis lineal, cuantía, muros estructurales

Abstract

This article shows the results of optimizing the amount of steel in reinforced concrete walls of a 7-story building. For this, a static linear analysis was carried out following what is described in the Peruvian standard E060, a non-linear Pushover analysis was also carried out, following what is stipulated in the ACI.318.19 standard. Regarding the linear analysis, aspects of irregularities, maximum drifts, shear forces at the base of the structure and the modes of participation were evaluated, in order to carry out the design of the structural walls of the building.

On the other hand, for the nonlinear analysis, a global analysis of the structure was carried out, evaluating it for service, design and maximum earthquakes. For these three the drifts were evaluated, in addition the capacity curves of the structure and the level of seismic performance were also obtained.

Finally, the amount of steel could be optimized, so the main objective could be met and the effectiveness of a non-linear analysis was proven.

Keywords: Nonlinear analysis, optimization, linear analysis, quantum, structural walls

Introductión

Earthquakes are natural catastrophes that cause significant human and financial losses around the world, seismic events have always existed, this is known as seismic threat. [1]

Therefore, the reason why this design is carried out is to prevent the structures from being damaged when this type of natural disaster occurs. What is sought is to restrict lateral movement as much as possible, the most usual and convenient is to reduce drifts to a certain value, but to carry it out it is necessary to enhance a correct design of the structural elements [2], many times the design is optimal with beams and columns with dimensions or characteristics smaller than those usually used, This is due to the fact that it is usually designed with certain parameters of many local regulations, which are only analyzed from one perspective, since it is usually a little more laborious to design structural elements that meet a seismic criterion and at the same time are economical in their construction. [3] [4]

As we well know, an earthquake usually causes catastrophes, that is why structures in which they damage a structural element of great importance, the building usually fails suddenly[5], without having the ability to reassign loads, causing human losses, this is because not all seismic aspects are evaluated, and it tends to be thought that a structure of larger dimensions will always be more resistant, when in reality this is not always the case, which is generating economic losses only from the design. [6] [7]

Another important point for which a more complete structural analysis must be made is the issue of external forces such as wind, which although there is already an approach to this phenomenon such as dynamic response, the tools of the theory of associated random vibrations and their complete capabilities do not yet have a standardized framework. That is why a nonlinear analysis can predict these phenomena in a more accurate way, which allows for a more optimal analysis and design. [8] [9]

Nowadays in construction, new construction and design systems are sought to be implemented, in order to create innovative and ecological systems, without oversizing or polluting too much. [10]

In order for a structure to be considered earthquake safe, many local regulations generally require that the structure be analyzed only in the elastic range. However, these analyses do not consider the behavior when the structure is most severely damaged. A lot of information is lost and there is no real model of the building; then we will not be able to determine by linear analysis how much the elements flow, which ones should have the ductility increased, what is the loss of capacity of the structure and if the expected damage will be repairable, how safe the repaired structure is, what is the level of structural damage and what would be the level of expected accelerations.

Faced with this problem, ACI.318.19 [11] included for the first time in one of its appendices, guidelines for verifying the design, through performance evaluated through nonlinear analysis. Appendix A of ACI.318.19 highlights the need to include performance-based design procedures in common design practice, which so far are not found in many current local standards.

Currently, the guides and documents used to verify seismic performance are aimed at existing structures, such as ATC 40 [12], ASCE 41-17 [13], etc. While standards such as ASCE 7-16 or Appendix A of ACI 318-19 include the possibility of using alternative design procedures through extrapolated performance verification in their use for new buildings. [14] [15]

That is why this research seeks to carry out a more complete structural analysis, taking as an example a 7-story building, for which it will mainly seek to know how the amount of steel in the structural elements is optimized, by carrying out an analysis and design more accurate to the real, for this a linear analysis will be carried out, taking as a reference the local Peruvian regulation called E030 and a non-linear analysis according to the stipulations of ACI 318-19.

Materials and methods

For the structural analysis, the Etabs and SAP2000 program will be used. This research consists of two phases, in the first the seismic modeling and analysis and the design of the superstructure will be carried out according to Peruvian regulations E020, E.030 and E.060. [16] [17] [18]
The second phase consists of performing static nonlinear analysis, performance evaluation and optimization of steel quantity, and the designs will be compared according to Peruvian and American standards.

Procedure and results

Analysis and static linear design:

For the analysis, the masses of the modeled elements such as columns, walls, slabs and beams were estimated using the ETABS program. The loads were distributed by area in all the slab panels and linearly in the beams where there are partitions. The dead load was considered to be the self-weight of the elements (beams, columns, walls and solid slabs), the equivalent weight of the partitions for each slab panel, the weight of the finishes (100 kg/m2) and the weight of the lightened slabs (300 kg/m2). It is important to note that these loads were obtained from a load calculation and in accordance with the load standards of the National Building Regulations E.020.
Concrete with a compressive strength of f’c=210 kg/cm2 was used in the structural elements such as columns, beams and shear walls, in the foundation it was used as recommended by the soil mechanics study f'c=280 kg/cm2 and mezzanine slabs. In addition, grade 60 steel bars were used as reinforcement, whose yield strength is 4200 kg/cm2 according to the ASTM A-615 standard.

Mathematical model

The ETABS program was used to develop the three-dimensional mathematical model of the building. The beams and columns were designed as frame elements, while for the walls it was considered shell type and the lightened slabs were considered as membrane. The analysis is carried out using the stiffness matrix of each linear and area object, applying the finite element method.

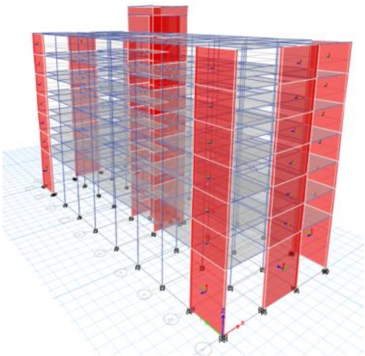


Figure 1: Three-dimensional mathematical model of the building in Etabs.

3.1.1 Seismic factors

The seismic zone for this project is zone 4, it corresponds to a maximum horizontal acceleration in the ground of 0.45, the soil parameters that result from the soil mechanics test yield a soil type S=2, intermediate soils, The use factor U=1, which corresponds to a category "C" building, because it is a multi-family building, the seismic amplification factor C depends on the period of the structure, the parameters Tp and Tl,

3.1.2 Analysis spectrum

In accordance with the following expressions of the seismic resistant standard:

$$T < T_p \qquad C = 2.5$$

$$T_p < T < T_L \qquad C = 2.5 * \left(\frac{T_p}{T}\right)$$

$$T > T_L \qquad C = 2.5 * \left(\frac{T_p * T_L}{T^2}\right)$$

The graph of the inelastic spectrum for both directions of analysis will be:

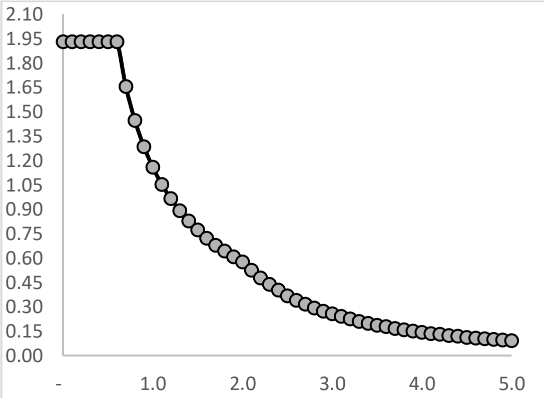


Figure 2: Three-dimensional mathematical model of the building in Etabs.

Modal Participation

In the seismic analysis, the full quadratic modal combination (CQC) method was employed [19]

to obtain the maximum elastic response expected both in the internal forces of the structural elements and in the overall parameters of the building, such as shear forces, absolute and relative displacements, among others. For the directional combination of seismic effects, the square root of the sum of squares (SRSS) method was used.

In each direction of analysis, vibration modes whose sum of effective masses represented at least 90% of the total mass as specified in the standard were taken into account. In addition, at least the first three predominant modes in the specific direction were considered, 21 vibration modes have been considered.

Mode	Period sec	UX	UY	RZ	SumRX	SumRY	SumRZ
1	0.567	0.57	0.00	0.79	0.00	0.28	0.00
2	0.529	0.53	0.78	0.00	0.00	0.28	0.28
3	0.494	0.45	0.00	0.00	0.78	0.28	0.28
4	0.138	0.1400	0.0001	0.1621	0.0001	0.8254	0.2849
5	0.122	0.1250	0.1633	0.0001	0.0002	0.8256	0.8193
6	0.12	0.1070	0.0002	0.0001	0.1641	0.8260	0.8199
7	6.30E-02	0.0620	0.0000	0.0356	0.0000	0.9421	0.8199
8	0.053	0.0530	0.0387	0.0000	0.0001	0.9421	0.9443
9	0.053	0.0470	0.0001	0.0000	0.0375	0.9422	0.9446

10	0.04	0.0380	0.0000	0.0097	0.0000	0.9809	0.9446
11	0.032	0.0310	0.0103	0.0000	0.0000	0.9809	0.9850
12	0.032	0.0290	0.0000	0.0007	0.0073	0.9838	0.9852
13	2.90E-02	0.0280	0.0000	0.0023	0.0019	0.9926	0.9852
14	0.024	0.0240	0.0000	0.0012	0.0000	0.9974	0.9852
15	0.023	0.0220	0.0028	0.0000	0.0000	0.9974	0.9957
16	0.023	0.0210	0.0000	0.0001	0.0000	0.9981	0.9957
17	0.022	0.0210	0.0000	0.0004	0.0000	0.9994	0.9957
18	0.019	0.0210	0.0000	0.0000	0.0024	0.9994	0.9958
19	0.018	0.0180	0.0008	0.0000	0.0000	0.9994	0.9990
20	1.70E-02	0.0170	0.0000	0.0000	0.0008	0.9994	0.9991
21	1.60E-02	0.0150	0.0002	0.0000	0.0000	0.9994	1.0000

Table 1: Modal analysis data extracted from Etabs

3.1.3 Structural system

It is a system of structural walls since the shear absorbed by the walls is greater than 70%, and the structure does not present irregularities.

	TOTAL BASAL SHEAR (V)	SHEAR THAT THE WALLS ABSORB (VM)	20% OF THE TOTAL SHEAR (0.2 V)	70% OF THE TOTAL SHEAR (0.7 V)	% THAT WALLS ABSORB	STRUCTURAL WALLS
SxE	865.75	840.51	173.15	606.025	97.08%	OK
SyE	815.27	793.58	163.054	570.689	97.34%	OK

Table 2: Building Structural System for Both Directions

3.1.4 Verification of lateral displacements or drifts

According to article 32 of the seismic resistant standard, the maximum mezzanine distortion for reinforced concrete structures is 0.007.

X Address					
Floor	**Elevation (m)**	**Drift x**	**Drift y**	**Drift Max**	**State**
Story7	22.8	**0.0052**	0.0004	0.007	**OK**

Story6	20	**0.0056**	0.0004	0.007	**OK**
Story5	17.2	**0.0057**	0.0004	0.007	**OK**
Story4	14.4	**0.0058**	0.0004	0.007	**OK**
Story3	11.6	**0.0055**	0.0004	0.007	**OK**
Story2	8.8	**0.0050**	0.0004	0.007	**OK**
Story1	6	**0.0026**	0.0002	0.007	**OK**
Y Address					
Floor	**Elevation (m)**	**Drift x**	**Drift y**	**Drift Max**	**State**
Story7	22.8	0.0007	**0.0054**	0.007	**CUMPLE**
Story6	20	0.0008	**0.0056**	0.007	**CUMPLE**
Story5	17.2	0.0008	**0.0058**	0.007	**CUMPLE**
Story4	14.4	0.0008	**0.0058**	0.007	**CUMPLE**
Story3	11.6	0.0008	**0.0056**	0.007	**CUMPLE**
Story2	8.8	0.0007	**0.0050**	0.007	**CUMPLE**
Story1	6	0.0004	**0.0027**	0.007	**CUMPLE**

Table 3: Drifts for both directions

3.1.5 Minimum shear force

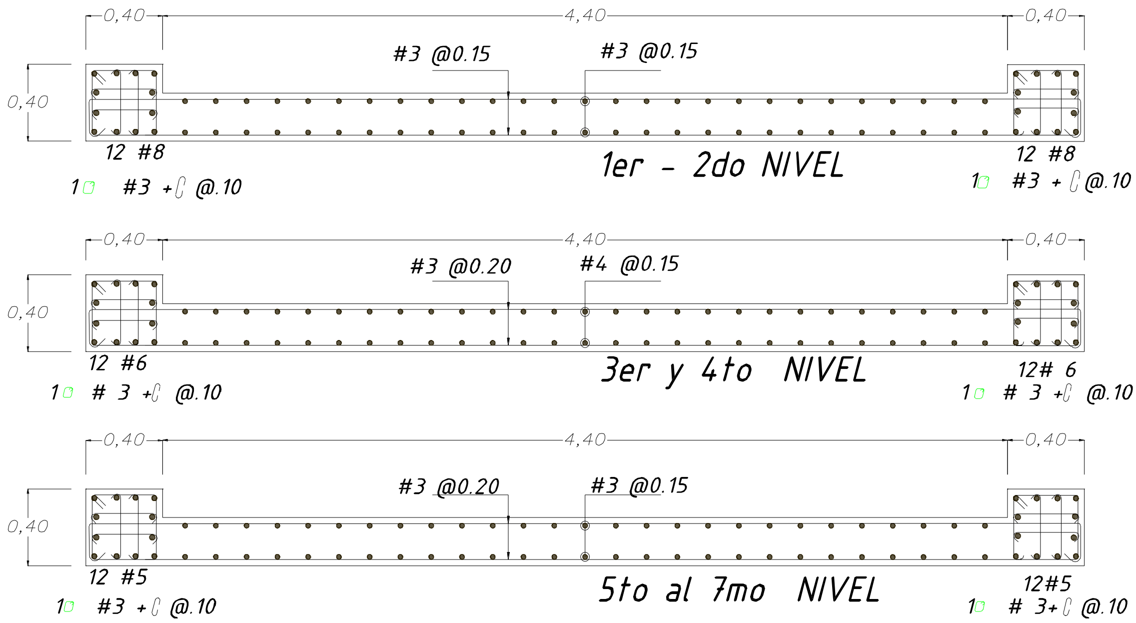
In each direction analyzed, a requirement is established for the shear force on the first mezzanine of the building. This value cannot be less than 80% of the calculation according to article 25 of the seismic resistance standard for regular structures.

X Address		**Y Address**	
Static Earthquake X	865.75 Tn	Static Earthquake Y	815.27 Tn
Dynamic Earthquake X	616.92 Tn	Dynamic Earthquake Y	619.46 Tn
80% Static Earthquake X	692.60 Tn	80% Static Earthquake Y	652.22 Tn
SCALE FACTOR	**1.1227**	SCALE FACTOR	**1.0529**

Table 4: Shear Force for Both Directions

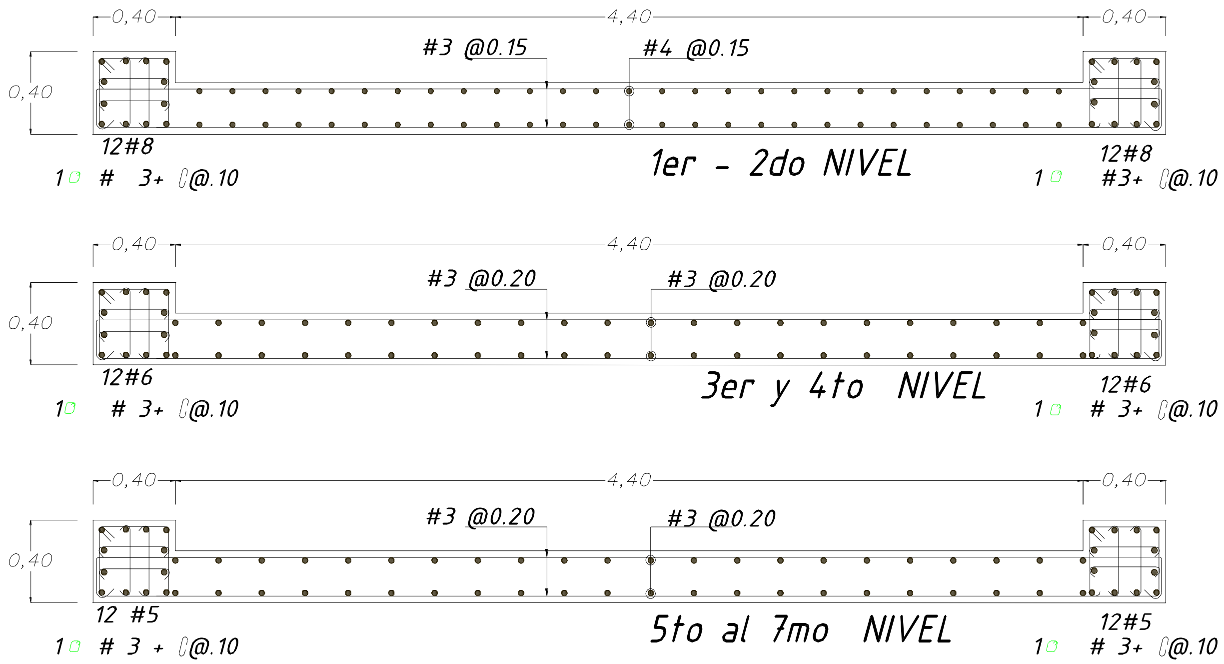
3.1.6 Wall design

For the design of structural walls, the guidelines of the E060 standard were followed.



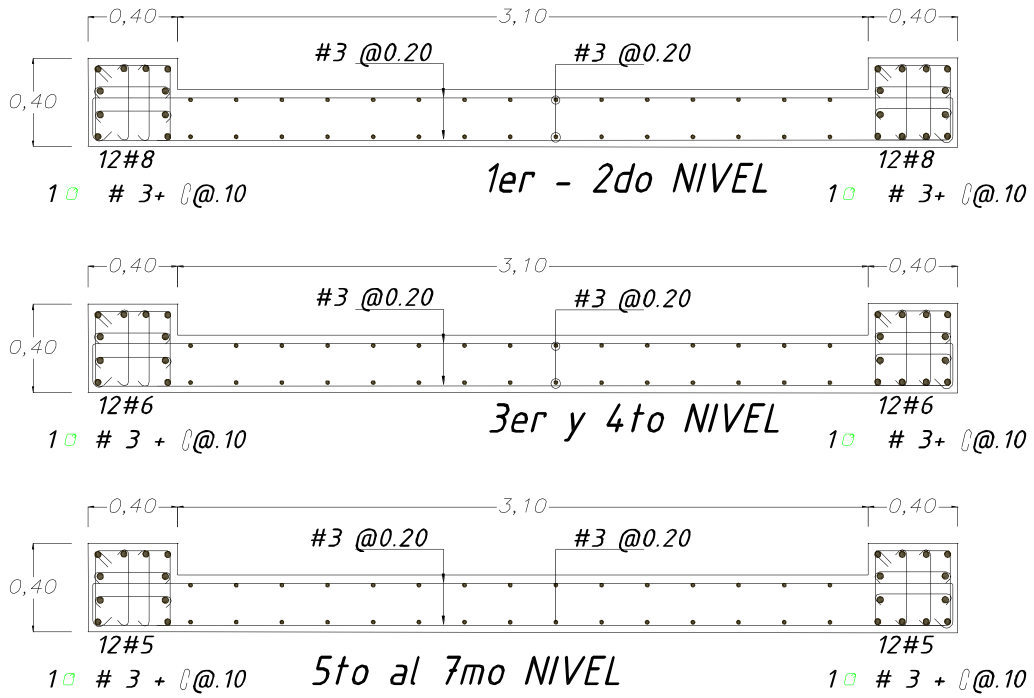
WALL	LEVEL	BORDER ELEMENTS		SOUL	
		BARS	ESIRRUPS	ERTICAL	HORIZONTAL
1 y 4	1 y 2	24 # 8	# 3 @ 0.1	# 5 @ 0.15	# 4 @ 0.15
	3 y 4	24 # 6	# 3 @ 0.1	# 4 @ 0.15	# 3 @ 0.15
	5,6,7	24 # 5	# 3 @ 0.1	# 3 @ 0.15	# 3 @ 0.20

Table 5: Reinforcement for wall 1 and 4



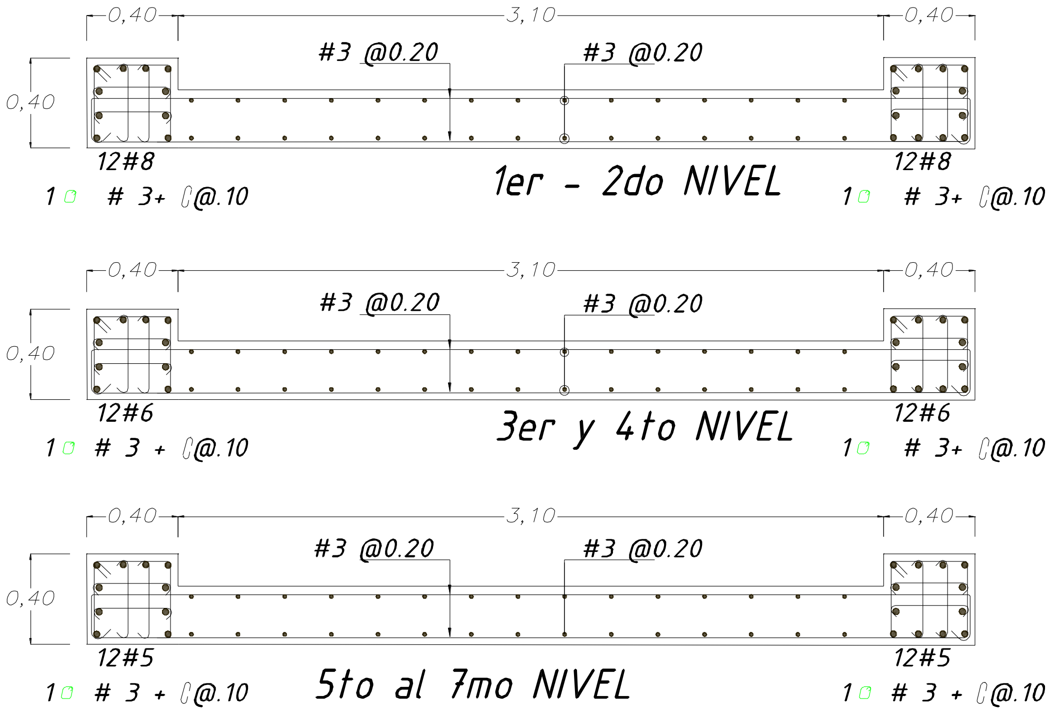
WALL	LEVEL	BORDER ELEMENTS		SOUL	
		BARS	ESIRRUPS	ERTICAL	HORIZONTAL
2 y 6	1 y 2	24 # 8	# 3 @ 0.1	# 4 @ 0.15	# 4 @ 0.15
	3 y 4	24 # 6	# 3 @ 0.1	# 3 @ 0.20	# 3 @ 0.20
	5,6,7	24 # 5	# 3 @ 0.1	# 3 @ 0.20	# 3 @ 0.20

Table 6: Reinforcement for wall 2 and 6



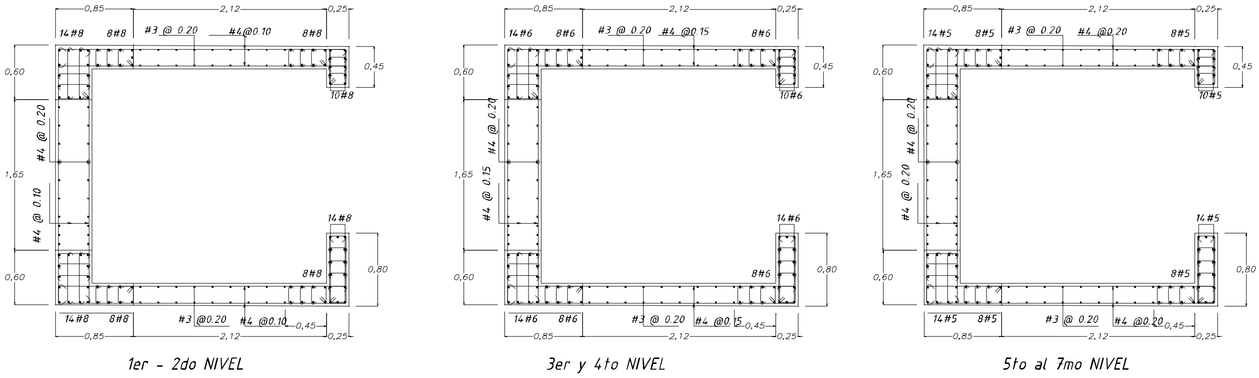
WALL	LEVEL	BORDER ELEMENTS		SOUL	
		BARS	ESIRRUPS	ERTICAL	HORIZONTAL
3 y 5	1	24 # 8	# 3 @ 0.1	# 3 @ 0.20	# 4 @ 0.15
	2	24 # 8	# 3 @ 0.1	# 3 @ 0.20	# 4 @ 0.20
	3 y 4	24 # 6	# 3 @ 0.1	# 3 @ 0.20	# 3 @ 0.20
	5,6,7	24 # 5	# 3 @ 0.1	# 3 @ 0.20	# 3 @ 0.20

Table 7: Reinforcement for wall 3 and 5



WALL	LEVEL	BORDER ELEMENTS		SOUL	
		BARS	ESIRRUPS	ERTICAL	HORIZONTAL
7 y 8	1	36 # 8	# 3 @ 0.2	# 3 @ 0.20	# 4 @ 0.15
	2	36 # 8	# 3 @ 0.2	# 3 @ 0.20	# 4 @ 0.20
	3 y 4	36 # 6	# 3 @ 0.2	# 3 @ 0.20	# 3 @ 0.20
	5,6,7	36 # 5	# 3 @ 0.2	# 3 @ 0.20	# 3 @ 0.20

Table 8: Reinforcement for wall 7 and 8



WALL	LEVEL	BORDER ELEMENTS		SOUL	
		BARS	ESIRRUPS	ERTICAL	HORIZONTAL
Asc	1 y 2	76 # 8	# 3 @ 0.2	# 3 @ 0.20	# 3 @ 0.10
	3 y 4	64 # 5	# 3 @ 0.2	# 3 @ 0.20	# 3 @ 0.15
	5,6,7	64 # 5	# 3 @ 0.2	# 3 @ 0.20	# 3 @ 0.15

Table 9: Reinforcement for elevator wall

Nonlinear Static Analysis – Pushover:

Unlike elastic analysis, this approach focuses on obtaining reliable information for the design, verifying the structure against displacement and curvature demands. Upon entering the nonlinear range, the structure can sustain damage without collapsing. Pushover analysis makes it possible to evaluate the structure's ability to resist seismic displacements and make more informed design decisions, ensuring the safety and durability of constructions in seismic zones.

Global evaluation of the structure

The objective of this research is focused on a standard building, therefore, the "Basic Objective" of performance has been chosen. For the overall assessment based on maximum deviations, the following proposal is proposed.

For the Service Earthquake (SLE), the maximum drift should be limited to 0.005.

For Design Earthquake (SD), the maximum drift should be limited to 0.015

For the Maximum Earthquake (MCE), the maximum drift should be limited to 0.025

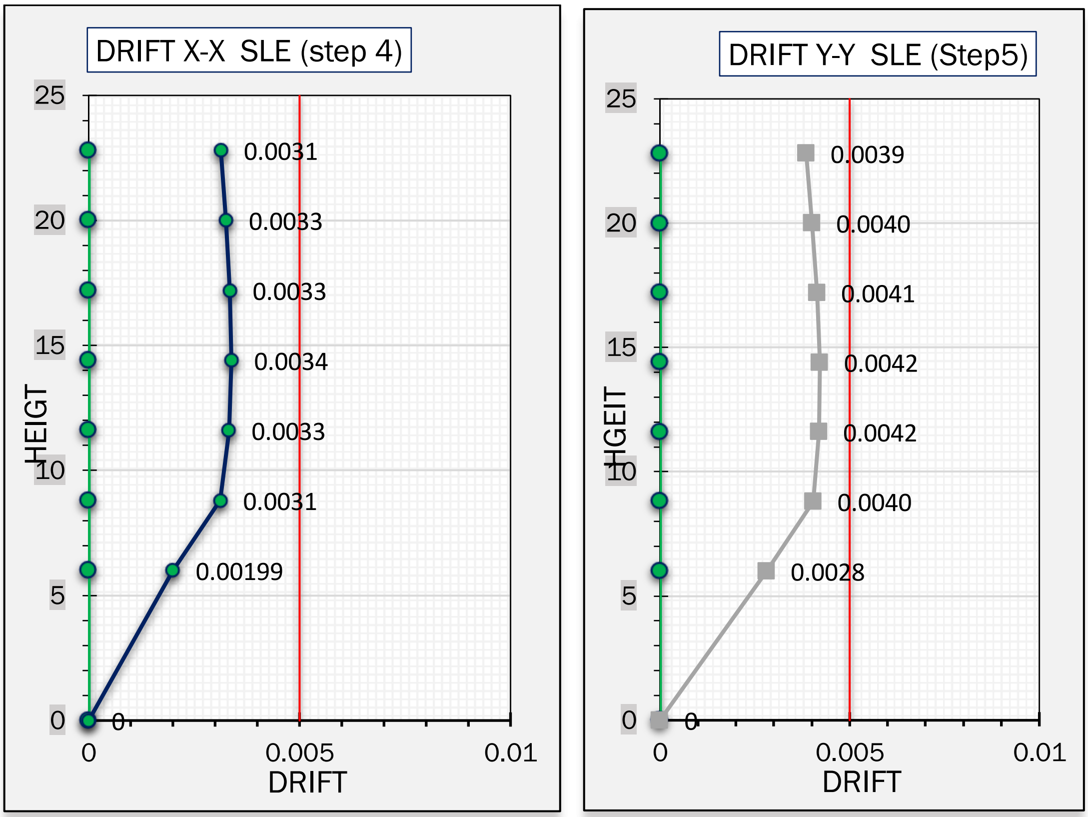


Figura 2: Derivas máximas SLE, dirección X-X y dirección Y-Y

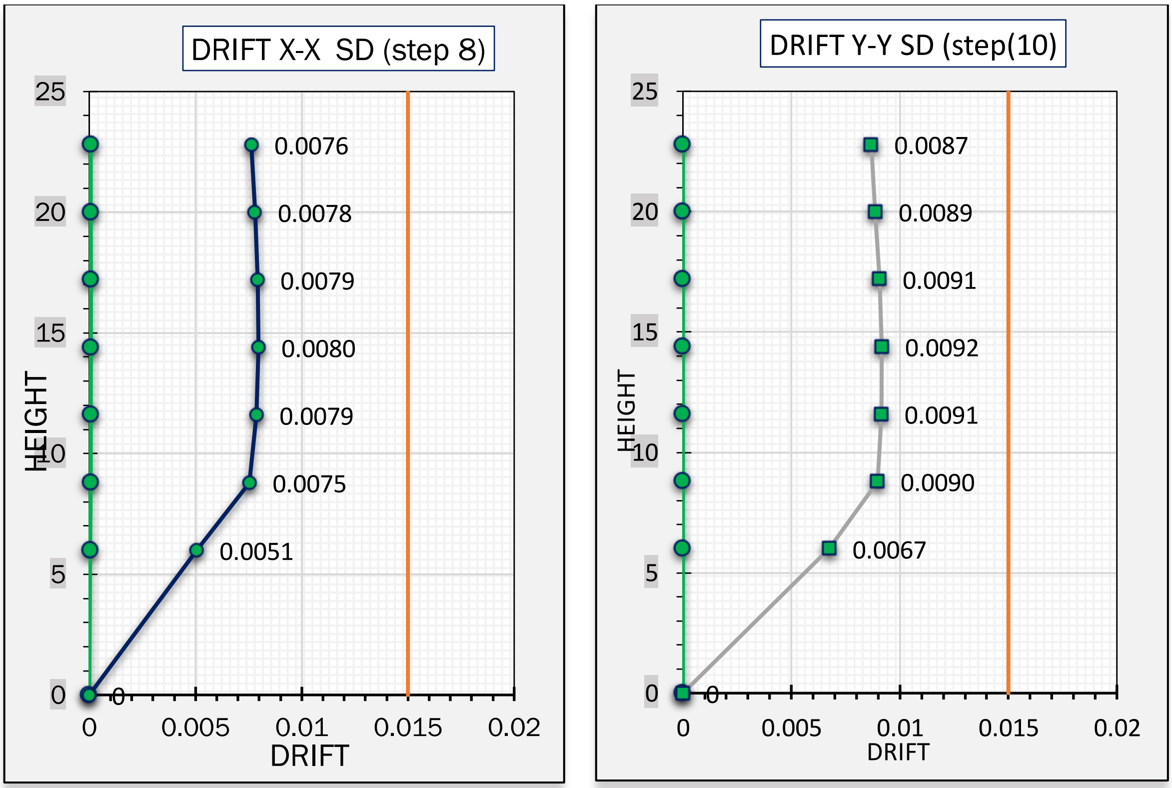


Figura 3: Derivas máximas SD, dirección X-X y dirección Y-Y

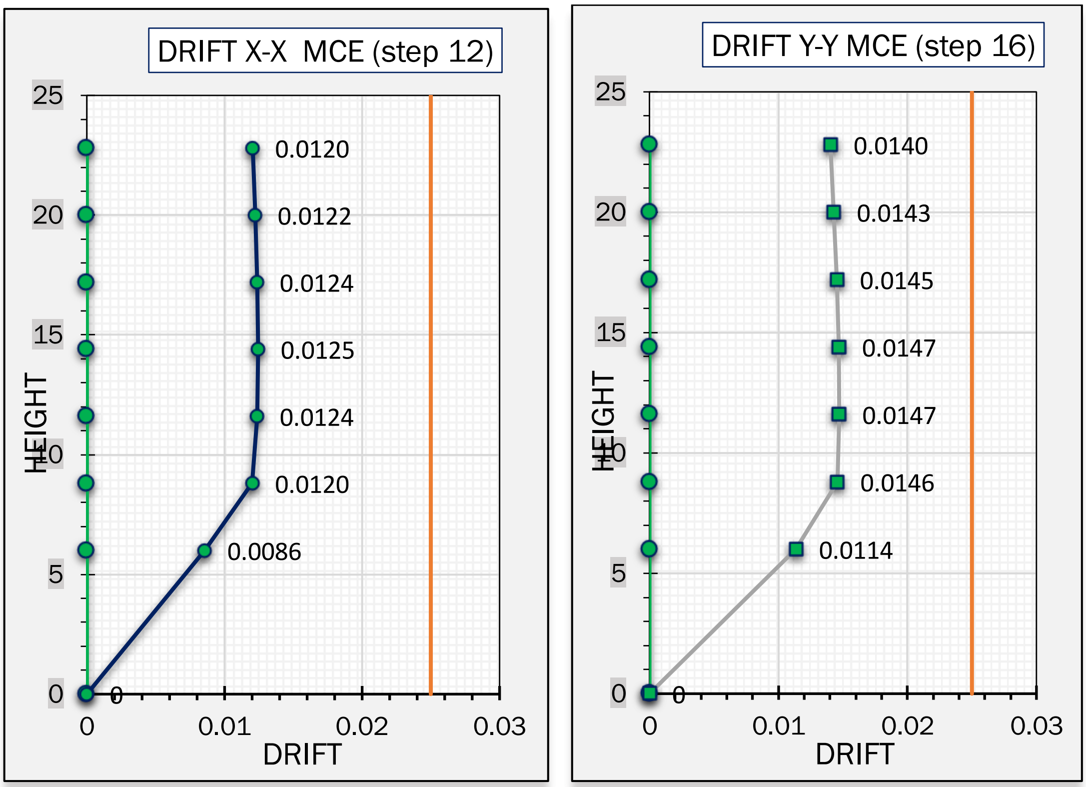


Figura 4: Derivas máximas MCE, dirección X-X y dirección Y-Y

3.2.1 Capacity curves

The capacity curve of the Pushover analysis as a function of the two-way mode for the X and Y directions.

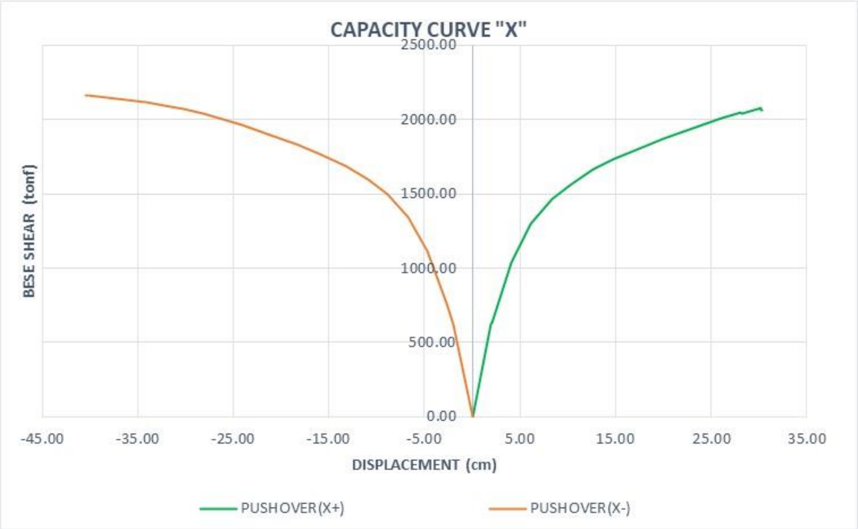


Figure 5: Capacity Curves, X-Direction

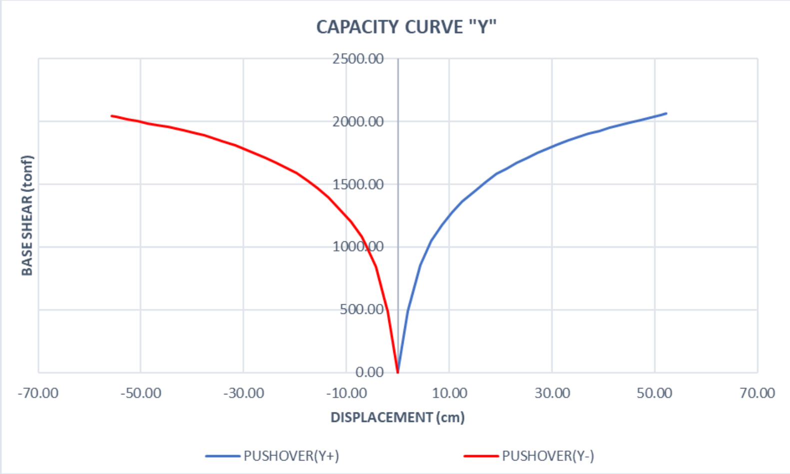


Figure 6: Capacity, Direction and Curves

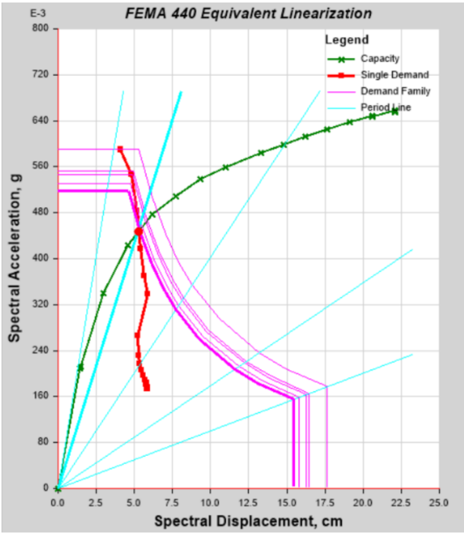
In the X direction as in the Y direction, the capacity curves tend to have the same influence for both types of load patterns. For direction X, as in Y, we will work with the capacity curve of the modal load pattern.

3.2.2 Seismic Performance Assessment

FEMA Proposal 440

SLE

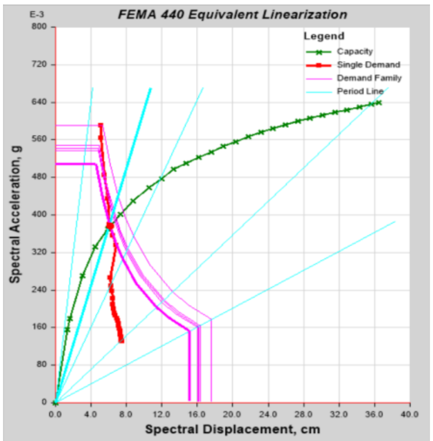
Direction X:



Sa (g)	0.4479
Sd (cm)	5.3101
V (tonf)	1375.1185
Performance Point (cm)	7.1384
T-blotting(s)	0.688
T effectivo (s)	0.746
Ductility	2.765776
Effective Damping	0.1418
Factor M	1.173649

Figure 7: SLE X Seismic Performance

Direction Y:

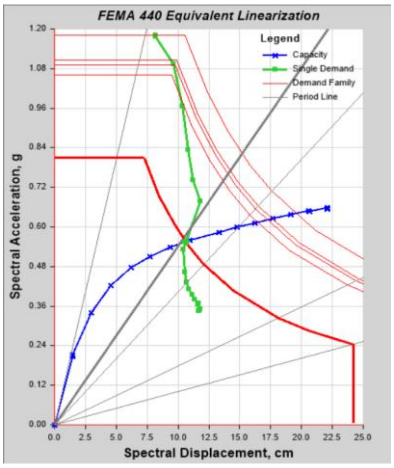


Sa (g)	0.4512
Sd (cm)	5.4343
V (tonf)	1387.5454
Performance Point (cm)	8.8637
T-blotting(s)	0.811
T effectivo (s)	0.891
Ductility	3.027261
Effective Damping	0.1593
Factor M	1.207082

Figure 8: SLE Y Seismic Performance

SD

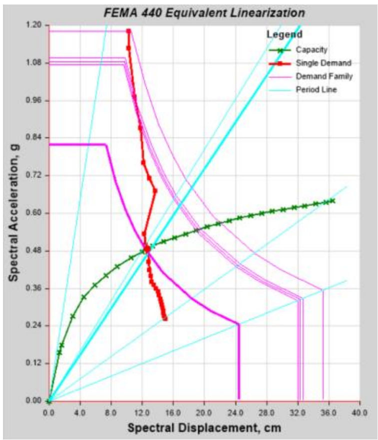
Direction X:



Sa (g)	0.554
Sd (cm)	10.635
V (tonf)	1719.63
Performance Point (cm)	14.34
T-blotting(s)	0.878
T effectivo (s)	0.899
Ductility	4.1303
Effective Damping	0.1988
Factor M	1.052093

Figure 9: SD X Seismic Performance

Direction Y

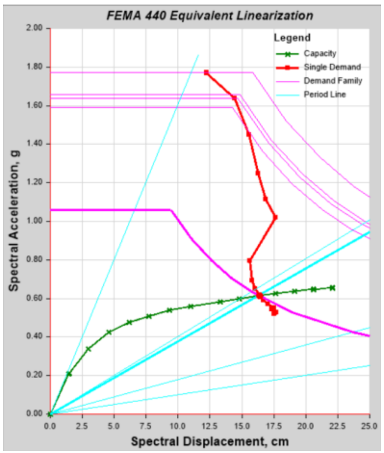


Sa (g)	0.487
Sd (cm)	12.6138
V (tonf)	1548.0904
Performance Point (cm)	18.0948
T-blotting(s)	1.02
T effectivo (s)	1.057
Ductility	4.871395
Effective Damping	0.2024
Factor M	1.072011

Figure 10: SD Y Seismic Performance

MCE

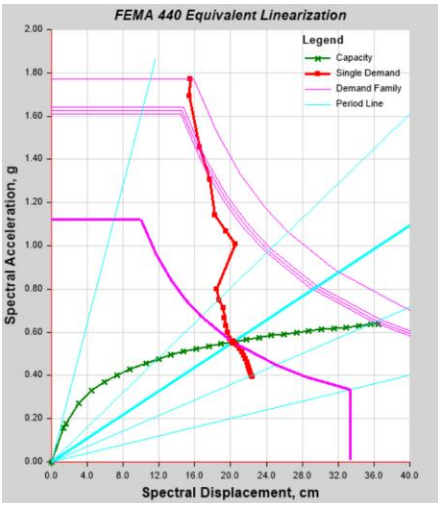
Direction X:



Sa (g)	0.614448
Sd (cm)	16.4112
V (tonf)	1925.6253
Performance Point (cm)	22.3118
T-blotting(s)	1.037
T effectivo (s)	0.993
Ductility	5.646235
Effective Damping	0.2049
Factor M	0.9178

Figure 11: MCE X Seismic Performance

Direction Y:



Sa (g)	0.55609
Sd (cm)	20.2742
V (tonf)	1782.532
Performance Point (cm)	29.0309
T-blotting(s)	1.211
T effectivo (s)	1.195
Ductility	6.659335
Effective Damping	0.2046
Factor M	0.97274

Figure 12: MCE Y Seismic Performance

Coefficient Method according to ASCE/SEI 41-13

Performance Point

Direction	earthquake	Step	Dt (cm)	V (tonf)
	SLE	4	6.1701	1301.6329
X-X	SD	8	14.0075	1708.897
	MCE	12	24.2202	1968.3565
	SLE	5	8.297	1162.1404
Y-Y	SD	10	18.6544	1565.5616
	MCE	16	30.4931	1807.6416

Table 10: Performance Point

Seismic Performance Assessment

X-X Direction

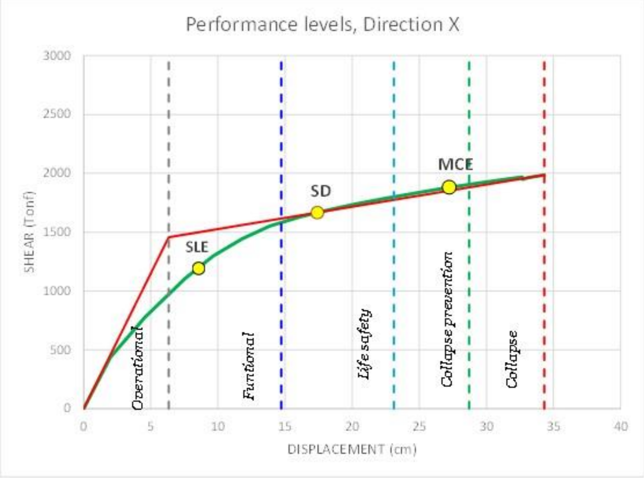


Figure 13: Seismic Performance Assessment direction X

Y-Y Direction

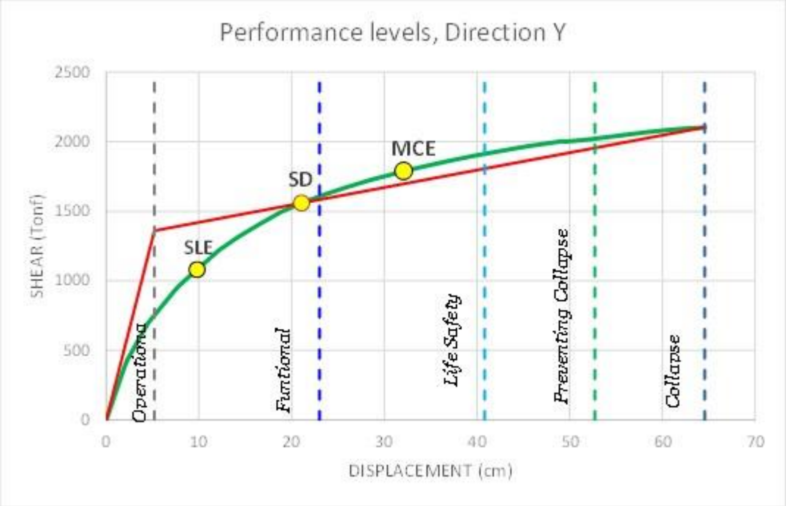


Figure 14: Seismic Performance Assessment Direction X

Performance objectives for service, design, and maximum earthquakes are met.

Direction	earthquake	Funtional	life safety	Collapse prevention	Does it meet performance objective?
X-X	Service				OK
	Design				OK
	Maximum				OK
Y-Y	Service				OK
	Design				OK
	Maximum				OK

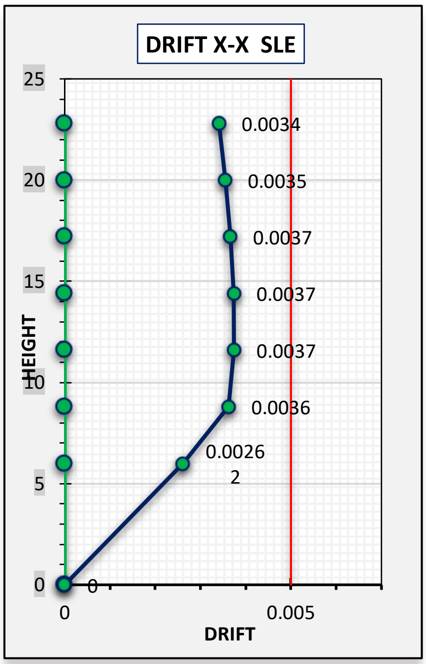
Table 11: service performance, design and maximum earthquakes.

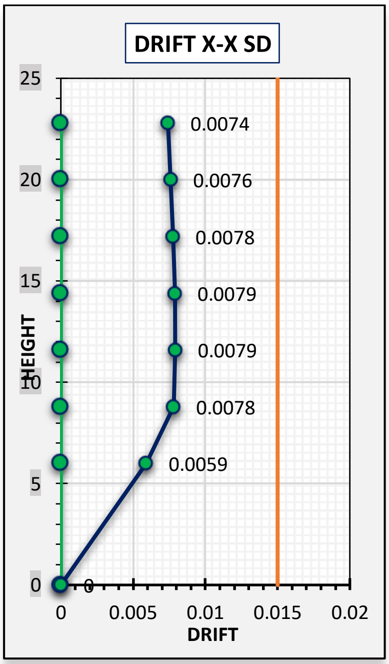
Reinforcement Optimization Process

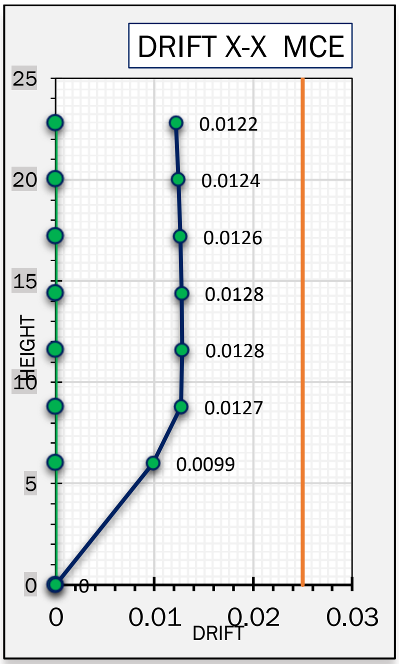
Performance Points for SLE, SD, and MCE

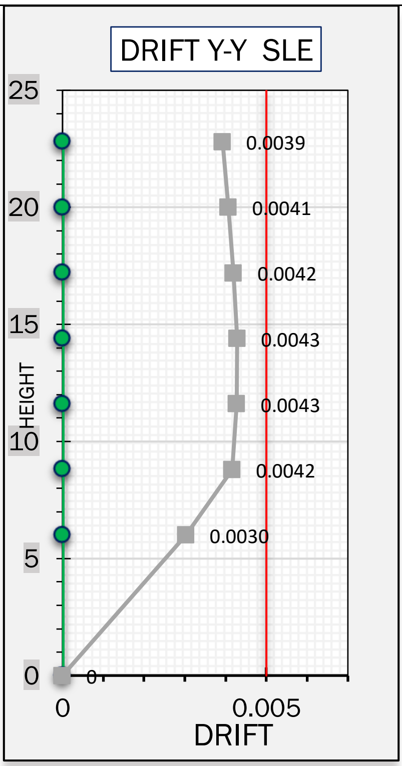
Address	Earthquake	Step	Dt (cm)	V (tonf)
X-X	SLE	4	6.4491	1179.28
	SD	12	**14.8123**	**1502.22**
	MCE	17	**25.9**	**1740.03**
Y-Y	SLE	11	8.4949	1080.71
	SD	17	**19.0639**	**1388.93**
	MCE	23	**31.7344**	**1615.26**

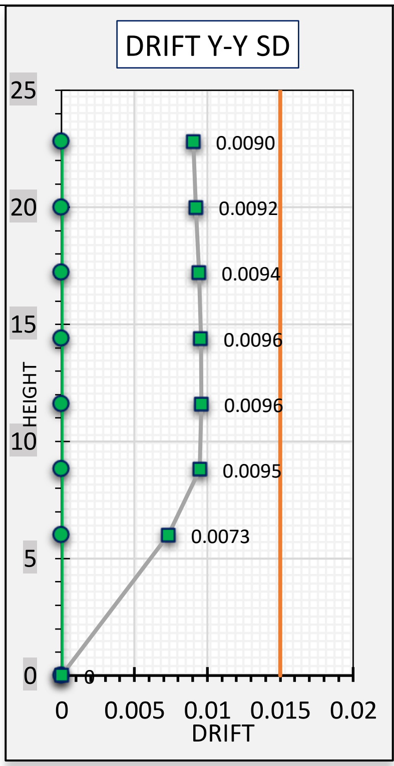
Maximum drifts

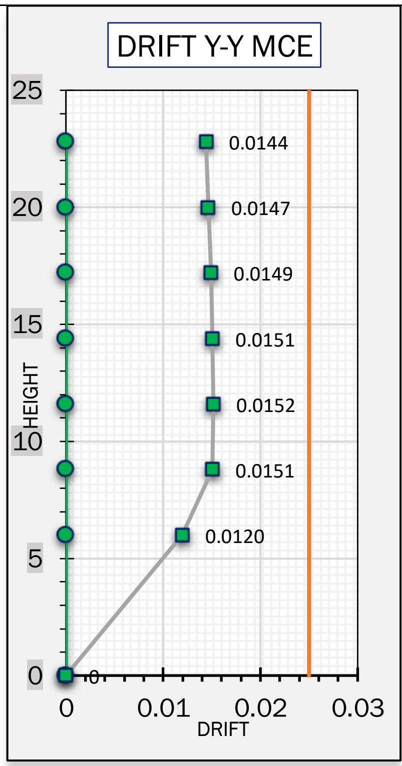












Performance appraisal

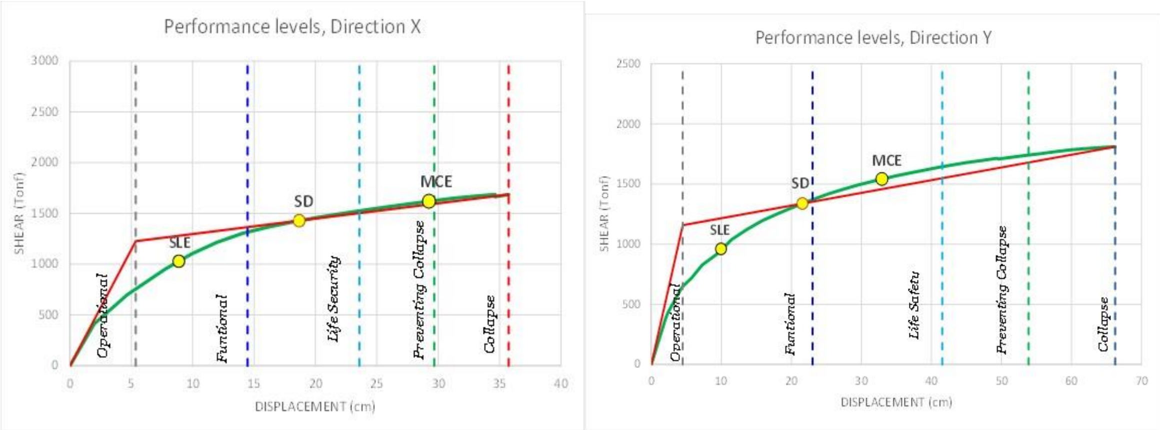


Figure 15: Seismic Performance Assessment Direction X and Y

By means of static nonlinear analysis, it has been possible to optimize walls 1 and 4 from the first to the seventh level, walls 2 and 6 of the first and second levels, walls 3 and 5 of the first and second levels and walls 7 and 8 of the first and second levels. For a maximum earthquake level considered, the unit deformation at the confined edges was less than 0.015 and 0.003 in walls with non-confined edges, for steel the maximum unit deformation of 0.05 is not exceeded.

The initial amount of vertical steel in the edge elements was 3.80%, with the optimization it was reduced to 2.69%, it was possible to reduce 1.11% for the first two floors. Thus, it is also observed that the maximum unit deformations of confined concrete are around 0.003 for a maximum earthquake level considered.

Conclusions

- The structuring, analysis and design of the structures was carried out according to the Peruvian design standard E030 and E060, obtaining a symmetrical and regular structure in plan and height, the system is of structural walls, the inelastic spectrum of pseudo accelerations has been obtained, the torsion is reduced by that fact the accidental eccentricity of mass has not been considered, It has been corroborated that the maximum mezzanine drifts do not exceed 0.007 for the mathematical model.

- According to ASCE 41-13, the performance point of the pushover analysis has been verified in the "X" direction, the performance point for an SLE is functional, for SD in life safety and for the MCE in collapse prevention being within the acceptable parameters according to the SEAOC Vision 2000 committee. For the "Y" direction, the performance point for an SLE and SD is at the functional level, and for the MCE in life safety, then the reinforcing steel can be optimized considering that the actions are controlled by deformation in structural walls.

- It should be taken into account in a pushover analysis that the actions in the structural walls are preferably controlled by deformation and not by shear force to obtain greater ductility and over resistance.

- It has been possible to optimize vertical steel in structural walls whose actions are controlled by deformation, since the demand-capacity of the actions controlled by shear force are close to unity, that is why it is not intended to optimize transverse steel.

- It has been possible to optimize the amounts of vertical reinforcement in the web and edge of the walls, M-2, M-3, M-5, M-6, M-7, M-8 in levels 1 and 2, in walls M-1 and M-4,

- The maximum drifts of the pushover analysis for service earthquakes are less than 0.005, 0.015 for design earthquakes and 0.025 for maximum earthquakes considered.

- Continued optimization is possible, but it is essential to recognize that loads and loads, including dead loads, live loads, and seismic loads, are probabilistic and not deterministic in nature. In other words, the design is made considering loads with safety factors that reflect the level of uncertainty present. It is important to remember that the goal of E030 is to reduce the risk of human losses, ensure the continuous operation of essential services, and minimize property damage

Recommendations

- It is suggested to employ a seismic performance-oriented design approach both in the construction of new buildings and in the evaluation of existing ones, when necessary. This methodology makes it possible to verify whether the structure complies with the design principles and the established performance objectives, according to the relevance and function of the building under evaluation. It also provides an estimate of the actual behavior that the structure would have at a given level of seismic demand. Additionally, it allows us to corroborate design assumptions such as ductility, additional resistance and the seismic force reduction factor, facilitating the implementation of corrections that may be necessary to ensure optimal performance against the action of earthquakes.

- It should be corroborated that the maximum unit deformation of unconfined concrete should be less than 0.003 and 0.015 for confined concrete, for reinforcing steel the unit deformation should not exceed 0.05 with respect to the maximum earthquake considered as indicated in Appendix A of ACI 318-19.

- To obtain the best results from the deformation-controlled actions on each panel (wall), they should be discretized at a height less than or equal to the length of the wall or considering Lw/2.

References

[1] M. Aghababaei y M. Mahsuli, "Detailed seismic risk analysis of buildings using structural reliability methods", Probabilistic Eng. Mech., vol. 53, pp. 23–38, 2018.

[2] S. Schnepf, L. Stempniewski, y D. Lungu, "Application of the capacity spectrum method for seismic evaluation of structures", Kit.edu, 2007. [En línea]. Disponible en: https://digbib.bibliothek.kit.edu/volltexte/beilagen/1/proceedings/pdf/53_Symposium_Bucharest_07_019_Schnepf.pdf. [Consultado: 14-oct-2023].

[3] J. . Arcila Zea, C. A. Riveros Jerez, and J. E. Rivero Jerez, "Optimization of beam and column sections for compliance drift of reinforced concrete buildings using Artificial Neural Networks", Rev.Fac.Ing.Univ.Antioquia, no. 70, pp. 34–44, Feb. 2014.

[4] Zou, X.-K., & Chan, C.-M. (2005). Optimal seismic performance-based design of reinforced concrete buildings using nonlinear pushover analysis. Engineering Structures, 27(8), 1289–1302. https://doi.org/10.1016/j.engstruct.2005.04.001

[5] NIST, «Recommended Modeling Parameters and Acceptance Criteria for Nonlinear Analysis in Support of Seismic Evaluation, Retrofit, and Design,» U.S. Department of Commerce Engineering Laboratory National Institute of Standards and Technology Gaithersburg, MD 20899-8600, Vols. %1 de %2Recuperado: https://doi.org/10.6028/NIST.GCR.17-917-45, pp. 203-208, 2017.

[6] A. Rajoria y R. Nagar, "'Progressive collapse analysis of flat slab building in hilly terrain using Non-Linear Analysis: A study on bracing system as a structural Remedy'", Structures, vol. 54, pp. 825–836, 2023.

[7] A. Di Cesare et al., "Identification of the structural model and analysis of the global seismic behaviour of a RC damaged building", Soil Dyn. Earthq. Eng., vol. 65, pp. 131–141, 2014.

[8] M. Iancovici, G. Ionică, F. Pavel, F. Moța, y G. B. Nica, "Nonlinear dynamic response analysis of buildings for wind loads. A new frontier in the structural wind engineering", J. Build. Eng., vol. 47, núm. 103708, p. 103708, 2022.

[9] Z. Ouyang y S. M. J. Spence, "Performance-based wind-induced structural and envelope damage assessment of engineered buildings through nonlinear dynamic analysis", J. Wind Eng. Ind. Aerodyn., vol. 208, núm. 104452, p. 104452, 2021.

[10] C. A. Gaviria M., A. P. Guerrero Z., and P. Thomson R., "A nonlinear finite element model of lightweight walls with cold formed steel members under lateral load", Rev.Fac.Ing.Univ.Antioquia, no. 69, pp. 136–151, Jan. 2014.

[11] ACI 318-19, Requisitos de reglamento para concreto estructural, 2019.

[12] ATC40, Seismic evaluation and retrofit of concrete buildings, California, 1996.

[13] ASCE 41-17, Seismic Evaluation and Retrofit of Existing Buildings, Reston, Virginia, 2017.

[14] ASCE 7-16, Minimum Design Loads and Associated Criteria for Buildings and other Structures, Reston, Virginia, 2017.

[15] Tall Buildings Intiative (TBI), Guidelines for Performance-Based Seismic Design of tall Buildings, California, 2017.

[16] Ministerio de Vivienda, «NTE E.020,» Lima, 2018.

[17] Ministerio de Vivienda, «NTE E.030,» Lima, 2018.

[18] Ministerio de Vivienda, «NTE E.060,» Lima, 2018.

[19] Valdés González, J., & Ordaz Schroeder, M. análisis de efectos sísmicos ortogonales horizontales en terreno blando. Revista de Ingeniería Sísmica, no. 79,pp 91-111. 2008.