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EVALUATION OF THE SEISMIC PERFORMANCE OF RC DUAL SYSTEM BUILDINGS USING THE PERUVIAN SEISMIC CODE E.030-2018

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Abstract

In recent years, the real estate demand of medium and high-rise buildings in Peru have been increasing in different regions of the country, especially in the city of Lima. These buildings, mostly of reinforced concrete and dual system, at past seismic events have presented different behaviors and failures than those assumed in the design stage. This is due to the methodology that has been applying does not contemplate the performance of the buildings in a nonlinear range. Hence, it is necessary to analyze the buildings in a nonlinear range in order to determine their structural performance. Furthermore, with a determined seismic demand, obtain their damage state and quantify the damage of the structural elements. In this paper, three dual system buildings of reinforced concrete of 6, 10 and 15 stories with similar characteristics and geometric configurations are evaluated in a nonlinear range in order to obtain their structural performances. First, through the methodology of "Resistance Design" it is obtained the design of the structural elements of the buildings. Then, the performance points are calculated through a nonlinear analysis following the methodology of "Performance Based Design". The capacity of the buildings is calculated applying a static nonlinear analysis (Pushover Analysis). The elastic demand spectrum is obtained according to the requirements of the peruvian seismic code E.030 and the inelastic demand spectrum is obtained according to ATC-40. Once the capacity and demand spectrum are defined, the performance points are obtained based on the methodology proposed by ATC-40. The structural performances of the buildings associated to a damage state are obtained according to the performance objectives proposed by Vision 2000. As a result, for a design earthquake demand, the buildings present a damage state of life safety (LS), which represents an adequate design and performance of the buildings in accordance with the requirements indicated in the peruvian seismic code E.030.

Keywords: structural performance, nonlinear analysis, reinforced concrete, dual system.

1. Introduction

Several seismic-resistant codes, including the Peruvian code E.030 [1], have as their design philosophy to avoid the collapse of structures during high-intensity earthquakes, as well as to ensure that they do not present significant damage during moderate earthquakes. However, the behavior of the structures during earthquakes of different characteristics suggests that these objectives have not been satisfactorily reached.

Based on that, the concept of "Performance Based Design" arises, which qualifies the performance of a building, based on the ideal performance that a building should have according to its importance and behavior during diverse seismic events.

The present paper focuses on evaluating the performance through a static non-linear analysis of dual system reinforced concrete structures under the seismic solicitations (serviceability, design and maximum earthquake), defined according to ATC-40 [2].

In the evaluation of the performance, the results obtained are compared with the design philosophy of the Peruvian code, which allows the validation of the resistance design based on the Peruvian code for a determined seismic solicitation (design earthquake).

2. Theoretical Framework

2.1 Performance based design

The performance-based design of the structure under earthquake conditions consists of the selection of appropriate assessment schemes that allow the dimensioning and detailing of the structural and nonstructural components, so that, for different levels of determined ground movements and with certain levels of reliability, the damage to the structure should not exceed certain limit states [2].

2.2 Structure performance levels

A performance level is an expression of the maximum damage to a building for a specific design level of earthquake. VISION 2000 [3] defined five levels of performance, each level defines the limit for a range of damage, which meets the basic needs of the user such as continuity of function, condition for repair, safety, etc.

The structural performance levels correspond to defined sectors of the structure's capacity curve. To sectorize the capacity curve, the effective yield displacement (Δe) and the inelastic displacement capacity (Δp) must first be defined. The effective yield displacement (Δe) corresponds to the instant in which a maximum of 50% of the inelastic incursions forming the failure mechanism have occurred, without the deformation in any section exceeding 150% of its yield deformation. The inelastic displacement capacity (Δp) corresponds to the lateral displacement of the structure from the effective yield point to the collapse. The inelastic section of the capacity curve is divided into four sectors defined by fractions of the inelastic displacement (Δp). [3]

As shown in Fig. 1, the performance levels are: Operational (DS1), Functional (DS2), Life Safety (DS3), Near Collapse (DS4) and Collapse (DS5).

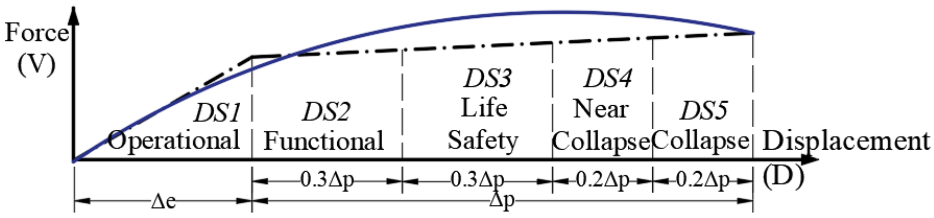

Fig. 1 – Division of the capacity curve

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2.3 Structural performance objectives

The structural performance objectives correspond to expressions of coupling between the required performance levels for a structure and the expected level of seismic motion.

VISION 2000 [3] classifies structures into three major groups according to their importance during and after an earthquake, and for each group a set of performance objectives is stablished for different level of seismic motion, as is shown in Table 1.

2.4 Structural capacity

In seismic engineering the capacity of a structure to resist seismic action is represented by a curve, which is defined by shear force at the base, acting on the structure as a function of the horizontal displacement at the top of the structure.

2.4.1 Non-linear static analysis

The procedure of the non-linear static analysis (Pushover analysis), consists of elaborating a mathematical model of a structure, initially without plastic hinges, which is exposed to lateral forces that act at floor level until some elements reach their elastic limit, then the structure is modified to take into account the reduced resistance of elements where plastic hinges have been produced. A distribution of lateral forces is again applied until additional elements produce plastic hinges. This process is continued until the structure becomes unstable or until a predetermined limit is reached. [2]

Table 1 – Performance objectives

	Operational	Functional	Life Safety	Near Collapse	Collapse
Serviceability Earthquake (SE)	*2*	*1*	*0*	*0*	*0*
Design Earthquake (DE)	*3*	*2*	*1*	*0*	*0*
Maximum Earthquake (ME)	-	*3*	*2*	*1*	*0*

0: Unacceptable performance
1: Basic structures
2: Essential/risky structures
3: Safety-critical structures

2.4.2 Capacity spectrum

The capacity curve is converted to a capacity spectrum, so that the base shear force (V) is transformed into spectral acceleration (Sa) and the displacement at the top floor (Δ_{roof}), into spectral displacement (Sd). This is obtained using the dynamic properties of the structure, as described below.

With the modal mass coefficient of the first mode (α_1), the modal participation factor of the first mode (PF_1), the modal shape coefficient of level i in mode 1 ($\phi_{i,1}$), and the weight of the building (W), the capacity spectrum is obtained with the following expressions:

$$S_d = \frac{\Delta_{roof}}{PF_1 * \phi_{roof,1}} \tag{1}$$

$$S_a = \frac{V}{W * \alpha_1} \tag{2}$$

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2.5 Seismic demand

ATC-40 [2] defines 3 levels of earthquake: Serviceability Earthquake (SE), Design Earthquake (DE) and Maximum Earthquake (ME). These are related as follows: SE = 0.5 * DE and ME = 1.25*DE. Generally, the design earthquake is defined as the spectrum of elastic demand presented by most of the earthquake-resistant codes. The seismic demand spectrum is obtained from calculating the spectral displacement (Sd) in relation with the spectral acceleration (Sa) and the period (T), as the following expression:

$$S_d = \frac{1}{4*\pi^2} * S_a * T^2 \tag{3}$$

2.6 Performance point

The performance point is obtained using B Method proposed by ATC 40 [2]. The method consists of: considering constant the relation between the yield point (a_y,d_y) and the end point (a',d') obtained from the bilinear representation of the capability spectrum, which determines the initial point (a_p, d_p). Calculate the damping coefficient of the demand spectrum (β_{eff}) with the displacement d_p. Generate a curve for several values of d_p, with the starting point and the damping. Intercept the capacity spectrum with the generated curve to obtain the performance point.

3. Description of the structures

The study analyzes 3 dual system reinforced concrete buildings of 6, 10 and 15 stories, which present similar characteristics and geometric configurations. The buildings are designed as office and are located in the city of Lima – Peru, corresponding to an intermediate soil type S2 according to Peruvian code E.030 [1].

The plan configuration is typical for the 3 buildings as shown in Fig. 2. The height of the first story is 3.60 m and the upper stories are 3.00 m.

The properties of the materials considered are:

- Concrete: Compressive strength of concrete (f'c) = 28 MPa, Modulus of elasticity (Ec) = $2.5x10^4$ MPa
- Steel Rebar: Yield strength (fy) = 420 MPa, Modulus of elasticity (Es) = $2x10^5$ MPa.

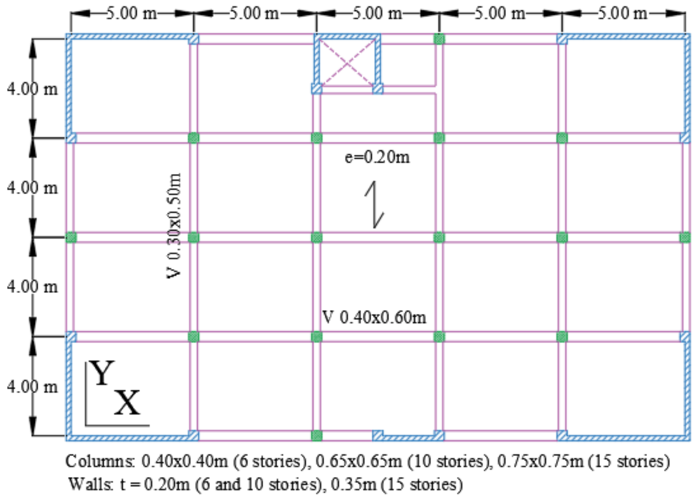

Fig. 2 – Floor plan configuration of the buildings

4. Seismic analysis

The seismic analysis of the 3 buildings is performed using the software ETABS v.2015. The beams and columns are modeled as frame elements, the walls are modelled as shell elements and the slabs are modelled as membrane elements.

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The buildings are analyzed in both X and Y directions using the pseudo inelastic or design acceleration spectrum as indicated in E.030 [1] as shown in Fig. 3, with the expression:

$$Sa = \frac{Z*U*C*S}{R}*g \tag{4}$$

Where Z is the zone factor (Z = 0.45), U is the use factor (U = 1), C is the seismic amplification factor, S is the soil factor (S = 1.05), R is the coefficient of reduction of seismic forces (R = 7) and g is the value of gravity (g = 9.81 m/s^2).

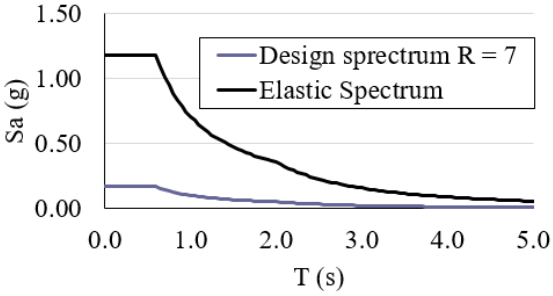

Fig. 3 – Design and elastic pseudo-acceleration spectrum

From the analysis, it is obtained that for the 3 buildings, the sum of effective masses is more than 90%. Also, it is presented that the first two modes are translational and the third mode is rotational.

The drifts, shown in Fig. 4, have been obtained from the seismic analysis multiplied by 0.75*R (regular structures), being these less than the maximum inter-story drift allowed of 0.007 (reinforced concrete) according to E.030 [1].

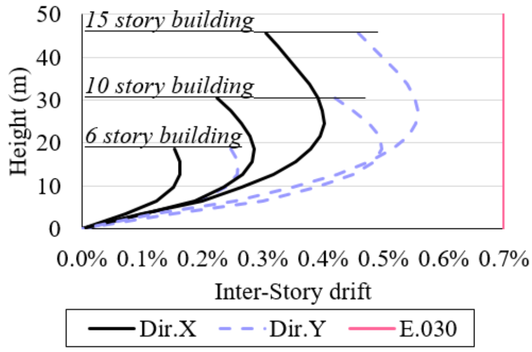

Fig. 4 – Drifts of the buildings

5. Structural properties of the elements

5.1 Design of the structural elements

The resistance design of the structural elements of the buildings is carried out according to Peruvian code E.060 [4]. The elements are designed to obtain in all sections design resistances (ϕRn) at least equal to the required resistances (Ru), calculated for the loads and forces amplified in the combinations stipulated in E.060 [4]. All sections of the structural elements comply with: ϕRn ≥ Ru

5.2 Non-linearity of structural elements

5.2.1 Beams and columns (frame elements)

For the sections at the ends of the beams, the moment - curvature diagrams are obtained for both positive and negative moments, according to ASCE 41-13 [5]. As a demonstration, the moment - curvature diagram for the beam 0.40x0.60m of the 6-story building is shown in Fig. 5.

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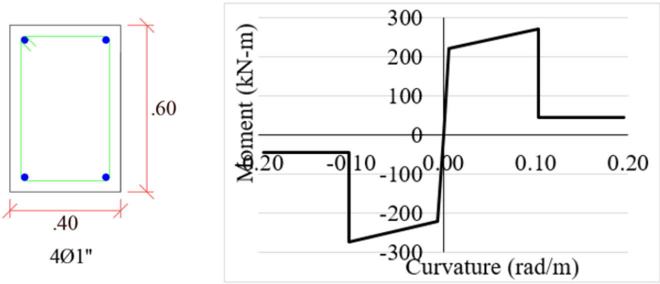


Fig. 5 – Moment – curvature diagram of the beam 0.40x0.60, 6-story building

For the definition of the moment - curvature diagrams of columns, the axial load is considered as the combination of the dead load plus the live load (service loads). Also, the moment - curvature diagram is calculated for one direction, since the column reinforcements are symmetrical. As a demonstration, the moment - curvature diagram for the column 0.40x0.40m of the 6-story building is shown in Fig. 6.

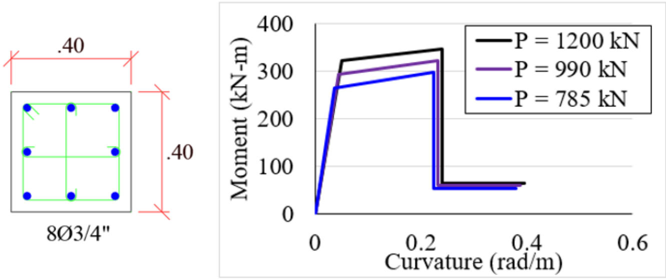
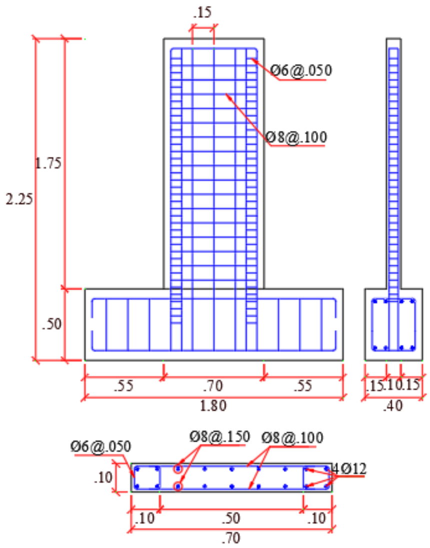


Fig. 6 – Moment – curvature diagram of the column 0.40x0.40, 6-story building

5.2.2 Walls (shell elements)

For the non-linearity of the walls, the moment - curvature diagram generated by the software ETABS v.2015 through the assignation of the plastic hinge at the middle of the section is considered. Because the walls, due to their slenderness, only work in bending. In order to validate the simulations of this study, test results obtained from Dae-Han [6] were used.

Fig. 7.a shows the detail of the geometry and the reinforcement of the wall used by Dae-Han [6], and Fig. 7.b shows the model and the location of the plastic hinge defined with the software ETABS v.2015.



(a) Geometry and reinforcement

(b) Non linear model

Fig. 7 – Characteristics of the wall for validation

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Fig. 8 shows the results obtained from the test and the analytical method, both performed by Dae-Han [6], and the result obtained of the non-linear static analysis with ETABS. v.2015. It is observed that the results generated by the software are reliable.

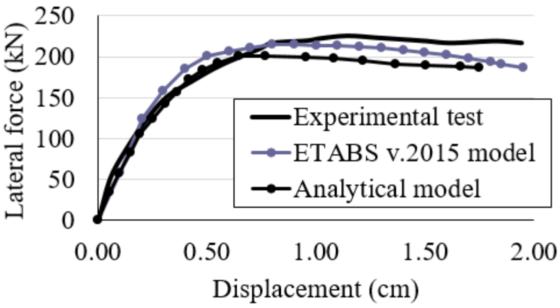


Fig. 8 – Comparison of results of Dae-Han [6] and numerical simulation in this study

6. Structural capacity of the structures

6.1 Structural modeling

For non-linear static analysis, beams and columns are modeled with a plastic hinge at the ends, flexural for the beams and flexo-compression for the columns. For the walls, plastic hinge by flexo-compression is located at the middle of the wall. The analysis is performed using the software ETABS v.2015 and the typical model of the structure is shown in Fig. 9.

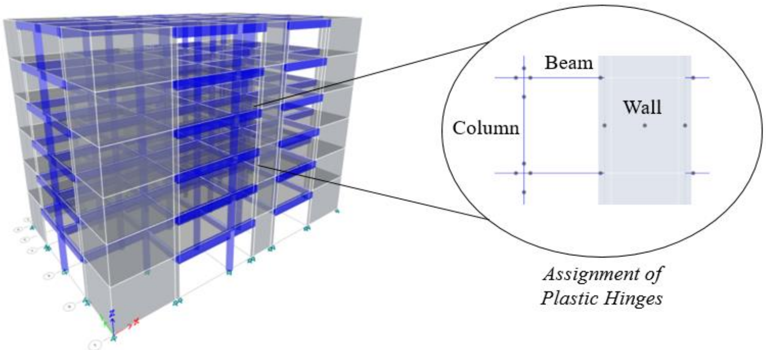
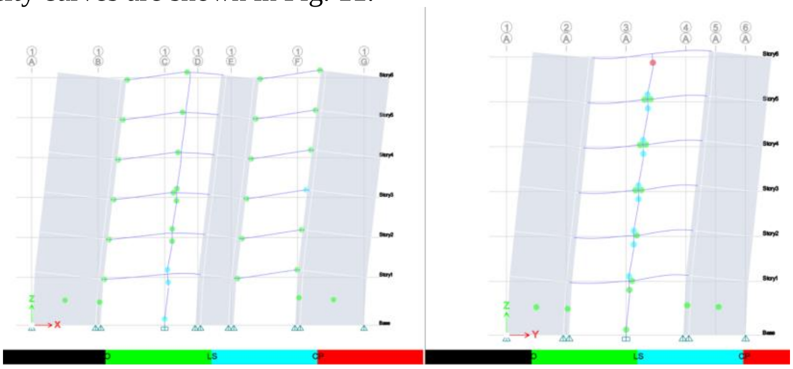


Fig. 9 – Structural modeling of the buildings

6.2 Non-linear static analysis

The non-linear static analysis (Pushover analysis) is performed in both directions (X and Y) for the 3 buildings, following the methodology described in item 2.4.1.

From the analysis, the plastic hinges formation associated with the failure shape of each building is shown in Fig. 10 and the capacity curves are shown in Fig. 11.



(a) 6 story building: *Dir. X* *Dir. Y*

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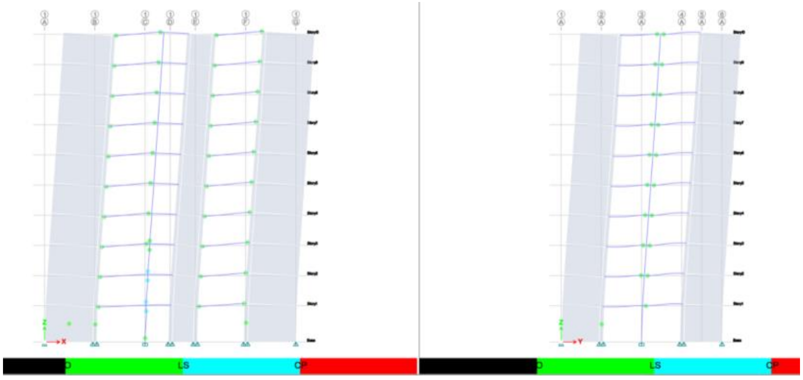
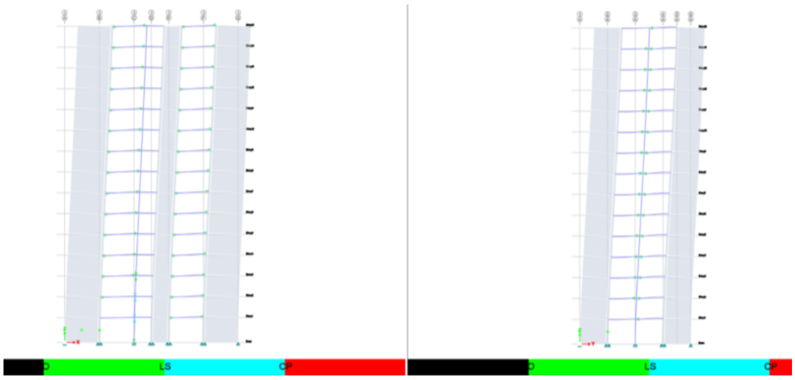


(b) 10 story building: Dir. X Dir. Y



(c) 15 story building: Dir. X Dir. Y

Fig. 10 – Failure shape of the buildings

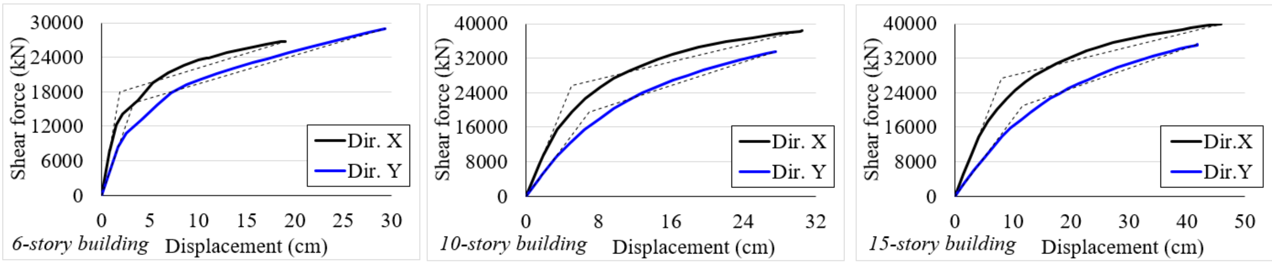


Fig. 11 – Capacity curve of the buildings

Applying Eq. 1 and Eq. 2, it is obtained the capacity spectrum curves of the buildings. As a demonstration, it is shown in Table 2 and Fig. 12 the calculation of the capacity spectrum curve for the 6-story building in the X-direction.

Table 2 – Dynamic properties of the 6-story building, X-direction

Story	W_i (kN)	W_i/g (kN*s^2/m)	Φ_{i1}
6	2795	284.9	1.00
5	3194	325.6	0.83
4	3194	325.6	0.64
3	3194	325.6	0.44
2	3194	325.6	0.25
1	3213	327.5	0.10
	18784	1914.8	
		PF_1 =	1.40
		α_1 =	0.75

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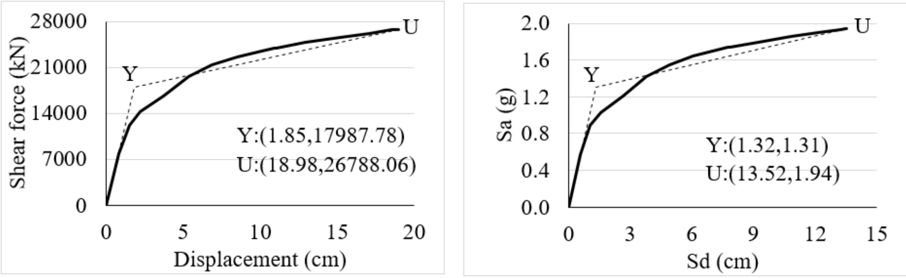

Fig. 12 – Capacity spectrum curve of the 6-story building, X-direction

7. Seismic performance evaluation

7.1 Seismic demand

The evaluation of the performance of the buildings is carried out for 3 levels of seismic movements, described in item 2.5. The seismic demand corresponding to the design earthquake (DE) is obtained considering the parameters proposed by E.030 [1], which are indicated in item 4.

Applying Eq. 3, the demand spectrum is obtained, which is shown in Fig. 13.

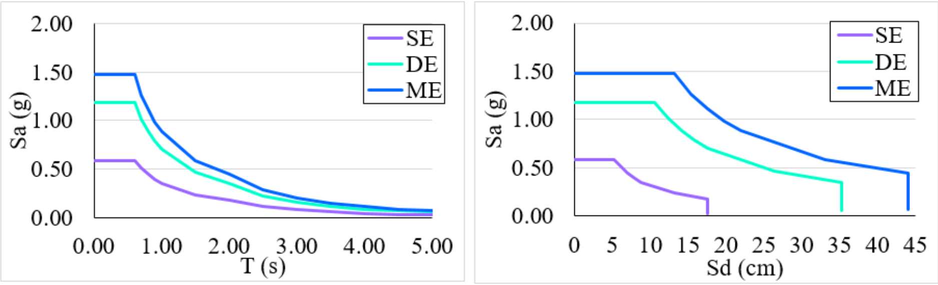

Fig. 13 – Demand spectrum curve of the defined earthquakes

7.2 Performance point

The performance point for the 3 buildings and for the 3 defined earthquakes is determined following the B Method developed by ATC-40 [2] described in item 2.6.

As a demonstration, it is shown in Table 3 and Fig. 14 the application of the method to obtain the performance point for the X-direction of the 6-story building and for the Design Earthquake (DE).

Table 3 – Procedure to determine the performance point (PP)

Sd (cm)	a_{pi} (g)	βo (%)	k	β_{eff} (%)	SR_A	SR_V	Sa (g)
1.35	1.31	1.42	0.67	5.95	0.94	0.96	1.11
1.50	1.31	7.26	0.67	9.86	0.78	0.83	0.92
2.00	1.34	20.02	0.67	18.41	0.58	0.68	0.68
2.50	1.37	27.23	0.65	22.82	0.51	0.62	0.60
3.00	1.39	31.68	0.62	24.74	0.48	0.60	0.57
3.50	1.42	34.58	0.60	25.85	0.47	0.59	0.56
a* =	1.94	g		ay =	1.31	g	
d* =	13.52	cm		dy =	1.32	cm	

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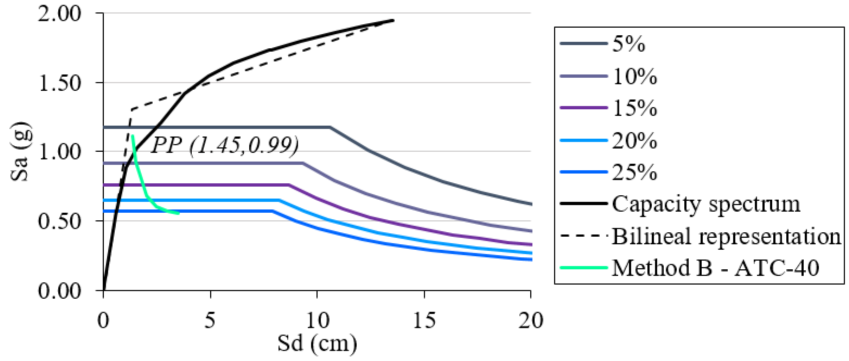

Fig. 14 – Calculation of the performance point (PP)

7.3 Seismic performance evaluation

In order to determine the performance of the buildings, the calculated performance points are converted in order to locate them in the capacity curve as shown in Fig. 15, which is divided according to the performance levels defined by VISION 2000 [3], as are indicated in item 2.2 and Fig. 1.

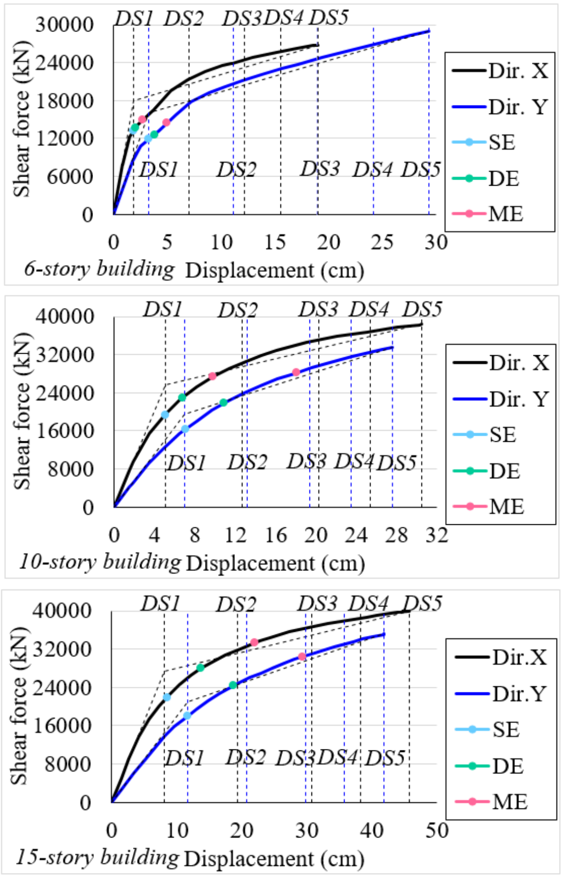

Fig. 15 – Performance points on the capacity curve of the buildings

To evaluate the performance of the buildings, and based on the design objectives defined by VISION 2000 [3], which are indicated in item 2.3 and Table 1, the performance levels obtained are evaluated in order to validate the design of the buildings.

Furthermore, as a study case, the evaluated buildings present a use factor (U = 1, E.030 [1]), which corresponds to an importance of basic structures (1, VISION 2000 [3]). According to this classification, the buildings meet the global performance objectives, as shown in Table 4, Table 5 and Table 6.

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Table 4 – Evaluation of performance of the 6-story building

Earthquake	Operational	Functional	Life Safety	Near Collapse	Collapse
	DS1	DS2	DS3	DS4	DS5
Serviceability (SE)	*X-Y*				
Design (DE)		*X-Y*			
Maximum (ME)		*X-Y*			

Table 5 – Evaluation of performance of the 10-story building

Earthquake	Operational	Functional	Life Safety	Near Collapse	Collapse
	DS1	DS2	DS3	DS4	DS5
Serviceability (SE)	*X*	*Y*			
Design (DE)		*X-Y*			
Maximum (ME)		*X*	*Y*		

Table 6 – Evaluation of performance of the 15-story building

Earthquake	Operational	Functional	Life Safety	Near Collapse	Collapse
	DS1	DS2	DS3	DS4	DS5
Serviceability (SE)		*X-Y*			
Design (DE)		*X-Y*			
Maximum (ME)			*X-Y*		

8. Conclusions

For the Serviceability Earthquake (SE), the buildings present an Operational and Functional performance level. This means that the buildings can continue to be used in such a way that the occupation is not interrupted. Also, it is concluded that the state of failure of any structural element does not occur.

For the Design Earthquake (DE), the 3 buildings presents a Functional performance level. This means that the buildings can continue to be used in such a way that the occupation is not interrupted. Also, it is concluded that the state of failure of any structural element does not occur.

For the Maximum Earthquake (ME), the buildings presents a Functional and Life Safety performance level. This means that the state of the structural elements allows its repair in a reasonable time and cost, there is no risk for the occupants.

The Peruvian seismic code E.030 [1], establishes as a philosophy of Seismic Resistant Design: "To avoid losses of human lives, to assure the continuity of the basic services and to minimize the damages to the property". Comparing this design philosophy with the performance objectives proposed by VISION 2000 [3], the design philosophy of the Peruvian code corresponds to a Life Safety performance level for the design earthquake. As a result, the 3 buildings present a Functional performance level for the design earthquake, so it is concluded that these buildings present a design and performance according to the established by the Peruvian code.

Comparing the drifts with the level of performance obtained in the buildings. It is concluded that the drift limit of 0.007 for reinforced concrete structures, according to E.030 [1], is a good indicator of the behavior of the structures and it corresponds to a Life Safety performance level. Also, it is inferred that exceeding this limit does not guarantee a good operation or repair of the buildings.

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