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Evaluation of Estructural performance using Nonlinear Static Analysis applied in an infrastructure

Evaluación del desempeño Sísmico utilizando el Análisis Estático no Lineal aplicado en una infraestructura

N.M. Gallardo Barreto*[1] https://orcid.org/0000-0003-0525-4045

*** Universidad Catylica Santo Toribio de Mogrovejo, Lambayeque, PERЪ**

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Abstract:
The methods of nonlinear analysis allow a better structural design and more accurate answers in the verification of a structure, through which it is based on the linear analysis of the educational institution Hйctor Rene Lanegra Romero; this has a current systemic structural configuration 780, which has been designed under the parameters of the E.030 standard. To know, verify and analyze the structural performance in which the educational institution is located, nonlinear static analysis was used; which consists of a Pushover analysis. The Pushover analysis allows us to calculate the seismic actions in structures where the elastic behavior of each element is observed, it also allows us to see the operational seismic performance in which the educational institution is located, through which capacity curves are obtained, generation of plastic ball joints and the verification of the point of performance of the building; which allows identifying if the said structure is active for a seismic signal.
Keywords: Nonlinear analysis; Pushover; distortions; seismic performance; plastic ball joints.

Resumen
Los mйtodos del anбlisis no lineal permiten un mejor diseco estructural y respuestas mбs exactas en la verificaciyn de una estructura, mediante el cual se fundamenta en el anбlisis lineal de la instituciyn educativa Hйctor Rene Lanegra Romero; este tiene una configuraciyn estructural sistйmico 780 actual, el cual ha sido disecado bajo los parбmetros de la norma E.030. Para conocer, verificar y analizar el desempeco estructural en el que se encuentra la Instituciyn educativa, se hizo uso del anбlisis estбtico no lineal; el cual consiste en un anбlisis Pushover. El anбlisis Pushover nos permite calcular las acciones sнsmicas en estructuras donde se observa el comportamiento elбstico Plбstico de cada elemento, tambiйn permite ver el desempeco sнsmico operacional en el que se encuentra la instituciyn educativa, mediante el cual se obtienen curvas de capacidad, generaciyn de rytulas plбsticas y la verificaciyn del punto de desempeco de la edificaciyn; el cual permite identificar si dicha estructura esta acta para una secal sнsmica.
Palabras Clave: Anбlisis no lineal; Pushover; Distorsiones; desempeco sнsmico; rytulas plбsticas

[1] Corresponding author:
Universidad Catylica Santo Toribio de Mogrovejo, Perъ
author: nelly.gallardo00@gmail.com

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1. Introduction

In recent years in Peru, there have been tolerant earthquakes, which have shown how vulnerable Peruvian educational buildings are, for this reason, there is a need to know and analyze the structural behavior of educational institutions to any seismic signal to prevent any damage that may cause since they are essential buildings. Considering that most educational institutions have been designed under old regulations, they have been changing and improving their parameters and structural conditions over time.

The problem raised in the article arises based on the fact that educational institutions need immediate intervention due to the age of these buildings, taking into account that the earthquakes that have occurred lately have left sequelae and internal structural damage that in the presence of a strong earthquake can collapse these essential buildings and cause great human losses. The seismic behavior of a structure depends on aspects such as the type of soil where they are built, in addition to its characteristics, mechanical properties, dynamics, and soil-structure interaction. (Hu and Qu, 2012)

Evaluating from the technical justification the present Research applied in the structural performance of the educational institution, which will allow for verification of the levels of performance of the said building; analyzing it in programs such as Etabs and Midas Gen to demonstrate if the structure needs reinforcement in the structural elements. Its objective is to determine through a non-linear static analysis the seismic performance of the Hŭctor Rene Lanegra Romero educational institution located in Ferrecafe.

2. Development

The research was carried out in the educational institution Hŭctor Rene Lanegra Romero through which there are plans of Architecture, this information will be used for non-linear analysis. It is located in the Jiryn Progreso in the district of Ferrecafe, province of Ferrecafe, and department of the city of Lambayeque. The school has a general area of 1703.85 m² with a perimeter of 165.79ml whose coordinates are -6.63203 latitude and -79.78573 longitude. (Figure 1).

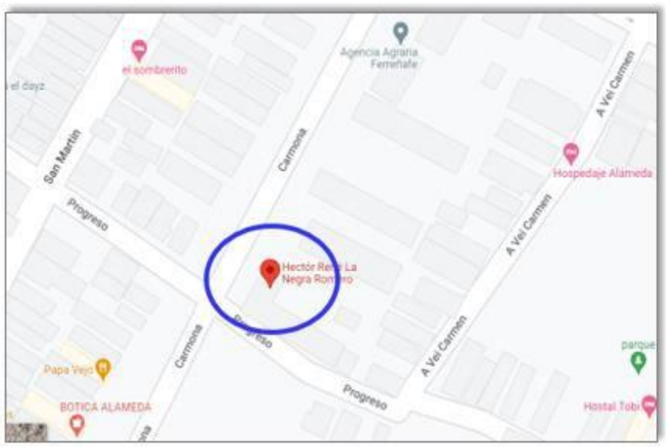

Figure 1. Location of I.E N°10056 Hŭctor Rene Lanegra Romero. Note: Google maps

The educational institution N°10056 Hŭctor Rene Lanegra Romero consists of 2 pavilions Block I and Block II built in 2001, which corresponds to the modern Architectural model INFES 780 new, these 2 blocks will be processed to a Static and Dynamic non-linear analysis. (Table 1) describes the built area of blocks I and II, generating a total of 603.04 m2 of the educational institution; Annexes 1 and 2 verify the architectural plans of the I.E.

Table 1. Description of existing blocks in the educational institution

Edification	No. of floors	Description of environments	built area (m2)
block 1	1st floor	04 classrooms	236.76
	2nd floor	04 classrooms	236.76
block 2	1st floor	04 classrooms	236.76
SS.HH1	1st floor	SS.HH	43.00
SS.HH2	1st floor	SS.HH	23.46
management	1st floor	management	62.58

Note: Own Elaboration

It can be verified in (Table 1); Block I has 2 levels and Block II has 1 level, each consisting of 4 classrooms. The blocks maintain a new modern architectural model INFES 780. Both blocks have a height of 3m per floor according to the survey that was done in the field. The two blocks maintain the same structural configuration since in the X direction they have a system of gantries with columns and banked beams; in the direction Y by walls of confined masonry. A structural redesign was made for the verification of the measures of the structural elements both vertical that would come to make the columns and the walls, and horizontal we have as structural elements the beams. The walls are of confined masonry rope posts which contain a thickness of 25cm, and are built of brick; Predominant vertical elements are the reinforced concrete columns, which have three different sections with the following: rectangular columns of 25x45cm, 25x75 cm, and T-shaped 45x90 cm belonging to the X-X and Y-Y axis. The lightened slabs of blocks I and II contain a thickness of 20cm whose banked reinforced concrete beams differ from two types of sections that are repeated on both floors and have dimensions of 25x55 cm and 30x70 cm; however, block I contain beams of 30x55cm, 25x55cm and 20x25cm that tie the balcony overhang.

The walls have a pasted and painted finish, however, the finishes of the floors are polished cement; the classrooms have wooden doors and the windows are with metal frames on each partition; the perimeter of the walls are rope partitions in block I and block II.

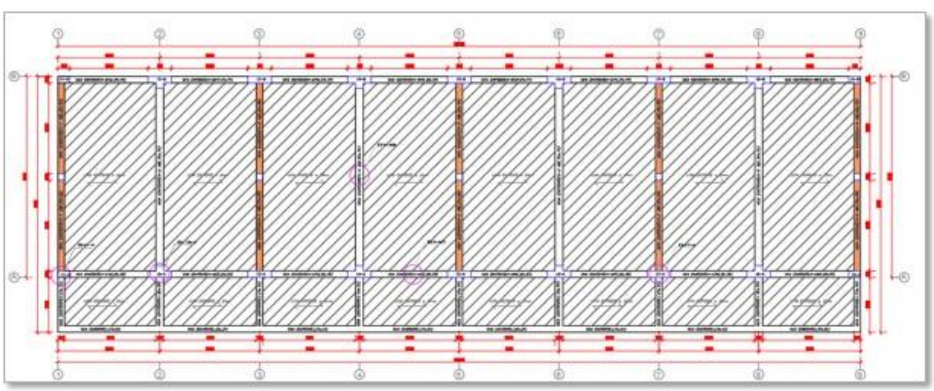

Figure 2. Block I-Systemic Type 780 Current-First Level Plant. Note: Own Elaboration

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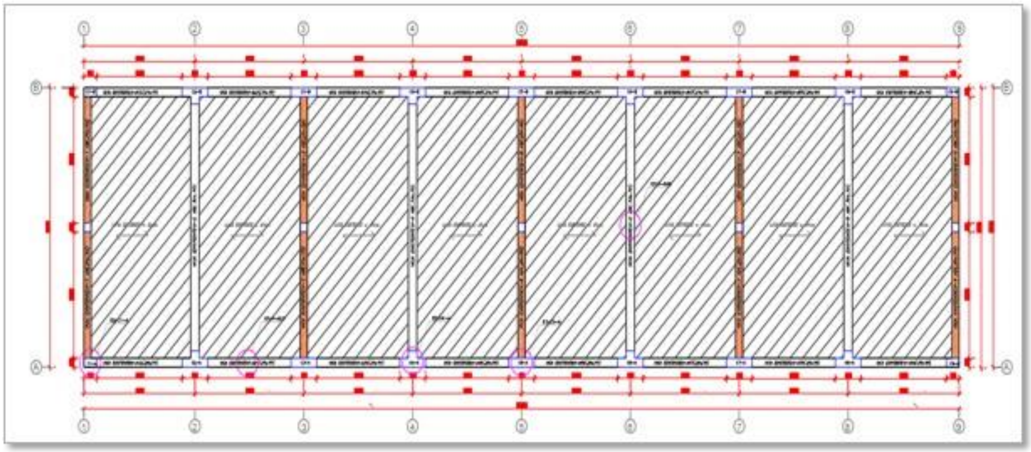

Figure 3. Block II-Systemic Type 780 Current-First Level Plant. Note: Own Elaboration

system of gantries whose design of columns are rectangular sections, T sections, and beams, while the Y is a system of confined masonry. As mentioned, the perimeter walls of the classrooms of both blocks I and II are composed of rope walls, whose element does not have a structural function.

2.1 Characteristics of materials

Reinforcing Steel

Emphasizing the structural reinforcement of the building of both blocks I and II, scanner tests had to be carried out to determine the longitudinal and transverse steels of the building. This test is developed in verifying the quantity and dimensions of the rods that have been used for the construction of both blocks. According to the test carried out, corrugated steel 60 was determined, and the number of steel that this building currently has was determined.

As shown for Block I and Block II the distribution of steels in the structural elements, for beams and columns. The longitudinal steels that stand out in their design are corrugated irons of 3/4 "and 5/8", and the stirrups have a diameter of 3/8" as can be seen in (Figure 4), through which the distribution of the steels in the beams and columns that were scanned for their correct finding the structural reinforcement is verified.

In the case of the second level, the scan was done to the type of beam V-6 AB and V-A 23 as can be seen in (Figure 5), through which both beams have abutments of the diameter of 3/8 "and their longitudinal rods of 5/8", therefore, the columns prevail with their design of the first level protruding the steels of 3/4 "and with stirrups of 5/8"

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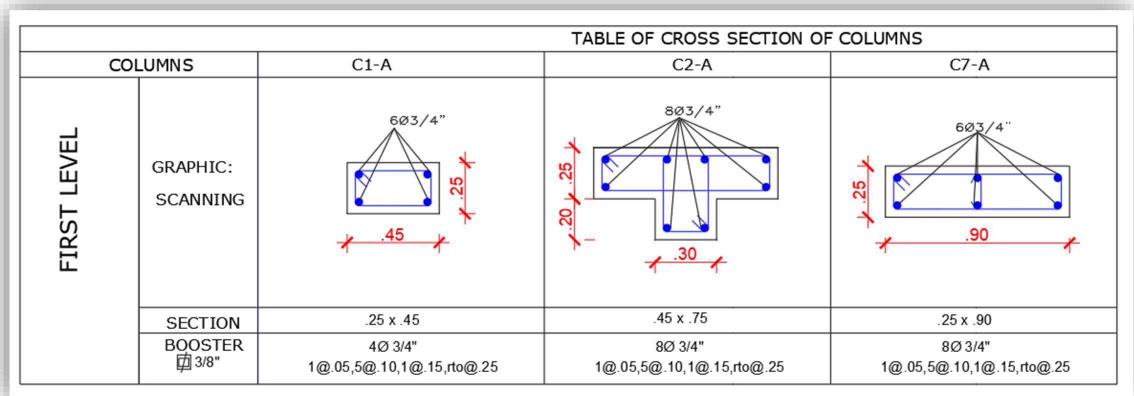

TABLE OF CROSS SECTION OF COLUMNS				
COLUMNS		C1-A	C2-A	C7-A
FIRST LEVEL	GRAPHIC: SCANNING	6Ø3/4" .25 .45	8Ø3/4" .25 .20 .30	6Ø3/4" .25 .90
	SECTION	.25 x .45	.45 x .75	.25 x .90
	BOOSTER ◫3/8"	4Ø 3/4" 1@.05,5@.10,1@.15,rto@.25	8Ø 3/4" 1@.05,5@.10,1@.15,rto@.25	8Ø 3/4" 1@.05,5@.10,1@.15,rto@.25

Figure 4. Table of a cross-section of columns of block I-First level. Note: Own Elaboration

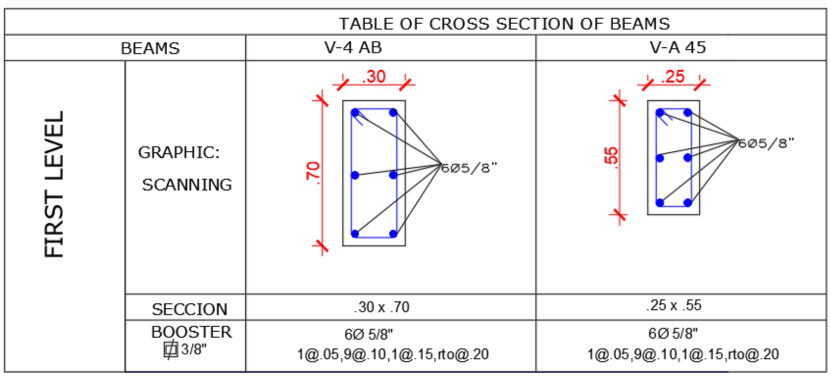

TABLE OF CROSS SECTION OF BEAMS			
BEAMS		V-4 AB	V-A 45
FIRST LEVEL	GRAPHIC: SCANNING	.30 .70 6Ø5/8"	.25 .55 6Ø5/8"
	SECCION	.30 x .70	.25 x .55
	BOOSTER ◫3/8"	6Ø 5/8" 1@.05,9@.10,1@.15,rto@.20	6Ø 5/8" 1@.05,9@.10,1@.15,rto@.20

Figure 5. Table of a cross-section of beams of block I-First level. Note: Own Elaboration

Block II, only has one level, the design of the columns and beams is formed by Grade 60 sheets of steel. In (Figure 6) and (Figure 7) it is observed that the steels that stand out in the design of columns and beams are 3/4" and 5/8", with the stirrups having 3/8" sheets of steel.

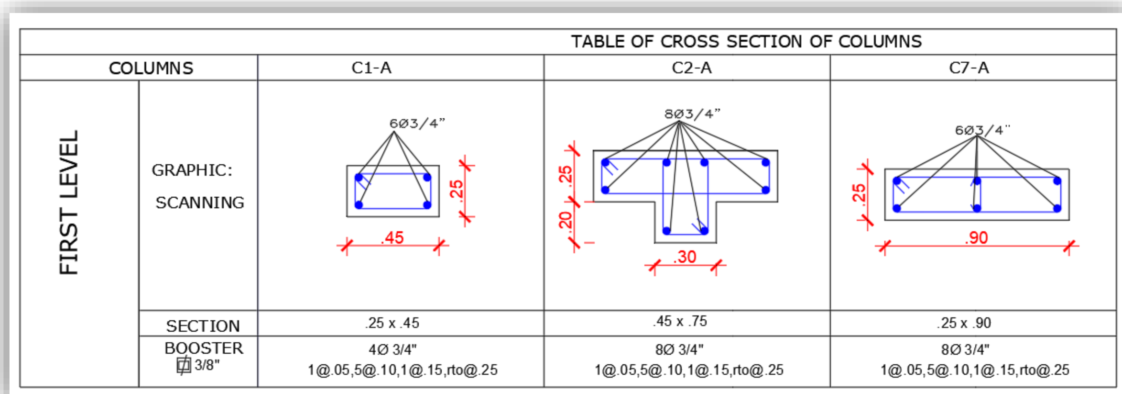

TABLE OF CROSS SECTION OF COLUMNS				
COLUMNS		C1-A	C2-A	C7-A
FIRST LEVEL	GRAPHIC: SCANNING	6Ø3/4" .25 .45	8Ø3/4" .25 .20 .30	6Ø3/4" .25 .90
	SECTION	.25 x .45	.45 x .75	.25 x .90
	BOOSTER ◫3/8"	4Ø 3/4" 1@.05,5@.10,1@.15,rto@.25	8Ø 3/4" 1@.05,5@.10,1@.15,rto@.25	8Ø 3/4" 1@.05,5@.10,1@.15,rto@.25

Figure 6. Cross-section table of beams of block I-First level. Note: Own Elaboration

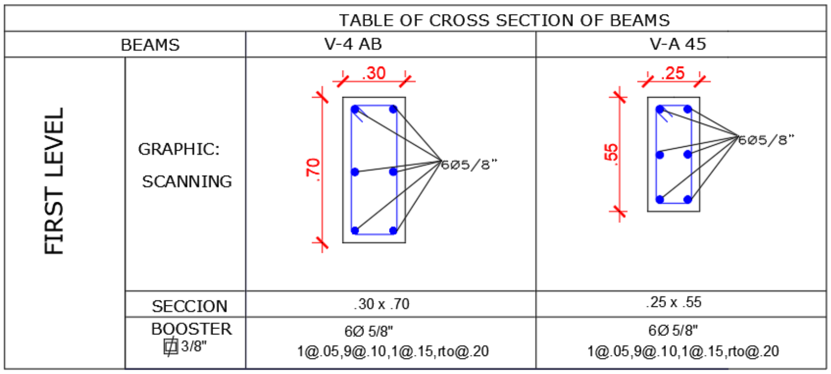

Figure 7. Cross-section table of beams of block I-First level. Note: Own Elaboration

Reinforced Concrete

To verify the strength of the concrete, the Diamantine Test was performed; which is based on extracting small concrete cores in structural elements such as columns and beams. Fully representative samples were tested; where 4 tests were carried out on beams and 3 tests on columns generating a total of 7 tests for Block I for both floors while for Block Block II a total of 5 cores were tested, 2 in beams and 3 in columns. Annexes 3, 4, and 5 verify in detail that part of the structural elements has already been vertical or horizontal concrete cores have already been tested. In (Table 2) a descriptive table of the resistance that has been obtained when testing the diamond hearts is verified indicating block I. (Figure 8) shows the strengths of the concrete that has been applied to a normal distribution to obtain more accurate data for the test performed. According to the calculations made, an arithmetic mean of 204.0 Kgf / cm² was obtained whose standard deviation generated is 22 Kgf / cm², these data were obtained for block I whose results obtained were within the range of the regulations whose concrete strength data was used for the modeling of the structure will be used is 182Kgf / cm² which results from the subtraction of the mean with the standard deviation.

Table 2. Description of the diamond sample extracted from block I

No. of test tubes	Sample description	Level	structural element	Diameter	Area mm2	Burden Kn	strength Kgf/cm2
1	ED:C4-A	1	column	81	5153	113.7	211
2	ED:C5-A	1	column	81	5153	110.7	207
3	ED:C5-B	1	column	81	5153	82.5	155
4	ED:V-4AB	1	beam	81	5153	111.9	215
5	ED:V-6AB	1	beam	81	5153	106.4	211
6	ED:V-4AB	2	beam	81	5153	107.2	212
7	ED:V-6AB	2	beam	81	5153	110.6	219

Note: Own Elaboration

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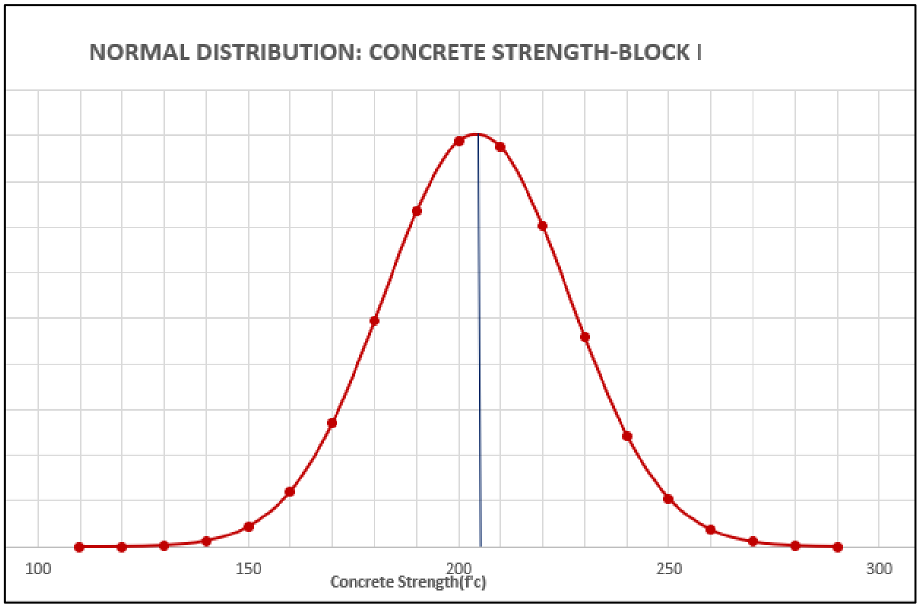

Figure 8. Normal distribution-Strength of the concrete block I. Note: Own Elaboration

For Block II the arithmetic mean of the strength of the concrete is shown in (Figure 9) having a value of 202 kgf / m² whose standard deviation gave a result of 25kgf / m². In (Table 3) the resistances obtained and tested are verified therefore the value obtained for the modeling of block II is 177 Kgf / cm². The results are shown in (Table 3) and (Figure 10).

Table 3. Description of the diamond sample extracted from block II

No. of test tubes	Sample description	Level	structural element	Diameter	Area mm2	Burden Kn	strength Kgf/cm2
1	ED:C2-A	1	column	81	5153	109.5	217
2	ED:C3-A	1	column	81	5153	110.7	206
3	ED:C2-B	1	column	81	5153	83.6	159
4	ED:V-2AB	1	beam	81	5153	116.1	216
5	ED:V-4AB	1	beam	81	5153	113.1	213

Note: Own Elaboration

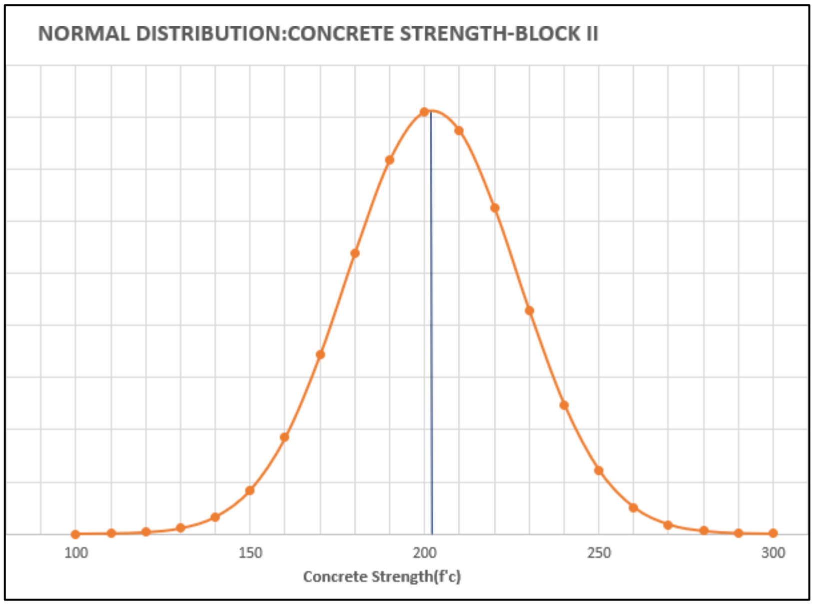

Figure 9. Distribuciyn normal-Resistencia del concreto bloque II. Note: Own Elaboration

Confined Masonry

In the Y direction, blocks I and II have a confined masonry composition, therefore, since there are no tests of prisms in the masonry walls in the said educational institution, reference values were used as shownin (Table 4) according to the Peruvian Masonry E.070 standard, whose values give the determination of the resistance of the masonry wall in Kg / cm² whose value or result serves to introduce it into the program for its correct verification. It was also considered with what materiality it was built. In summary, the table shows the characteristics of the brick whose result has an I'm equal to 65 kg / cm².

Bearing capacity of the soil

For the determination of the quality of the soil where the educational institution has been built. Hйctor Rene Lanegra Romero the verification of the soil was carried out using the mechanics of soils where three silicates were made to be tested by direct cutting, confirming the non-presence of the water table, therefore, soil samples were taken to verify l the resistance to cutting, the limits of atterbeg and the percentage of humidity, whose fundamental objective is to find the bearing capacity that the soil has. For Block I and Block II, excavation was made for three silicates at a Depth of 2.80 meters. In (Table 5) shows a summary observing the values obtained in the tests of blocks I and II, which allows classifying by means of the nomenclature ASSHTO and SUCS a type of soil SC-SM, giving as a final result a type of soil sand Clay silt for both blocks, which was also determined that they do not reach a water table; whose bearing capacity is verified in (Table 6) generating a summary of admissible loads that have been obtained when performing direct cutting soil tests concluding that for Block I and II an average allowable load of 0.64 Kg / cm² has been obtained. (Figure 11).

Table 4. Summary of bearing capacity and settlement

PITS	Df(m)	B(m)	L(m)	Load (Kg/cm2)	Settlement (mm)
C-1	1.50	1.00	1.00	0.65	0.88
C-2	1.50	1.00	1.00	0.70	0.97
C-3	1.50	1.00	1.00	0.64	0.85

Note: Own Elaboration

Table 5. Summary of bearing capacity and settlement

	C-1	C-2	C-3
	Stratum N°1	Stratum N°2	Stratum N°3
Depth	0.00-2.80m	0.00-2.80m	0.00-2.70m
Density	1.584	1.617	1.633
%Humidity	14.10%	9.70%	13.80%
Cohesión Parameters	0.216	0.278	0.193
friction angle	13.60°	11.20°	14.80°
Nomenclature AASTHO	A-4(1)	A-4(1)	A-4(1)
Nomenclature SUCS	SC-SM	SC-SM	SC-SM

Note: Own Elaboration

3. Discussion

Seismic Analysis

For the modeling of the nonlinear analysis of the educational institution for both blocks I and II, seismic values of the Peruvian regulation E0.30 (2018) are defined. According to the map of the seismic zoning of the educational institution, through this regulation there are parameters for the design of a building which are the following according to where the educational institution is located in the province of Ferrecafe which to zone 4 whose zoning factor is 0.45; Having the results the essential importance educational institution is assigned the corresponding a factor of use 1.5, therefore the school I.E. No. 10056 Hector Rene Lanegra Romero is an educational institution belonging to category "A2" defined as an essential building that serves for the shelter of people from any natural disaster. For the determination of the seismic amplification factor of the soil was determined by soil tests knowing the type of soil profile in which it is built, concluding a soil type S2 intermediate resulting in sand clay silt whose consistency is regularly compact, obtaining this parameter could define a soil expansion factor of 1.05 and the periods TP equal to 0.6 and TL equal to 2.0; using a reduction factor of 8 on the X-X axis and a value of 3 on the Y-Y axis, whose axis is composed of a confined masonry structural system. It can be seen in (Table 7) the values of the seismic parameters described in blocks I and II are observed.

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Table 6. Seismic parameters according to E-030

Seismic Parameters		
Usage Factor	U	1.5
Zone Factor	Z4	0.45
Soil Profile	S2	1.05
Periods	TL	0.6
	TP	2
Reduction Factor	Rx	8
	Ry	3

Note: Own Elaboration

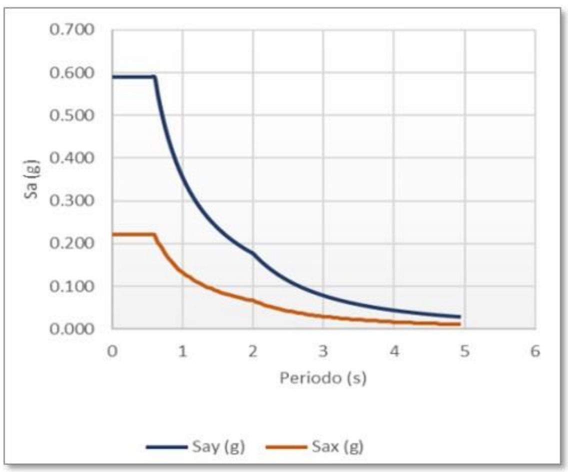

Figure 10. Block I and II inelastic spectrum. Note: Own Elaboration

The educational institution presents a current systemic model 780 whose blocks I and II are determined by a system of gantries on the X-axis and Y-axis designed under a confined masonry system. The placement of arms was made in the connection of columns and beams, it was considered a lightened slab in a direction of 20 cm was considered, by means of the load regulations, for a slab of the thickness of 20 cm it is considered a dead load of 300kgf / m², modeling with rigid diaphragms, therefore, the fully recessed supports. (Figure 2) and (Figure 3) shows the structural model made in ETABS for each block.

Distortions

According to the drifts calculated by the Etabs program for both blocks, I and II following the parameters of the Peruvian regulation E.030 indicate that the maximum concrete torque drift reinforced is 0.007, according to the tables generated by the program to calculate the drift the *distortion between floors 0.75 is multiplied by the reduction factor that the direction X is equal to 8, in the direction Y is equal to 3. As can be seen for the block I generating tables the program with a maximum drift of 0.0021 in the X direction and with a maximum drift of 0.0012 in the Y direction. (Figure 12) and (Figure 13) .*

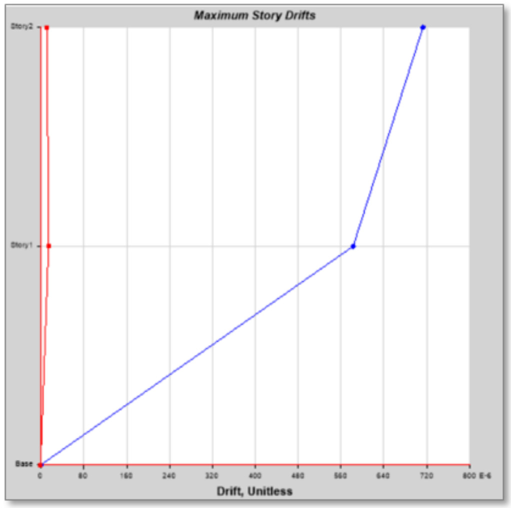

Figure 11. Distortions on the X and Y axis of block I. Note: Own Elaboration

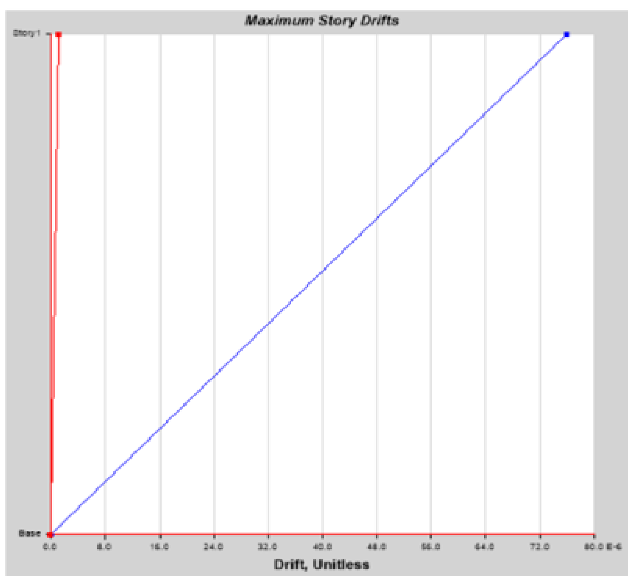

Figure 12. Distortions on the X and Y axis of block II. Note: Own Elaboration

Cutting at the Base

The Dynamic shear force according to Peruvian regulations E.030 should not be less than 80% of the product of the static basal shear, as can be seen in the planes blocks I and II are symmetrical structures, observed in (Table 7) and (Table 8) the generation of both dynamic and static shears by the Etabs program.

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Table 7. Static and Dynamic Block I Shear

0.8Vxest (Tnf)	0.8Vyest (Tnf)	Vxdinam (Tnf)	Vydinam (Tnf)
86.7408	231.31536	101.8984	284.5588

Note: Own Elaboration

Table 8. Static and Dynamic Block II Shear

0.8Vxest (Tnf)	0.8Vyest (Tnf)	Vxdinam (Tnf)	Vydinam (Tnf)
36.9588	98.55968	46.1985	123.1996

Note: Own Elaboration

Nonlinear static analysis

For the realization of the Pushover was modeled in the Midas Gen which is a program where the most accurate nonlinear methods are calculated analysis for both blocks I and II, which used inelastic models, are more complex models that generate more accurate results and analyze the structural elements. In (Figure 14) and (Figure 15) you can see block I and block II modeled in the Midas Gen program.

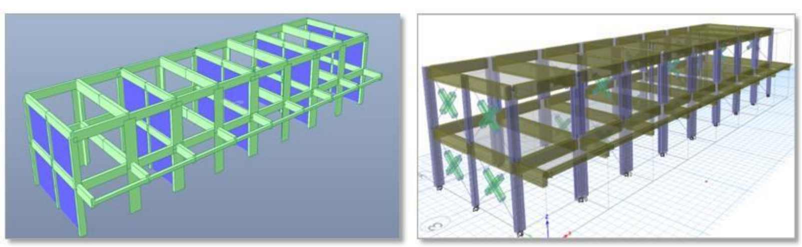

Figure 13. A nonlinear model of the block I. Note: Own Elaboration

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As is known, the institution to be evaluated consists of 2 blocks I and II; through which block I consist of 2 floors with 4 classrooms per floor, while block II consists of a floor with 4 classrooms, which have been subjected to the Pushover analysis to analyze at what operational level blocks I and II are.

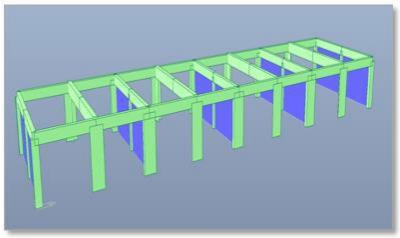
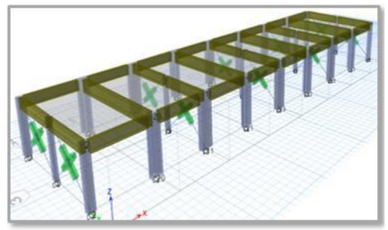

Figure 14. A nonlinear model of block II. Note: Own Elaboration

According to the Pushover analysis, applied in the X-X axis, it is observed that in the Image the first structural elements that fail against incremental loads are the columns of the porticoed system that do not correspond to the confined masonry walls of the Y-Y axis, the Graph shows as an example one of the first columns that collapse, this is because it exceeds the rotation limits according to table 11.4 of ATC40 when the moment reaches a value of -11.8785 Tonf-m its rotation results in -0.019588 rad being the maximum point in the graph of Moment – Rotation, however the final collapse point in the curve reaches a rotation of -0.032189 rad which exceeds the level of structural stability performance. The image already shows a generalized collapse in the first level, this is because the limit rotation was already exceeded in each of the columns, in the case of the beams, the graph shows an example in which the maximum rotation of a beam in its moment – rotation curve is 0.002334 rad, which indicates that its level of performance is of immediate occupation, but because it is a porticoed system if the columns fail the structural system would collapse as a whole.

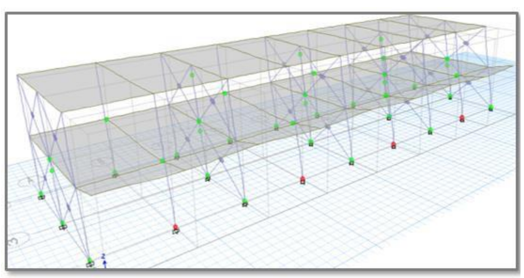

Figure 15. Pushover Analysis – Block I. Note: Own Elaboration

Plastic Ball Joints

The Pushover analysis, applied in Block I of the X axis through the FEMA356 regulations, shows in (Figure 16) the generation of plastic ball joints. Mainly the structural elements that generate failures before incremental loads we have the columns of the gantry system which do not belong to the confined masonry walls of the Y axis, which break at the base of the columns being in an ultimate state of the element, it is also observed that the rest of the structural elements are at an immediate occupation level, others in life safety and also in collapse prevention.

24.4% has flowed with an insipient creep, 0% security of life and 19.8% of the elements have failed since they are in an ultimate state.

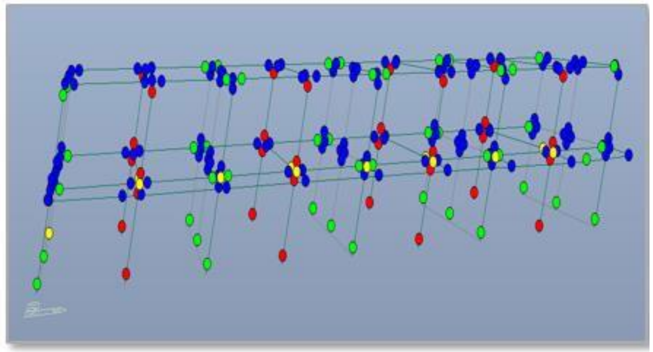

Figure 16. Generation of plastic ball joints – Block I. Note: Own Elaboration

For the Pushover method, which has been applied in Block II of the X axis through the FEMA356 regulations, the generation of plastic ball joints is observed in (Figure 17). The structural elements generate failures before incremental loads are the columns of the gantry system, as previously said that they were not the ones that were related to the confined masonry walls of the Y axis, which break at the base of the columns being in a state of immediate occupation, some in life safety and others in the prevention of collapse. 81.3% and 18.8% of the elements have failed since they are in an ultimate state.

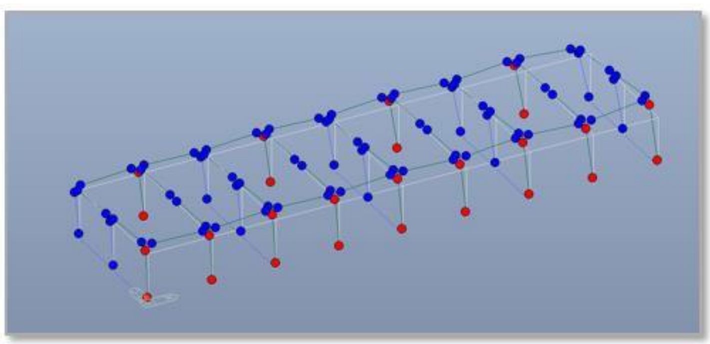

Figure 17. Generation of plastic ball joints – Block II. Note: Own Elaboration

Capacity curve

To obtain the capacity curve of block I in the direction X-X a pattern of incremental lateral loads is subjected to the structure until finding a point where the building reaches its maximum capacity, through the case of pushover load X-X applied in the program the graph is obtained which shows that the point where the nonlinearity begins on the X-X axis of the structure occurs when the value of the Basal shear is 80.39 Tnf and the greatest force it receives is 84.85 Tnf, which occurs in the non-linear part of the curve, finally the maximum displacement that the structure can reach in said axis is 0.201 m being this the final collapse point. (Figure 18).

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Figure 18. Base shear vs displacement – Block I. Note: Own Elaboration

Performance Point

Performance point In order to obtain the performance point, the capacity curve must be converted into a capacity spectrum so that it can intersect in the ADRS (Acceleration Displacement Response Spectra) format with the demand spectrum obtained in the graph. Based on the FEMA 440 criteria, two different performance points were found since the maximum elastic spectrum and the design spectrum were used. The graph shows the performance point for a maximum earthquake, obtaining as an intersection point in the axis of the abscissas a pseudo displacement of 0.1884 m and for the axis of the ordinates, a pseudo acceleration of 0.2679 g, in the case of the graph a performance point is shown based on a design earthquake spectrum, taking the values of 0.1004 m for pseudo displacement and 0.288 g for pseudo acceleration.(Figure 19).

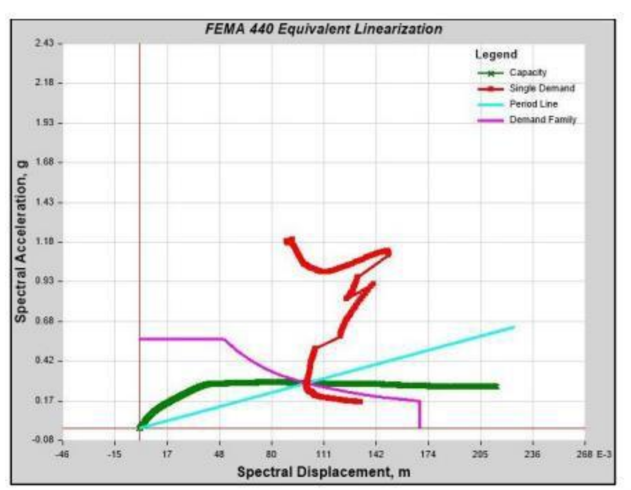

Figure 19. Performance point for a maximum earthquake spectrum – Block I. Note: Own Elaboration

4. Conclusions

In school No. 10056 Hйctor Rene Lanegra Romero, 7 diamond trials were carried out in block I; 3 in columns and 4 in beams for both the first and second level obtaining concrete resistance of 182Kgf / cm².

In block II, 5 diamond assays were performed; 3 in columns and 2 in beams The structure is one floor obtaining as resistance to the concrete of an f□c of 177kg / m². For block I and block II according to the scanner tests that were carried out they gave results generating steels of 3/4'' and 5/8'', in this case for sections of columns of 25x45, 25x90 and for the section of columns in T is 45x75; In beams, the sections obtained are 25x55 and 25x70.

According to Peruvian standard E.30, the dynamic shear force generated in the mezzanine of the building is greater than 80% of the static shear obtained, which the designed model does not need to be scaled. In the generation of the capacity curve, a pattern of incremental lateral loads is elaborated to the building until finding a point where the building reaches its maximum capacity in the modeling, according to the case of load Pushover Structural collapse in the nonlinear method of the structure occurs when the value of the basal shear is 75Tnf and the greatest force it receives is 115 Tnf, that occurs in the non-linear part of the curve, finally the maximum displacement that can reach the is 0.004m being this the final collapse point.

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