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Non-Linear Pushover Analysis of Tall Buildings with Post-Tensioned Slabs and Rigid Core Designed Under Peruvian Codes

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Abstract **–** Currently at Peru tall building over 30stories began to be built. However, Peru structural design codes are based on medium and low-rise buildings. As well, posttension slab is used to these high rise buildings. The objective of this research was to evaluate the behavior of 6 high-rise buildings through nonlinear Pushover analysis. These buildings include rectangular and square floor plans of 40, 36 and 32 floors, and were designed considering the Peruvian regulations E 0.20 'Loads', E 0.30 'Earthquake Resistant Design' and E 0.60 'Reinforced Concrete'. Rigid core systems were used, varying the compressive strength of the concrete. Also 35 cm thick post-tensioned slabs were used as part lateral structural resistant system, which behavior presented high punching shear forces due earthquake loads. The results of the nonlinear pushover analysis show that the capacity curves are fragile because the hinges of the walls fail in shear instead of in bending moment-compression. On the other hand, it was also found that buildings with 32 and 26 levels with a rectangular floor plan are unable to meet the displacement demand.

Keywords**:** Tall buildings - rigid core - post-tensioned slabs - non-linear pushover analysis - plastic hinges

1. Introduction

During the last decades, the imminent population increase and demand for space in foreign countries have brought with it the application of unconventional construction alternatives, such is the case of the residential complex "The Sharp Star City Apartment" in Seoul, Korea. Which consists of 4 high-rise buildings with rigid core systems in which, for aesthetic reasons, flat post-tensioned slabs were used instead of conventional beams [1].

The use of post-tensioned slabs has been widely applied and studied in the most developed countries. In 2006, the University of Berkley carried out a series of studies to evaluate the structural behavior of 20 cm post-tensioned slabs adhered to columns and shear walls. The results showed that these slabs support high lateral forces and that they have very high flexural inertia. In addition, it was concluded that these slabs resisted mezzanine drifts up to 5%. Figures 1 and 2 show the trials of this study. [2].

Fig. 1: Post-tensioned slab cracking at 0.5% drift

Fig. 2: Post-tensioned slab cracking at 5% drift

For this reason, it is important to incorporate post-tensioned slabs into the structural system in this investigation, where the thickness of the post-tensioned slabs must meet the maximum drift requirements stipulated in the Peruvian code, considering a reduction in the moment of inertia due to cracking. To obtain an optimal thickness of the post-tensioned slab in high-rise reinforced concrete buildings with a rigid core, it is also necessary to obtain adequate values for bending, shear and punching shear, which results in an iterative process between changes in the thickness of walls, slabs, and resistance values. compression [3].

Faced with an imminent future need for spaces as a reflection of examples abroad and given the low presence of tall buildings in Peru, this research aims to assess whether Peruvian regulatory codes are prepared to cover the design of tall towers, which It will be evidenced through the nonlinear Pushover analysis of 6 buildings composed of rigid core systems and post-tensioned slabs.

2. Design of building

2.1. Buildings characteristics

For the study, two types of floors were used, square of 29 m x 29 m and rectangular of 52 m x 25.6 m for buildings of 32, 36 and 40 levels with a mezzanine height of 3.8 m, which were modelled in the ETABS 19 software [4]. The 6 buildings are made up of 100 cm2 columns on the first 19 floors and 80 cm2 columns on the rest of the levels. The bidirectional post-tensioned slabs used were 35 cm thick for all levels.

Different thicknesses were used for the walls that vary as the height of the buildings increases, being the walls of the first levels the ones with the greatest dimension. In the 40-story buildings, 60 cm thickness was used in the first 10 levels and 45 cm for the rest. In the 36-story models, the thickness was 45 cm and remained constant throughout the building. Lastly, in the 32-story buildings, thicknesses of 45 cm were chosen for the first 10 levels and 35 cm for the rest.

The compressive strength (f'c) of the vertical elements varies between 700 kg/cm2, 630 kg/cm2 and 560 kg/cm2, while the horizontal elements have an f'c of 420 kg/cm2. Regarding the design loads assigned to each building, these were selected according to the office and hotel uses established in the Peruvian standard E.020 [5]. In figures 3 and 4 we can see the plan and 3D views of the 40-story buildings with a square and rectangular floor plan.

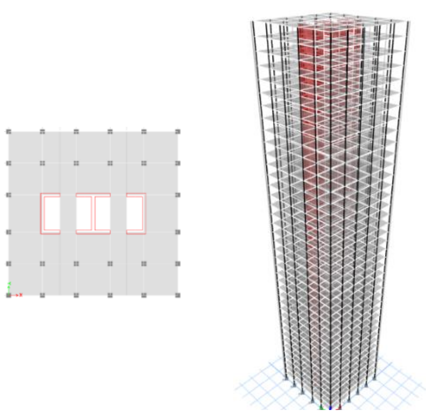

Fig. 3: Plan view and 3D view of the 40-story square building.

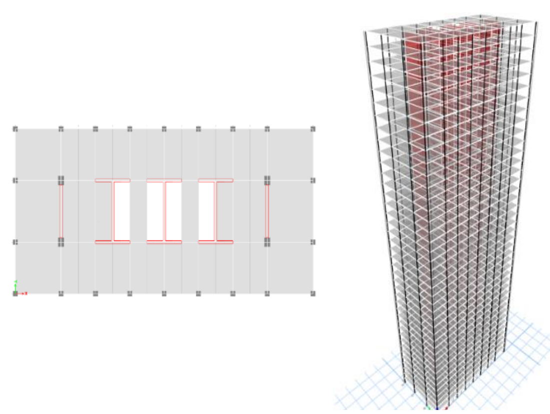

Fig. 4: Plan view and 3D view of the 40-story rectangular building.

2.2. Linear Seismic Analysis

Once the 6 models were made in ETABS, the static and modal spectral seismic analyses were carried out as established in the Peruvian standard E.030 [6]. The seismic coefficients used were Z = 0.45 (high seismicity), U = 1 (use factor), S=1 (soil factor) and R=6 (reduction factor), having regular shapes.

The inelastic mezzanine drifts in the 6 buildings complied with the maximum limit established by the E.030 standard, which is 0.007. In addition, the comparison between the base shear results obtained between the static and spectral modal analyzes allowed finding the seismic amplification factors as established by the E 0.30 standard, with the dynamic shear being at least 80% of the static shear for regular buildings. This amplification is necessary to comply with Peruvian regulations for the estimation of internal forces and subsequent design of structural elements.

2.3. Design of reinforced concrete structural elements

Once the verification of drifts was carried out and the internal forces in the structural elements were determined with the ETABS 19 software, the design was carried out by flexo-compression and shear of vertical elements as stipulated in the Peruvian standard E 0.60 [7].

For the design of the post-tensioned slabs, the American standard ACI 318-14 [8] was used since the design of these is not contemplated in the Peruvian regulations. The calculation of the number of tendons and the amount of necessary reinforcement was carried out, in addition to the bending and punching design using the ADAPT software [9], the results were verified manually.

3. Non-linear pushover analysis

3.1. Non-linear modelling of buildings

After the buildings were designed by the current Peruvian regulations, they were modelled in SAP2000 [10]. The walls were modelled as columns connected with rigid beams with a modified modulus of elasticity, which allowed simulating the contribution of stiffness. After the calibration of the models, the steel reinforcement in the structural elements were added and subsequently the non-linear properties that govern the behavior of the materials. In the case of the post-tensioned slabs, these were modelled as beams with equal steel reinforcement to the design results.

3.2. Determination of capacity curves

The nonlinear Pushover analysis consists of subjecting the buildings to incremental loads with one degree of freedom until they collapse, in this way we obtain the structural capacity curves according to the provisions of ATC-40 [11].

The nonlinear gravitational loads used were 1.1 times the dead and live loads for each building type, these amplified loads were used to create the pushover load patterns in each direction. To know the sequential behavior of the structural elements when entering the nonlinear range, plastic hinges were assigned according to the ASCE/SEI 41–13 tables [12]. For the columns, flexo-compression hinges were assigned at 7% and 93% of the free span, for the walls both shear hinges at 50% of the span and flexo-compression hinges were considered because they are slender elements at 6% and 94%.

To consider the non-linear behavior of the post-tensioned slabs, 2% and 98% plastic hinges will be applied. These hinges were determined through the calculation of resistant moments, which vary according to the number of existing reinforcing strands. In Figure 5 we can see the moment of curvature graph for a post-tensioned slab in the 40-story square plan.

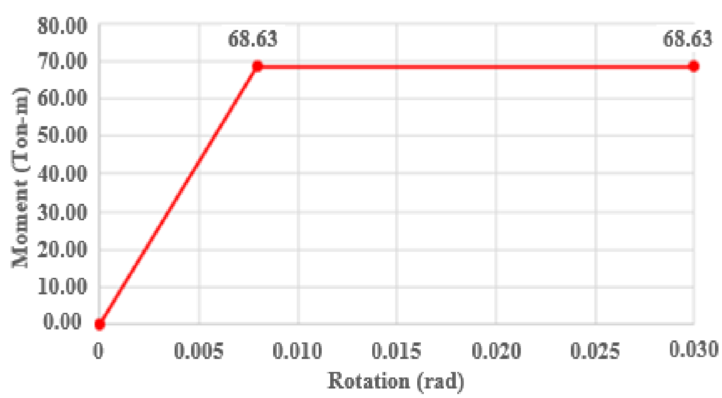

Fig. 5: Graph Moment vs Rotation of post-tensioned slab on the 40-story square plan

Pushover load patterns were applied by iterating target displacements until collapse, obtaining structural capacity curves for each axis of the 6 buildings. Figures 6 and 7 show how the plastic hinges of the 40-story buildings with square and rectangular floors, respectively, were formed.

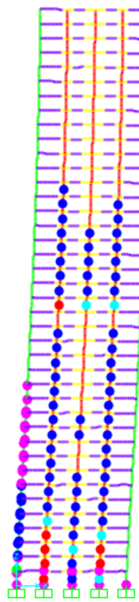

Fig. 6: Formation of plastic hinges in a square plan of 40 floors in the X direction

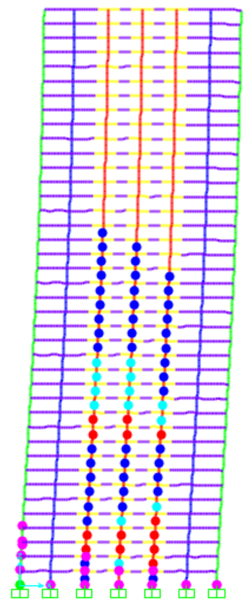

Fig. 7: Formation of plastic hinges in a rectangular plan of 40 floors in the X direction

The behavior of the buildings in both directions is similar for all the models with a rigid core, it is not possible to see hinges in the post-tensioned slabs since they present great resistance to bending. Flexo-compression hinges are observed at immediate occupancy (IO) and life safety (LS) levels for first floors columns. Furthermore, shear hinges are observed for walls at all levels, these being the reason for the collapse.

The capacity curves of the 6 buildings on each axis were obtained. Figures 8 and 9 correspond to PC40, PC36 and PC3 while figures 10 and 11 correspond to PR40, PR36 and PR32.

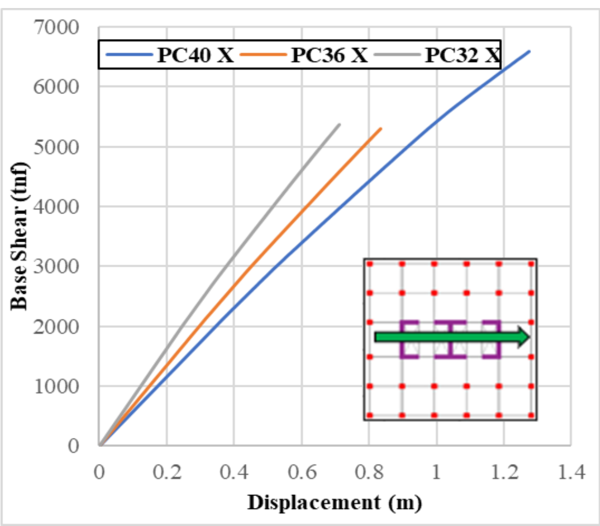

Fig. 8: Pushover curve of the square plan of 40, 36 and 32 levels (X axis)

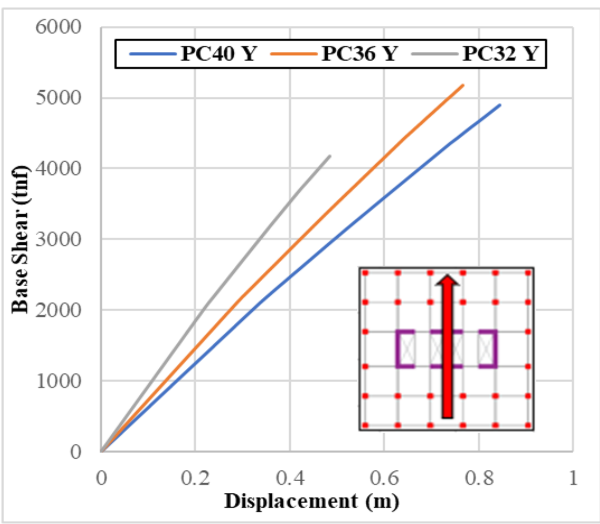

Fig. 9: Pushover curve of the square plan of 40, 36 and 32 levels (Y axis)

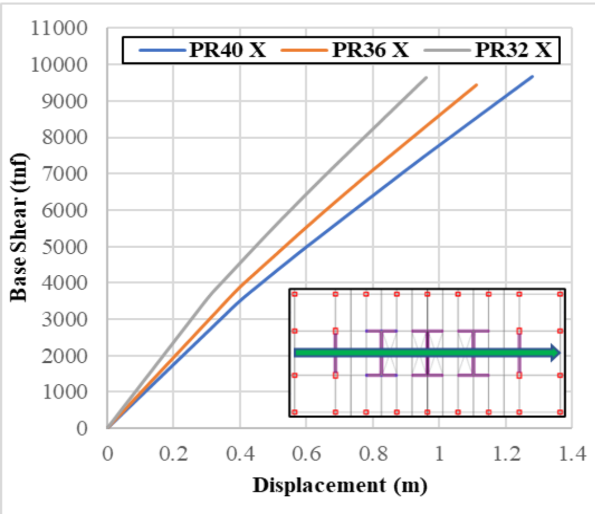

Fig. 10: Pushover curve of the rectangular plan of 40, 36 and 32 levels (X axis)

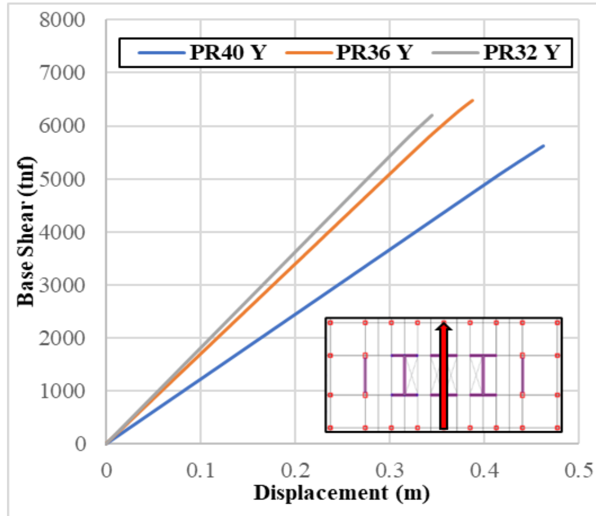

Fig. 11: Pushover curve of the rectangular plan of 40, 36 and 32 levels (Y axis)

3.3. Seismic demand

The seismic demand was obtained through the scaling of 7 pairs of seismic records using the Seismomatch software [13] between 0.2 and 1.5 times the fundamental period of each building, taking the target spectrum of the E 0.30 Peruvian standard, which is shown in the figure. 12. All the accelerograms used correspond to stations located on ‘‘S1 soils’’ according to the classification of the Peruvian standard and were previously corrected by baseline using the Seismosignal software [14]. Table 1 shows the seismic records used.

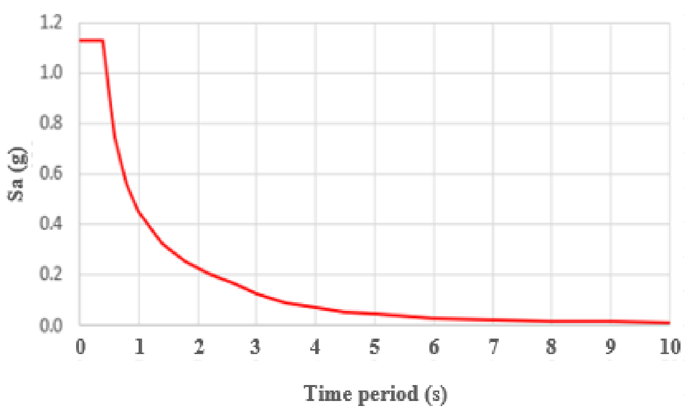

Fig. 12: Target spectrum according to peruvian code E 0.30

Table 1: Seismic Records

N°	Seismic Records		
	Station	**Magnitude**	**Date**
1	Parque de la Reserva (PRQ)	8.1	17/10/1966
2	Parque de la Reserva (PRQ)	6.6	31/05/1970
3	Surco (SCO)	6.2	03/10/1974
4	La Molina (MOL)	6.0	09/11/1974
5	UNSA (AQP001)	6.5	07/07/2001
6	Jorge Basadre Grohmann (TAC001)	7.2	13/06/2005
7	Jorge Basadre Grohmann (TAC001)	6.3	22/03/2015

4. Results

4.1. Performance Points

To intercept the capacity curve with the seismic demand, the conversion procedures to the "Acceleration-displacement response spectra" (ADRS) format provided by the ATC-40 were used, allowing to find the performance point for each axis in each model. For a better visualization, the following figures were made that show the performance points of the buildings of 36 and 32 with a rectangular floor plan. Figure 13 corresponds to PR36 and figure 14 to PR32.

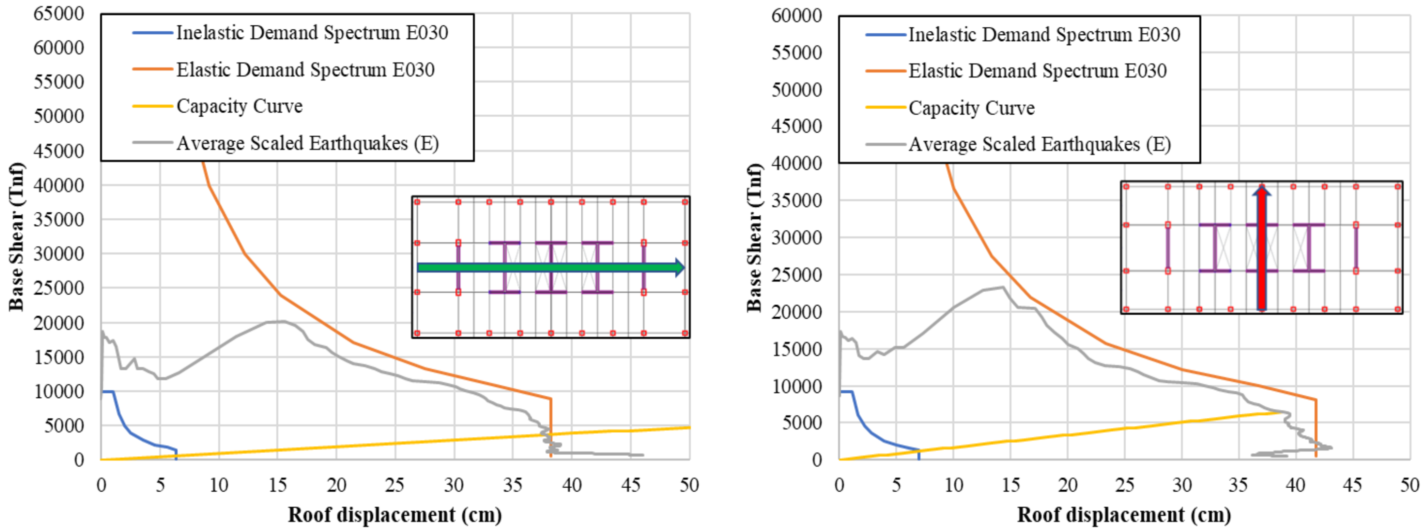

Fig. 13: Performance Points of PR36

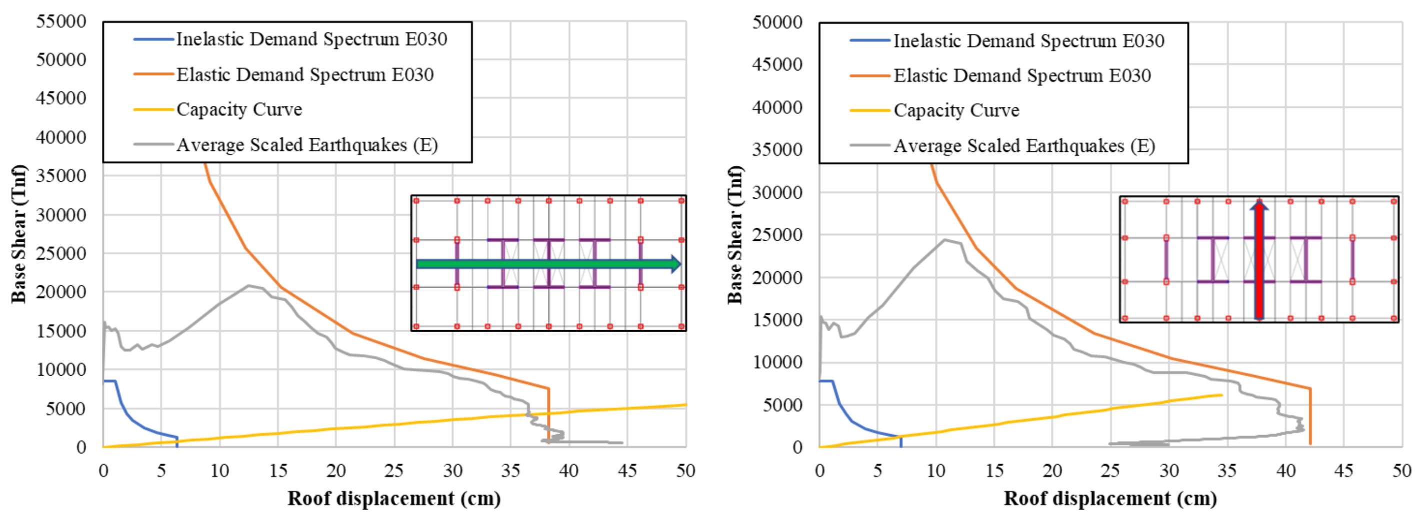

Fig. 14: Performance Points of PR32

4.2. Tables of results

The following tables show a summary of the performance points obtained for each building according to the type of plant and direction of analysis. Tables 2 and 4 show the displacement results, while tables 3 and 5 show the results of the base shear obtained.

Table 2: Displacement results of square plant

Demand	Roof Displacement (cm)					
	PC40		***PC36***		***PC32***	
	X-X	***Y-Y***	***X-X***	***Y-Y***	***X-X***	***Y-Y***
NTP - E.030 Elastic	39	39.4	38.64	39.33	38.46	39.30
E. Average	38.65	38.8	37.45	38.25	36.70	37.45

Table 3: Square plant basal shear results

Demand	Base Shear (Tnf)					
	PC40		***PC36***		***PC32***	
	X-X	***Y-Y***	***X-X***	***Y-Y***	***X-X***	***Y-Y***
E. Average	2255	2402	2509	2738	2940	3316
NTP - E.030 Inelastic	373	410.5	434.5	479.5	524	604.5

Table 4: Displacement results of rectangular plant

Demand	Roof Displacement (cm)					
	PR40		***PR36***		***PR32***	
	X-X	***Y-Y***	***X-X***	***Y-Y***	***X-X***	***Y-Y***
NTP - E.030 Elastic	38.3	42.1	38.2	-	38.2	-
E. Average	38.1	41	37.8	38	36.4	-

Table 5: Rectangular plant basal shear results

Demand	Base Shear (Tnf)					
	PR40		***PR36***		***PR32***	
	X-X	***Y-Y***	***X-X***	***Y-Y***	***X-X***	***Y-Y***
E. Average	3345	5013	3683	6470	4193	-
NTP - E.030 Inelastic	560.5	857.5	620.5	1178	753.5	1227

5. Validation

According to the E 0.30 standard, the seismic amplification coefficient “C” decreases markedly when the fundamental period exceeds the limit “Tp”, resulting in lower pseudo-accelerations. Likewise, a lower value of "C" also represents lower internal forces for the design of structural elements, which means that the minimum structural reinforcement stipulated by Peruvian regulations is insufficient to withstand shear forces in high-rise buildings.

This is demonstrated by the results obtained in the nonlinear Pushover analysis, in which the capacity curves manage to intercept the inelastic spectrum E 0.30, satisfying the base shear demands according to the Peruvian code. Despite this, it is observed that the slender core walls of 3.8 m height, in which a behavior governed by flexo-compression deformations is expected, fail earlier due to shear.

For this reason, it has been possible to demonstrate a deficiency in the minimum amounts established for the design of buildings with this type of structural configuration, for which it would be necessary to modify the regulations E 0.30 "Earthquake resistant design" and E 0.60 "Reinforced concrete" considering its application for tall buildings with long periods and provide adequate reinforcement to ensure ductile behavior.

Next, in figure 15 we can see the shear hinge obtained using the minimum steel reinforcement established in the E 0.60 standard, the core walls were provided with residual ductility to observe the progressive failure behavior of the rest of the elements when they fail. by shear.

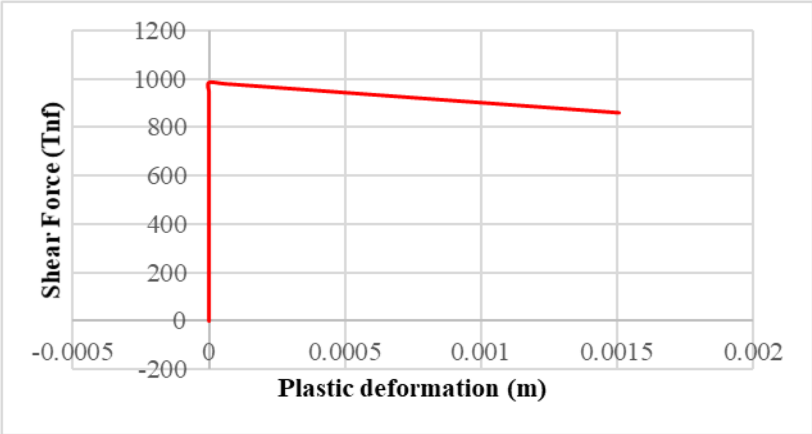

Fig. 15: Shear hinge of a structural wall

6. Conclusions

It was concluded that the seismic amplification coefficient "C" of the standard is not appropriate for the design of tall buildings. During the analysis of the models, long periods will be acquired that oscillate between the intervals [4.04 s - 5.76 s] and [3.35 s – 5.71 s] for the square and rectangular plants, respectively, being in all cases greater than the TL (2.5) established in the seismic design Peruvian code for type 1 soils.

In all cases, the value of R was 6 (structural wall systems and without the presence of irregularities) and the C/R ratio showed very small values that should have been considered at least 0.11 according to the Peruvian code. As a result, low pseudo accelerations and lower basal shears were obtained in the linear analysis, which are decisive for estimating the internal forces in the structural elements.

The formation of plastic hinges was not witnessed in the post-tensioned slabs with concrete of f'c 420 kg/cm2, in addition they presented great resistance to bending and shear due to their thickness of 35 cm and the number of tendons calculated in the design that ranges between 18 and 24.

As mentioned above, the capacity curves for buildings in each direction are fragile, this is because the minimum reinforcement stipulated in the E.060 standard for the design of structural walls is not sufficient to withstand the shear forces that the rigid core receives. Therefore, these walls fail by shear instead of by flexo-compression despite being slender elements.

The buildings have performance points that meet the demands of the Peruvian shear and displacement standard in most cases. The exception occurs on the Y axis for buildings with a rectangular floor plan of 32 and 36 levels, where the capacity curves were able to intercept the inelastic spectrum E 0.30 but were unable to meet the displacement demand. This is due to the low ductility of the capacity curves in the face of sudden shear failure of the structural walls.

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