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PERFORMANCE BASED SEISMIC DESIGN OF PIERS OF REINFORCED CONCRETE BRIDGES

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Abstract

In Peru, there is a great demand for bridges with reinforced concrete piers, which are designed for resistance (force-based methodology). Currently, the seismic design is based on displacements due to its close relationship with structural damage. It is for this reason that it is necessary to know the capacity of lateral displacement of the bridges, and compare it with the demand for displacement to know the level of performance according to the regulations used.

The objective of this research is to present an empirical expression to calculate the lateral displacement capacity of the reinforced concrete piers from a parametrically defined bridges family. It is known that nonlinear static analysis (NSA) - Pushover is a powerful tool to calculate the displacement capacity of a structure and in this particular case of bridges. The analysis Type-NSA was performed in the longitudinal and transverse direction of the bridge to determine the lateral displacement capacity in all the cases studied. In addition, the ductility criteria and the verification were considered in the analysis so as not to consider the P-Delta effect on the piers, according to Caltrans [1].

The family of bridges used in the research is made up of continuous bridges of three span with two intermediate piers, composed of columns of circular cross-section of reinforced concrete. Three-dimensional modeling of the bridges was performed, using the following elements: area element in the superstructure, and frame element in the piers. In addition, it was considered that the superstructure is supported on the infrastructure and considering an isolator compose by neoprene layers between both, the columns of the piers were considered to be fixed into the base and the flexibility of the cap beam that connects them at the top end was taken into account. Also, the lateral restraint in the abutment and the seismic control devices in the intermediate piers that restrict both transverse and longitudinal movement during the occurrence of an earthquake was considered. The parameters considered were the following: the height of the intermediate columns, the number of columns in each pier, and the amount of vertical reinforcement of the columns.

It was observed that in both directions of analysis of the bridge, the capacity for lateral displacement of the bridge decreased as the amount of vertical reinforcement for geometrically equal piers increased and that the lateral displacements were better related to the height of the piers. For this reason, the formulas developed for the calculation of lateral displacement are related according to the height of the pier and are affected by the limitations of ductility and the not need to consider the P-Delta effect, according to the recommendations indicated in Caltrans [1].

Finally, the proposed formula may be used in the pre-sizing stage of reinforced concrete piers, allowing the seismic design by displacement to be more expeditious, because the lateral displacement capacity obtained by this formula has a margin of average error of 5% in the longitudinal direction and up to 5% in the transverse direction, compared to the displacement capacity obtained from an NSA, decreasing the iterative process involved in seismic design by displacements.

Keywords: Bridges, Performance based design, Pushover, Seismic design.

1. Introduction

Peru is a highly seismic country, for this reason the structures must be designed to withstand a severe seismic event during its useful life. In the particular case of bridges, these must remain operational precisely after the occurrence of such events.

There are different methodologies to perform the seismic design of bridges, such as the force-based design (FBD), which takes into account the forces produced during a seismic event and the displacement-based design (DBD) that takes into account the displacements induced during seismic event.

It is known due to the experience in past earthquakes, the FBD has many shortcomings that do not provide the confidence to be able to carry out the seismic design of bridges, because not all structures designed by this method will achieve the intended performance during the design stage [2, 3].

On the other hand, the DBD, relates the design directly to the demand for displacements which are related to the damage and therefore there is greater reliability in this design methodology [4]. Therefore, it is expected the bridge exceeds the elastic limit during a severe seismic event, so it is more appropriate to use a design that relates the seismic intensity to the damage that occurs on the bridge during this severe seismic event.

The linear elastic analysis gives you much information about the dynamic behavior of bridges, this analysis does not give you information about the mechanism of its failure, i.e., it does not take into account the formation of the plastic hinges and the progressive deformation of the bridge until it collapses. For this reason, it is advisable to use a nonlinear analysis that gives you information about the mechanism of bridge collapse, as the case one of nonlinear static analysis (NSA) – Pushover [5, 6].

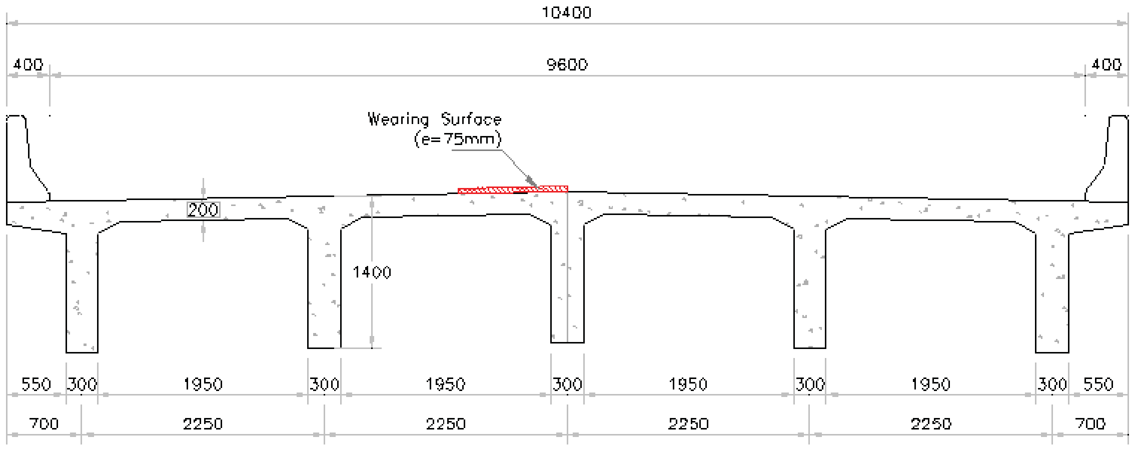
The Pushover consists of applying a lateral load pattern that properly represent the action of the earthquake, which increases slowly and provides, step by step, the formation of plastic hinges until the bridge collapses. Finally, the displacement capacity of the bridge will be obtained from this NSA.

In this paper, it describes the bridges under study and indicates the parameters used to define the family of bridges considered, the description of their modeling and the definition of nonlinear parameters. Subsequently, we performed a NSA - Pushover, to obtain the deformation capacity of the bridge piers and finally to find a correlation between the displacement capacity and the height of the pier.

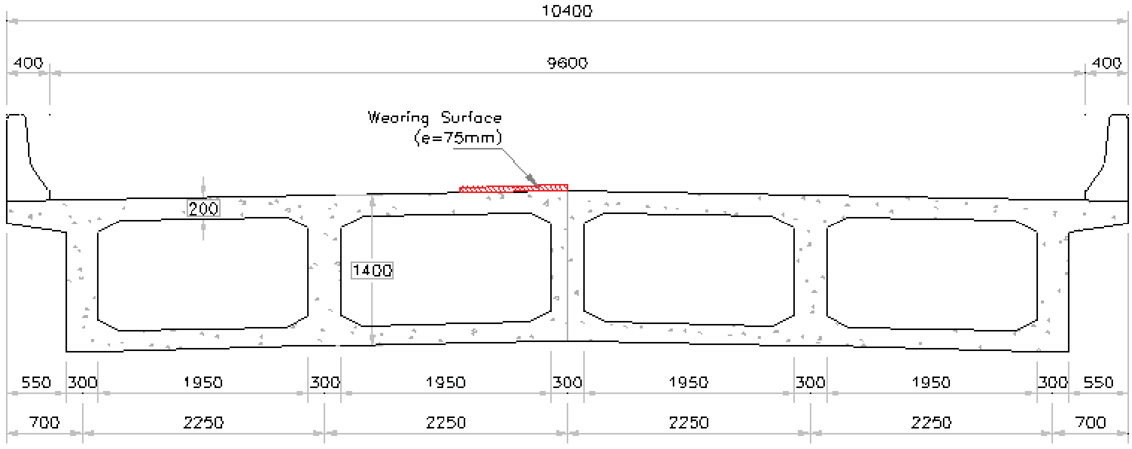
The structural analysis of the bridge family was performed using the CSIBridge [7] software, which is a widely used tool worldwide for seismic design of bridge.

2. Characteristics of the bridges

The bridges under study have a superstructure of 90.00 m of length consisting of three continuous spans simply supported of 30.00 m each span, where the positive moment section is made up of T-beams and the negative moment section by box beams (Figure 1). The beams of the superstructure are made of post-tensioned concrete of f'_c = 350 kg/cm^2. The longitudinal beams are transversely connected by a total of three intermediate diaphragms for each span and one diaphragm in each support, in order to give unity to the longitudinal beams.



a)

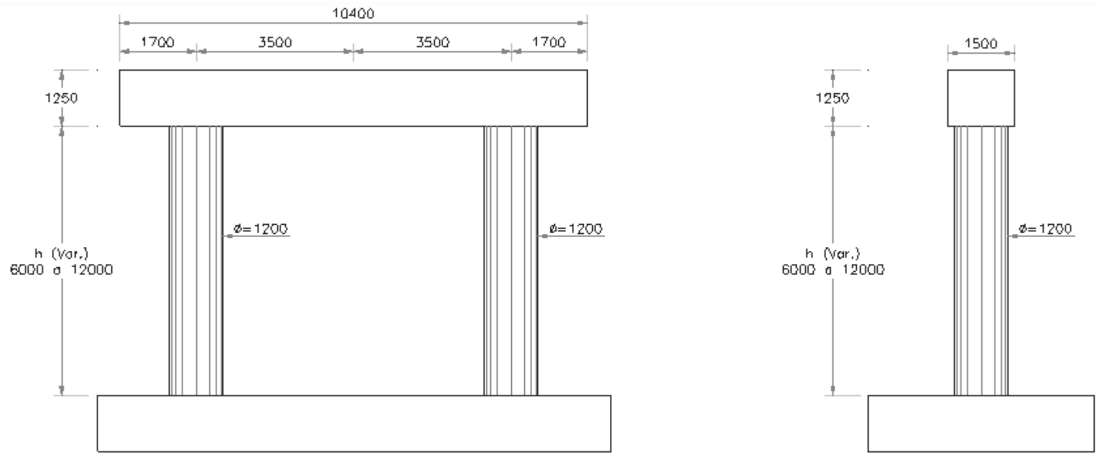


b)

Fig. 1 - Typical section of the superstructure. a) Positive moment section b) Negative moment section

The substructure is made up of multi-columns of concrete $f'_c = 210 kg/cm^2$, with a circular section of 1.20 m in diameter and connected to each other, the upper end by a cap beam of rectangular section of 1.25 m x 1.50 m. There are two types of substructure, type I formed by two columns and type II formed by three columns, in the bridges under study (Figure 2).

Furthermore, different free heights for the supports have been considered, such as 6 m, 8 m and 10 m, as well as the combination of these heights in the different interior supports (Figure 3). In the analysis of the bridges, it was considered that the joints in the abutments have sufficient capacity to control the longitudinal displacements produced by the superstructure, and the shear keys between the supports located at the abutments in order to control the transverse displacements. Therefore, in the modeling of the seismic control devices in the abutments, it was considered that these are be restricted to transverse displacement.



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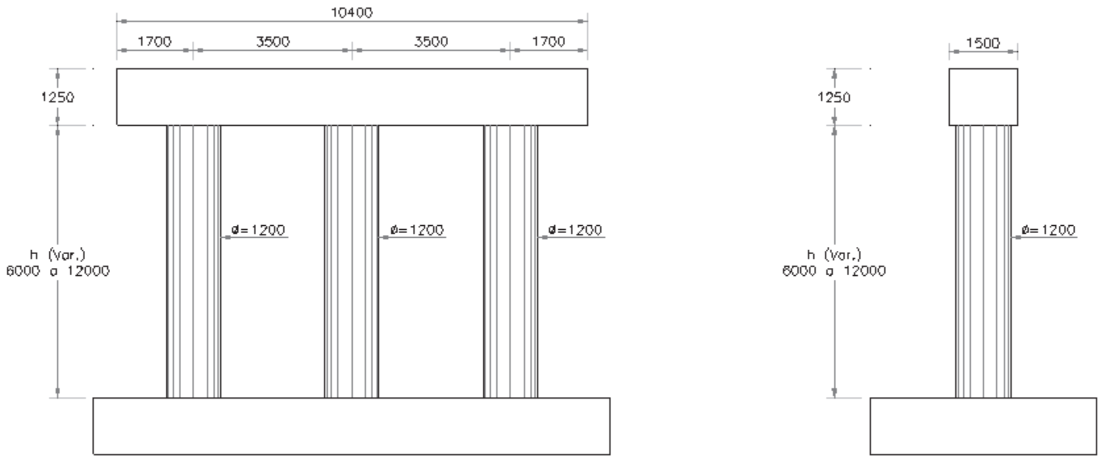


b)

Fig. 2 - Typical section of the substructure. a) Type I - 2 columns. b) Type II - 3 columns

Another principal parameter that has been considered in the bridges under study is the amount of vertical steel, with assigned values equal to ρ=1%, 2%, 3% and 4% of amount. The amount of horizontal steel is set in 0.0104 and complies with the minimum requirements of the AASHTO regulation [8, 9].

All the proposed dimensions for the superstructure and infrastructure have been designed for the resistance limit state according to the AASHTO regulation [9], giving validity to the models proposed in this paper.

Taking into account the parameters such as: a, b, c; a total of 48 cases of parametric models of reinforced concrete bridges were obtained.

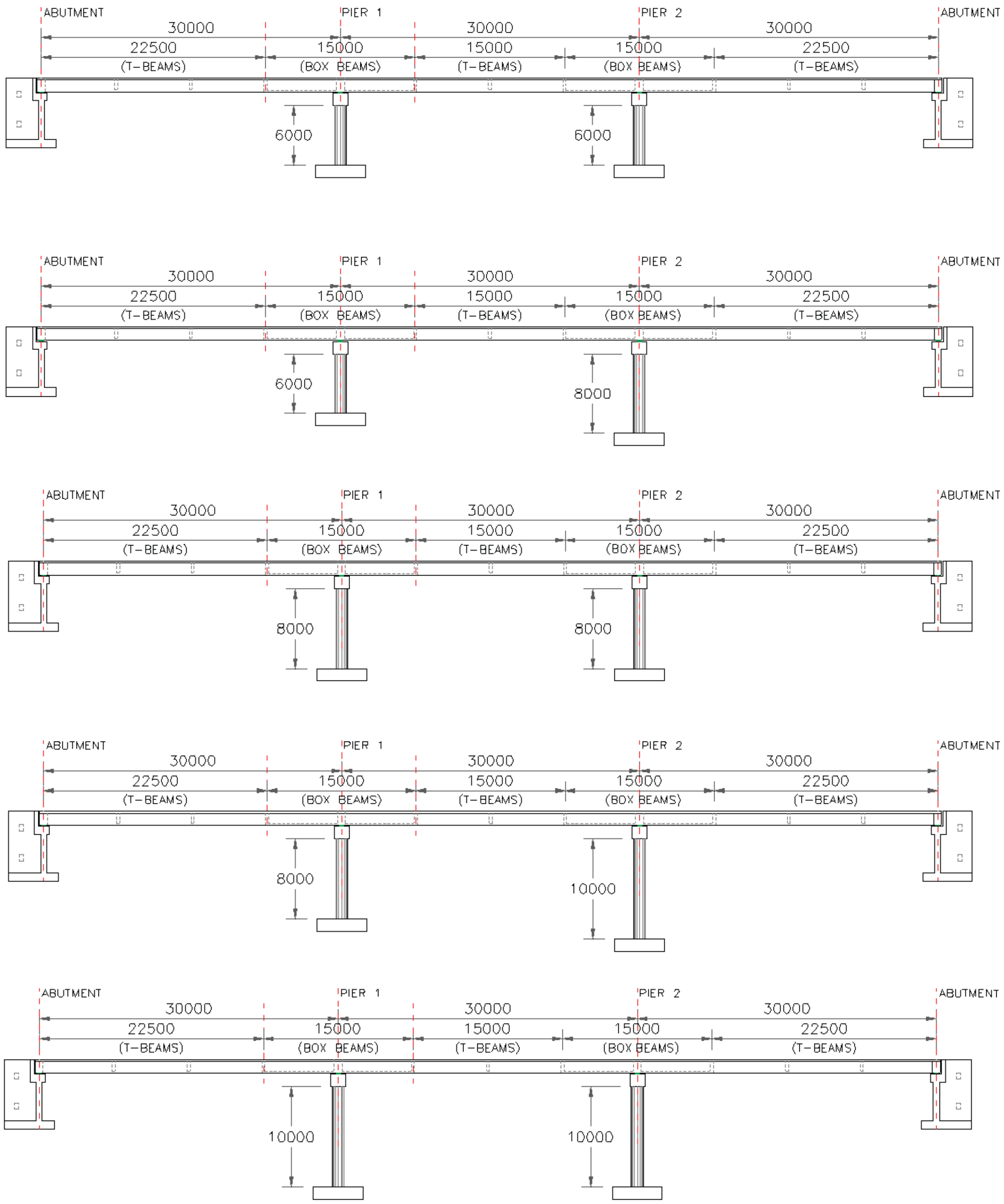


Fig. 3- Typical elevation of the bridges under study

3. Bridge Modeling

A three-dimensional typical model of a reinforced concrete bridge using the CSIBridge computational tool [7] is shown in Figure 4.

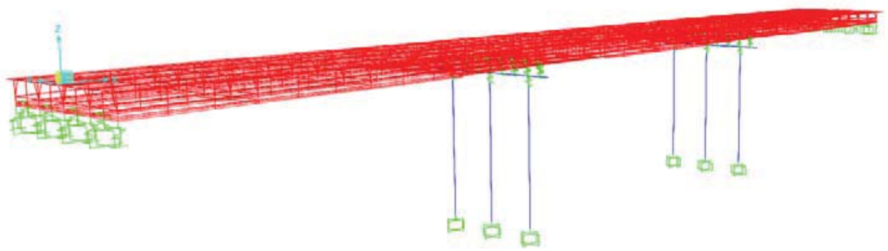


Fig. 4 - Bridge Modeling in CSIBridge [7].

3.1 Superstructure.

The slab and the beams of the superstructure were idealized with Shell-type area elements, all the loads corresponding to its service stage have been considered on the board as weight of wearing surface, barriers and two mobile load lines corresponding to the HL-93 load. The superstructure is expected to have an elastic behavior during a seismic event, so the properties of the superstructure have not been modified for the performed analyzes.

3.2 Columns.

As indicated in previous paragraphs, the columns are of circular section with a diameter of 1.20 m, with variations of vertical amount of 1%, 2%. 3% and 4% as shown in Figure 5. The columns have been modeled with frame-type elements, to which their properties have been modified according to the recommendations to the AASHTO regulation [8]. The effective properties considered were: The effective moment of inertia for column, which was calculated using the idealized bilinear moment-curvature diagram (Figure 6), taking into account the axial load corresponding to each column and the effective torsional moment of inertia as the 0.2 of the gross torsional moment of inertia of the section. Additionally, the columns are considered to be fixed into the base.

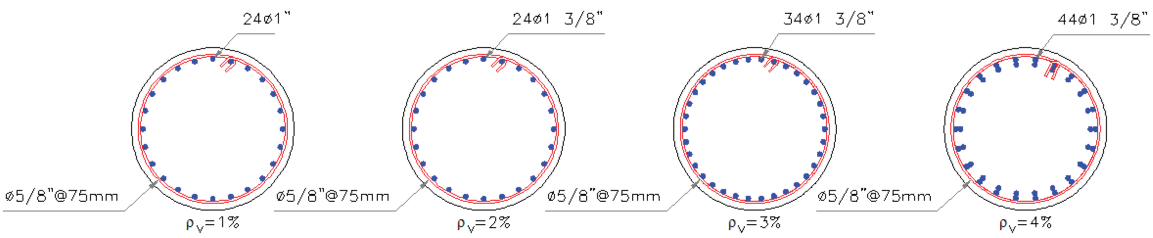


Fig. 5 - Column sections

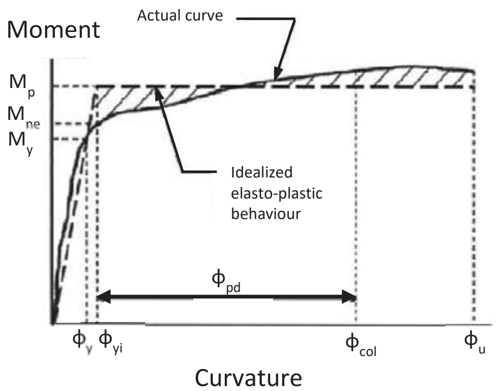


Fig. 6 - Moment-curvature diagram [8]

3.3 Elastomeric bearings.

The bearings are reinforced elastomers, modeled with link type elements and with an elastic stiffness both in the longitudinal and transverse direction equal to:

$$k = GA/h \tag{1}$$

Where G is the shear modulus of elastomer, A is the cross-sectional area of the elastomer and h is the effective height of the elastomer.

3.4 Restriction to the lateral displacement.

It is considered in the intermediate piers that the superstructure of the bridge has transverse and longitudinal restriction to the bridge, the modeling of these restrictions has been carried out by means of link type elements located between two consecutive beams (Figure 7). It is considered that there is only restriction in the transverse direction in the supports at the ends of the bridge.

3.5 Plastic hinge.

The plastic hinge used for the NSA-Pushover, is a hinge with concentrated plasticity. In the case of bridges, the plastic hinges should be produced on the piers and in this case in particular the plastic hinge is at the base of the pier; and it is at the ends of the pier for the longitudinal direction and for the transverse direction. The plastic hinges have a length that is defined according to AASHTO [8] as:

$$L_p = 0.08L + 0.15\, f_{ye}d_{bl} \geq 0.3 f_{ye}d_{bl} \tag{2}$$

Where "L" is the distance in inches, from the point of maximum moment to the point of inflection, "d_{bl}" is the diameter of the longitudinal reinforcing bar also in inches and "f_{ye}" is the expected yield strength of the longitudinal reinforcing. Therefore, the location of the plastic hinge for modeling will be at the center of the plastic hinge calculated according to equation 1. We use the Caltrans type hinge [1] that is defined in the CSIBridge program [7], which is more recommended for concrete columns and also give more conservative results than the hinges of FEMA356 [10, 11, 12, 13].

3.6 Lateral load pattern.

The lateral load pattern considered in the model for the NSA-Pushover is the one corresponding to a horizontal (gravity) load in the longitudinal direction, in both directions, and in the transverse direction, applied to the superstructure of the bridge. This consideration is valid since the participation of the mass in the fundamental longitudinal and transverse modes is greater than 75%, according to the recommendations of FEMA 356 [10] and as indicated in [14].

3.7 Mass Source.

The mass source considered for the analysis is the own weight of the structure, loads of wearing surface and barriers, 50% of the live load.

3.8 P-Delta.

From the performed studies [15, 16], it can be noticed that the recommendations given by Caltrans [1] and AASHTO [8] to ignore the P-Delta effects are not entirely correct.

$$P_{dl} \cdot D_r \leq 0.2 \cdot Mp^{col} \tag{3}$$

Where, P_{dl} is the unfactored dead load that acts on the column, Dr is the relative displacement between the point of contraflexure and the furthest point of the plastic hinge, and Mp^{col} is the plastic moment of the column in the section of the plastic hinge considering the properties of the expected materials.

However, for cases where the height to diameter ratio for the column (H/D) is low or medium (less than 9) and the axial load levels are low or medium ($< 15\% \cdot f'_c \cdot A_g$), the results obtained are acceptable. The analyzed bridges have an H/D ratio of less than 9 and an axial load of less than $15\% \cdot f'_c \cdot A_g$, so the results obtained are considered acceptable.

3.9 Ductility.

In all cases, the capacity of displacement of the piers has been limited, in order to obtain a ductility of $\mu < 5$, which is the ductility limit for multi-column piers and it has been verified that the capacity of ductility of the piers is $\mu > 3$, to ensure a rotational capacity in the plastic hinge area, as indicated in the Caltrans [1].

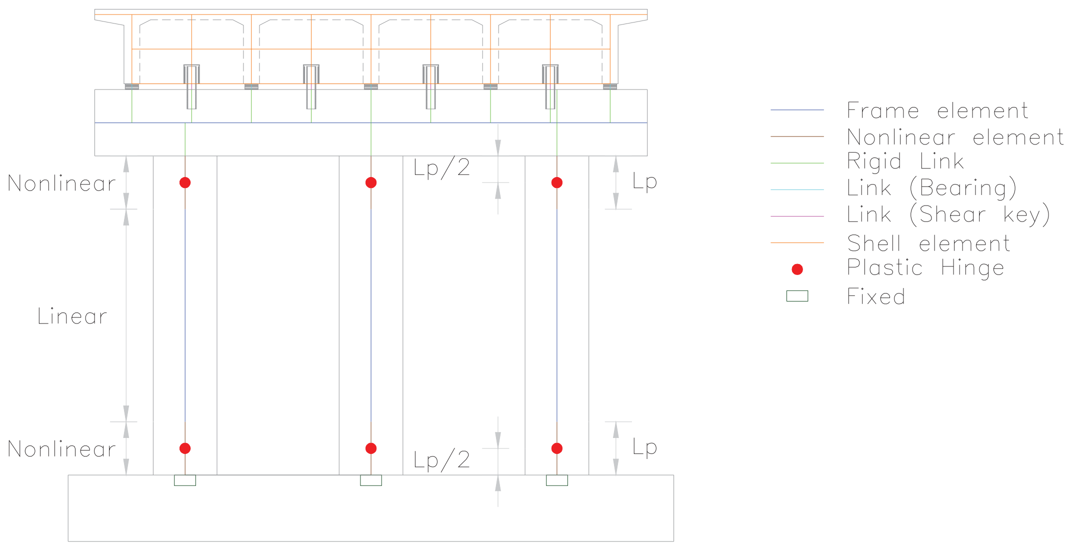


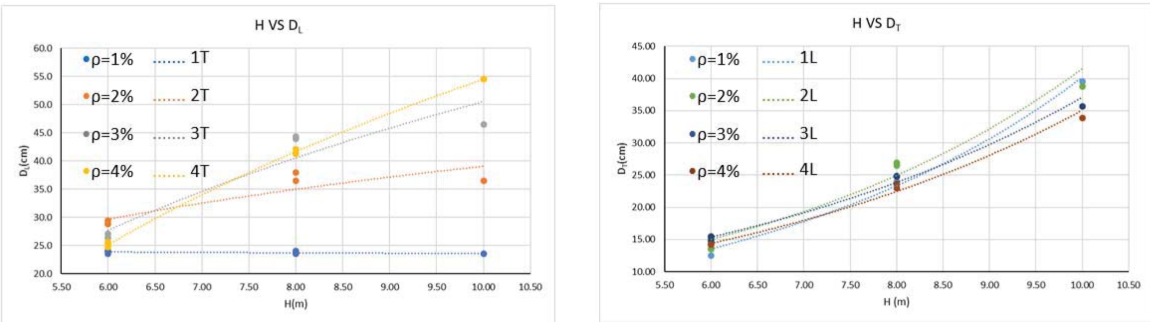
Fig. 7 - Typical Modeling of Pier

4. Analysis of results.

Given the large number of cases under study, it was chosen a parameter that has a better correlation with the lateral displacement of the bridge, this parameter was the height of the piers. The control point and data collection for all cases analyzed is the upper end of the free height of the external column.

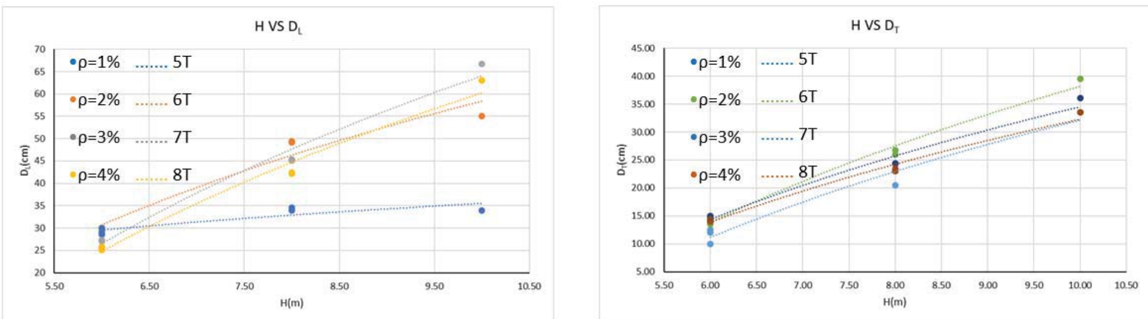
After performing the NSA-Pushover of the bridges under study, the capacity for longitudinal and transverse displacement for each case was obtained. The longitudinal displacement corresponding to the height for the piers Type I for an amount of 1%, 2%, 3% and 4% respectively is shown in the Figure 8a; and the transverse displacement corresponding to the height for the piers Type I for an amount of 1%, 2%, 3% and 4% respectively is shown in the Figure 8b. Likewise, the responses were represented in the graphs (Figure 9a and 9b), corresponding to bridges with piers Type II.

From the graphs, it can be noticed that lateral displacements in the longitudinal direction increase as the vertical amount of the columns increases, while in the transverse direction it decreases as the vertical amount increases. This difference is due to the fact that there are limitations not only of ductility, but also the not need to consider the P-Delta effect in the longitudinal direction; while that only the ductility verification is critical in the transverse direction.



a) b)

Fig. 8 - Height VS Lateral Displacement - Pier Type I



a) b)

Fig. 9 - Height VS Lateral Displacement - Pier Type II

Table 1 shows the equations found for the calculation of the displacement capacity for piers Type I and II, complying with the requirements of ductility and the verification of not need to consider P-Delta effects [17].

Table 1 - Equations for the calculation of displacement capacity of piers Type I and II.

		LONGITUDINAL	*TRANSVERSE*
		H ∈ [6.00-10.00]	*H ∈ [6.00-10.00]*
PIER TYPE I	*1*	1L: D = -0.556ln(H)+24.846	1T: D = 46.821ln(H)-71.119
	2	2L: D = 18.356ln(H)-3.1987	2T: D = 45.792ln(H)-67.634
	3	3L: D = 44.722ln(H)-52.446	3T: D = 38.359ln(H)-53.873
	4	4L: D = 57.503ln(H)-77.904	4T: D = 36.559ln(H)-51.620
PIER TYPE II	*5*	5L: D = 11.653ln(H)+8.677	5T: D = 41.082ln(H)-62.462
	6	6L: D = 54.157ln(H)-66.314	6T: D = 47.658ln(H)-71.563
	7	7L: D = 73.512ln(H)-105.22	7T: D = 39.443ln(H)-56.289
	8	8L: D = 69.44ln(H)-99.66	8T: D = 36.031ln(H)-50.673

5. Example of application

The NSA-Pushover of a bridge is presented to obtain its lateral displacement capacity and is compared with the lateral displacement obtained of the developed formulas. The bridge under study is of 3 spans of 22.60 m, with a width of 14.80 m and columns with a vertical amount of 2.85% and a height of 9.60 m. For the application of the formulas, the columns should be reinforced concrete with $f'_c = 210$ kg/cm^2, diameter of 1.20 m and a reinforcing steel of 5/8" @ 0.075m.

The bridge modeling was made in the CSIBridge [7], as shown in Figure 10a for the bridge with Pier Type I and Figure 10b for the bridge with Pier Type II. First, the resistance based design of the columns was carried out and the nonlinear properties of the materials were defined, according to the AASHTO regulation [4, 5]. Then, the NSA-Pushover was carried out, following the indications of item 3.00.

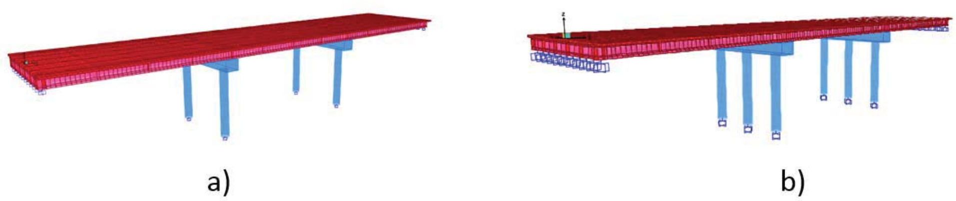


Fig. 10 - Model of the bridges with Pier Type I and Pier Type II.

From the NSA-Pushover, they were obtained for the pier Type I: the longitudinal displacement capacity is 47.10 cm, and 36.69 cm in the transverse direction. For the pier Type II: the longitudinal displacement capacity is 66.52 cm, and 36.45 cm in the transverse direction.

For the application of the formulas, we have to choose the equations corresponding to the vertical amount of the columns of the bridge. In this particular case, there is an amount of 2.85%, therefore, we interpolate between the equations corresponding to the amounts of 2% and 3%, these being the equations 2L, 3L for the longitudinal direction and 2T, 3T for the transverse direction of the bridge with pier Type I; and equations 6L, 7L for the longitudinal direction and 6T, 7T for the transverse direction of the bridge with pier Type II. The results obtained by applying the indicated formulas are shown in Table 2.

Table 2 - Calculation of lateral displacements for piers Type I and II.

		D_L (cm)	$D_{L\_Pushover}$ (cm)	D_T (cm)	$D_{T\_Pushover}$ (cm)
PIER TYPE I	*ρ=2%*	39.07	----	38.78	----
	ρ=3%	50.53	----	34.53	----
	ρ=2.85%	48.81	47.10	35.17	36.69
PIER TYPE II	*ρ=2%*	58.39	----	38.17	----
	ρ=3%	64.05	----	34.53	----
	ρ=2.85%	63.20	66.52	35.08	36.45

(--) Not data

From the results shown in Table 2, we can observe that the lateral displacements obtained of the given formulas in Table 1, have a margin of error of less than 5% compared to the NSA-Pushover performed.

6. Conclusions.

The formulas developed for the calculation of the lateral displacement of bridges with reinforced concrete piers, are valid for bridges of two and three spans with symmetric superstructures, and the dimensions of the piers must be within the range of values assigned to the parameters defined in this paper. The main conclusions were the following:

i. The application of the formulas in Table 1 shows a 5% error in the calculation of the lateral displacements of reinforced concrete piers, both for the transverse and longitudinal directions, compared to the NSA-Pushover.

ii. In symmetrical spans, the length and width dimensions of the bridge board do not influence the application of the developed formulas, as long as the axial load on the columns is not more than $15\%\cdot f'_c\cdot A_g$.

iii. In bridges with piers of different heights, it was observed that the shorter pier is critical because it collapses before the remaining piers. Therefore, for cases of bridges with piers of different heights, the formulas in Table 1 with H equal to the lowest height of the remaining piers should be used.

iv. The lateral displacements obtained with the formulas in Table 1, meet the ductility requirements and the criterion for neglecting the P-Delta effects ([1], [8]).

7. Acknowledgments

We would like to express our deepest appreciation to all those who contributed to the development of this paper and also to the Instituto de la Construcción y Gerencia (ICG) for permit us to use the CSIBridge program [7].

8. Copyrights

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