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Instrumentation and response of drilled piers under overturning moments for a viaduct in the Metropolitan Area of Mexico City

Instrumentation et réponse des piles forées sous moments de renversement pour un viaduc dans la zone métropolitaine de Mexico

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ABSTRACT: To the Northwest of Mexico City, a 22 km-long elevated highway called the Bicentennial Viaduct was built. Its superstructure was solved with Gerber-type prestressed concrete beams. The girders are supported each 35 m approximately by precast columns; each column was monolithically cast in plant with its footing. Each footing has four cylindrical holes through which the drilled and cast-in-place piers were connected. The geotechnical instrumentation of a support of this viaduct is described in this paper; it includes strain deformimeters placed along the piers, and pressure cells under the footing. The monitored site is characterized in the first few meters by a clay layer of medium to firm consistency; below, compact sandy silt and sandy strata are present. Once the work was completed, some tests were carried out to verify the design hypothesis, measuring the response of the foundation and superstructure to different loading conditions. In the studied support in particular, static lateral load tests were carried out, appling forces to the upper part of the column, through a heavy crane. The response of the foundation piers under these overturning moments is described and discussed in this paper. It could be shown that the resistant moments of the foundation in response to the overturning moments caused by the applied lateral loads, were predominantly resisted by the piers; no more than 10% was contributed by the footing base. It was measured that the load transfer mechanism in the piers was by lateral friction. Also, it was appreciated that for the significant applied moments, the tip of the piers is practically not requested.

RÉSUMÉ: Au nord-ouest de Mexico, une autoroute surélevée de 22 km de long appelée le viaduc du Bicentenaire a été construite. Sa superstructure a été résolue avec des poutres en béton précontraint de type Gerber. Les poutres sont supportées chacune 35 m environ par des poteaux préfabriqués; chaque colonne a été coulée monolithiquement dans le chaussure de fondation. Chaque chaussure comporte quatre trous cylindriques à travers lesquels les piliers forés et coulés en place ont été connectés. L'instrumentation géotechnique d'un soutènement de ce viaduc est décrite dans cet article; il comprend des déformimètres de contrainte placés le long des piles et des cellules de pression sous la semelle. Le site surveillé est caractérisé dans les premiers mètres par une couche argileuse de consistance moyenne à ferme; en contrebas, des limons sableux compacts et des strates sableuses sont présents. Une fois les travaux terminés, des tests ont été effectués pour vérifier les hypothèses de conception, en mesurant la réponse de la fondation et de la superstructure à différentes conditions de chargement. Dans le support étudié en particulier, des essais de charges latérales statiques ont été effectués, en appliquant des charges à la partie supérieure de la colonne, à travers une grue lourde. La réponse des piles de fondation sous ces moments de renversement est décrite et discutée dans cet article. Il a pu être démontré que les moments résistants de la fondation en réponse au moment de renversement causé par la charge latérale appliquée étaient principalement dus aux piliers; pas plus de 10 % ont été apportés par la base de la chaussure de fondation. On a pu mesurer que le mécanisme de transfert de charge dans les piles était par frottement latéral. Aussi, il a été apprécié que pour les moments significatifs appliqués, le bout des piles n'est pratiquement pas sollicité.

KEYWORDS: drilled piers, overturning moments, geotechnical instrumentation, viaduct.

1 INTRODUCTION

The Metropolitan Area of Mexico City (ZMCDMX) increasingly requires fast, safe and inexpensive communication routes. Such is the case mainly for its northwest area where a large number of industrial and urban developments are located, as well as the main connection to the center and northwest of the country, and even for the United States. To meet this objective, viaducts over wide avenues already in operation have been added in recent decades. More than 70 km (Murià-Vila et al., 2015) of these

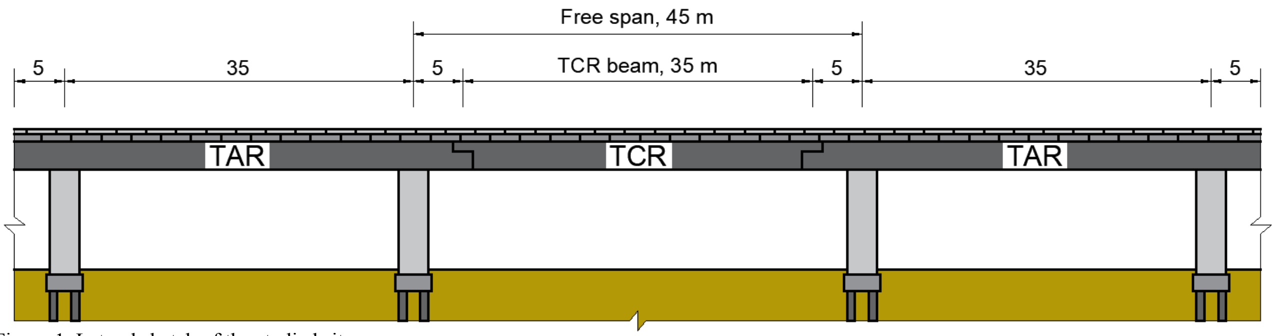


Figure 1. Lateral sketch of the studied site.

elevated structures have been built in the ZMCDMX, which has improved vehicular traffic. The studied site is located in Section 3, from Santa Mónica to Valle Dorado (4.3 km) of an elevated road over the Periférico Norte, with extreme sites Cuatro Caminos in Mexico City and Cuautitlán Izcalli in the State of Mexico, with a total length of 23.5 km.

The body of the superstructure is of the Gerber type, with beams made up of a trapezoidal box in which the 35 m long central isostatic beams (TCR) are supported by the cantilevered ends of the 45 m long support beams (TAR), see Figures 1 and 2. The TAR beams form a frame with the columns 35 m apart, thus maintaining a cantilevered section of 5 m on both ends. A very peculiar feature of this project was that each precast and prestressed, hollow-core rectangular column (PCF) was monolithically integrated with its footing, which is prepared for the connection to four cast-in-situ drilled piers. This novel characteristic of using a column-footing unit, together with the use of precast concrete for almost all the elements of the superstructure and built on near plants, allowed to carry out the construction in a very short period of time; even more so if it is taken into account that all assembly processes were restricted to running at night (11 pm to 5 am) see Figure 3, since the Periférico road was always open to traffic during most of the day in both directions.

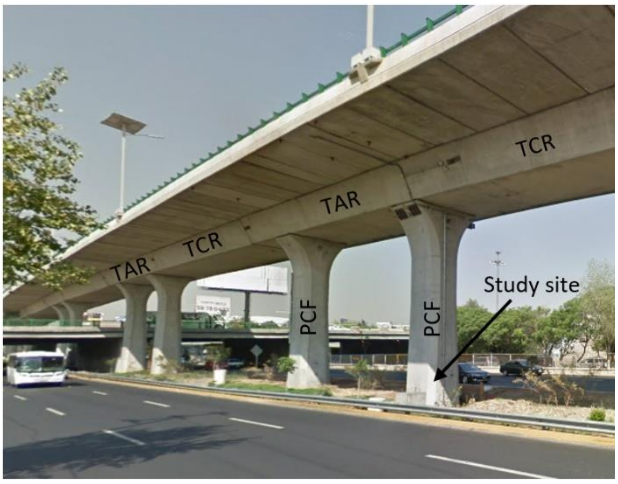


Figure 2. General view of the studied site.

The footing-column weighs around 1,200 kN and was manufactured with a concrete of resistance f'c = 60 MPa. The footing measures 4.60 m by 3.60 m and is 1.70 m height. The four foundation piers are 0.80 m in diameter and of variable lengths, taking into account the stratigraphic conditions along the road. At the site in question, they were 23 m long, they were cast in situ with a concrete of resistance f'c = 25 MPa. The top of the reinforcing steel bars of the piers was introduced to the cylindrical holes of the footings, in order to structurally link them to the footing-column unit; additionally, a high-strength steel device (spike) was introduced to the fresh concrete of the piers for anchoring. Once the concrete of that connection had set and having reached a certain resistance, it was post-tensioned to ensure a correct union between the foundation piers and the footing-column unit.

2 GEOTECHNICAL CONDITIONS

Marsal and Mazari (1969) proposed a geotechnical zoning for Mexico City (CDMX). Currently, the mandatory regulations in CDMX, largely following what was proposed by these authors, and with updated information, subdivides it into three zones (Figure 4): Hills (zone I, green), Transition (zone II, yellow) and Lake (zone III, blue). The site under study is located on the border between the hills and the transition zones. In the transition zone, in general, the organic clay or silty deposits of the Becerra formation are superficially present, covering highly compressible volcanic clay strata of variable thickness, interspersed with layers of compact silty sand or clean sand, which rest on thick layers in which the predominant material is gravel and sand.

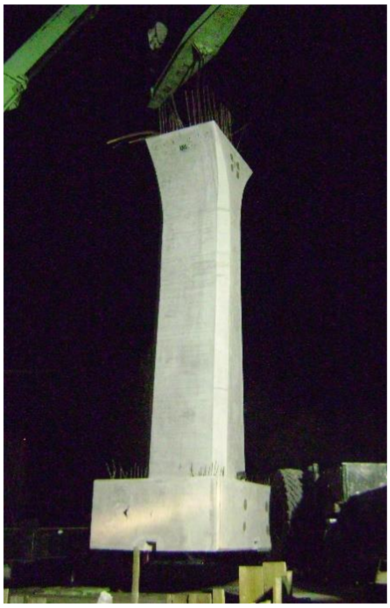


Figure 3. Mounting the footing-column unit during the early morning.

At the studied support, it was found a sandy clay in the first 11 m, with 10 to 18 blows in a SPT boring. Sandy silts with medium relative density were detected from 11 m down to 23 m depth. Below 23 m depth, silty sands of medium to high relative density were detected, with greater than 50 blows (Figure 5).

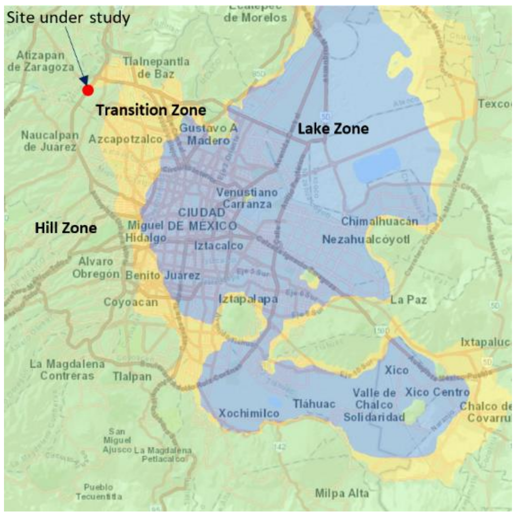
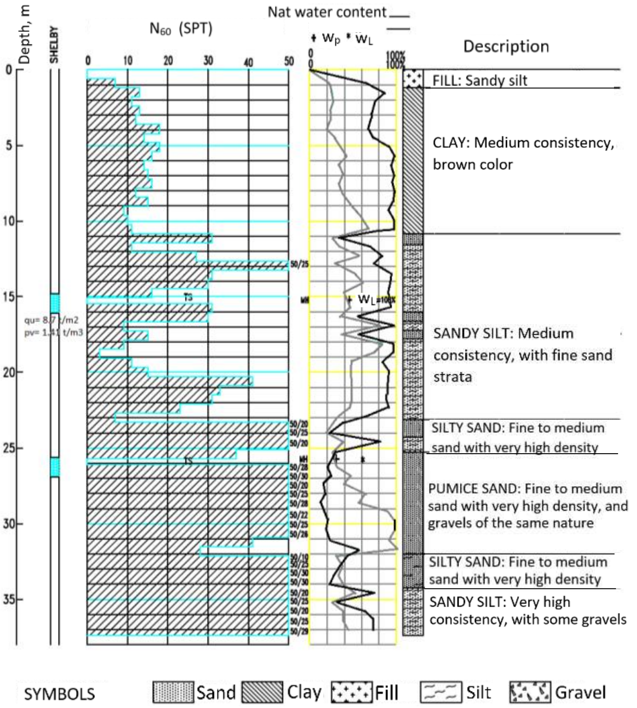


Figure 4. Location of the site in the geotechnical zoning of the current regulations of Mexico City.

Below these fine materials, from a depth of 23 m to 37 m, the maximum exploration depth, there is a sequence of granular materials formed by silty sands and fine to coarse pumice sands. As can be seen in Figure 5, greater than 50 blows was found with the SPT boring. No groundwater table was detected, both in the borings and during drilling tasks for the piers.

Figure 5. Stratigraphic profile of the studied site.



Given the difficulty of obtaining undisturbed samples in sandy silt, phycometer tests were performed. With such equipment, internal friction angles of 34° to 40° and undrained shear resistance of between 30 and 60 kPa were obtained; Table 1 summarizes the results of the performed tests.

Table 1. Summary of mechanical properties measured using the phycometer.

Depth	Parameters	
m	c_u (kPa)	Friction angle (degrees)
21	30	35
24	50	34
25	40	34

It should be noted that it was chosen to study this site since it corresponds to a stratigraphy with rare clay formations in the entire line of the Bicentennial Viaduct.

In the disturbed samples obtained by field exploration, the index properties presented in Table 2 were determined in the laboratory.

Table 2. Summary of index properties of the explored soils at the site

Boring No.	Sample depth from	to	USCS	Atterberg limits w_L (%)	w_P (%)	PI (%)	Granulometry G (%)	S (%)	F (%)	G_s	e
SM-EM-02	3.6	4.2	CL	40	19	21				2.41	
	8.8	9.2	CL	34	20	14				2.46	
	13.4	14.4	CH	66	40	26				2.64	
	19.2	20.2	CH	96	34	62				2.26	
	25	26					0	57	43	2.33	
SM-1	2.5		MH	29						2.65	0.98
	6.2		MH	42						2.66	1.23
	9.3		MH	40						2.63	1.20
	11.6		SC	30						2.57	1.00

Some undisturbed and unsaturated samples were obtained in the clay strata. Unconsolidated undrained triaxial tests (UU) were carried out with these samples, in which an apparent cohesion of between 58 and 70 kPa, and a friction angle of between 16 and 36° were obtained. The results are gathered in Table 3.

Table 3. Summary of resistance properties of the explored soils at the site

Boring No.	Sample depth from	to	Triaxial UU c_{uu} (kPa)	ϕ_{uu} (degrees)	E (kPa)	γ (kN/m^3)
SM-EM-02	3.6	4.2	30	5		17.5
	8.8	9.2	50	0		18.2
	13.4	14.4	100	8		15.3
	19.2	20.2	110	2		14.6
	25	26	40	0		17.4
SM-1	2.5		60	33	19000	17.5
	6.2		70	36	18000	17
	9.3		58	16	16000	17
	11.6		40	39	35000	16.5

3 INSTRUMENTATION OF THE FOUNDATION

To know the geotechnical variables involved in the behavior of this foundation, a sensor system was arranged. This system was designed to measure those variables under static conditions, and allowing to have some redundancy. Professor Ralph B. Peck stated that "… each instrument installed in a project must be selected and located to help answer a specific question. Following this simple rule is the key to successful field instrumentation". In accordance with this idea, the following questions were posed:

- How is the load transfer mechanism along the piers?
- What is the contribution of the tip of the piers to the load capacity?, and
- What is the pressure under the footing?

According to Dunnicliff (1988) the first six stages of a planning process for geotechnical instrumentation are the following:

- Definition of the project conditions,
- Predict the mechanisms that will govern behavior,
- Definition of geotechnical questions that need to be answered,
- Define the purpose of the instrumentation,
- Select the parameters to monitor,
- Predict the magnitude of the changes,

Following what was previously established, the project conditions were studied to define the type of instrumentation to be used. It was considered that the piers would work mainly per skin resistance and that the contribution of the tip would be minimal. Indeed, as has been measured in load tests (Mendoza et al., 2008 and 2010, Fellenius, 2014, Ibarra et al., 2016,) the load capacity per tip comes into play until the capacity per shaft is exhausted, and large penetration of the piers occurs.

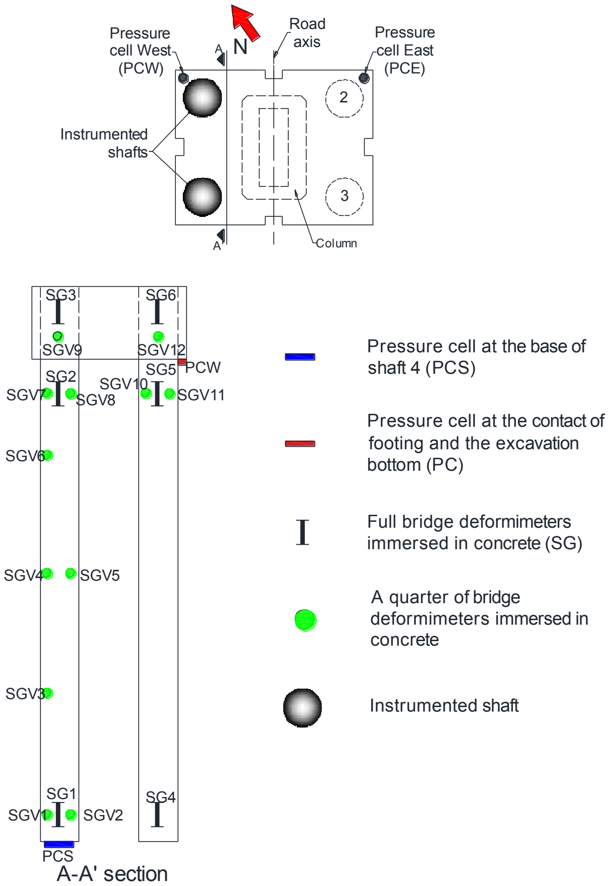
So, the main objectives of the instrumentation placed were to measure the following variables:

- Load transfer along two foundation piers,
- Contribution to the load capacity of the tip of two piers,
- Pressure under the footing.

To meet these objectives, the geotechnical instruments listed below were installed (Figure 6):

- Six (6) full-bridge electrical strain gauges (SG) embedded in the concrete. Two (2) at one meter from the tip of piers 4 and 1, two (2) at the footing-piers connection and two (2) at a meter of the head of the piers.
- Three (3) full-bridge transducer pressure cells. One (1) at the tip of pier 4 (CPP) and two (2) just below the footing.
- Twelve (12) quarter-bridge electrical strain gauges (SGV) embedded in the concrete. Two (2) one meter from the tip of pier 4, four (4) were placed in intermediate sections of pier 4 and six (6) at the footing-pier connections on piers 4 and 1.

Figure 6. Location of the instruments in the foundation of the support.



The main difference between quarter-bridge and full-bridge sensors is their stability to variation in temperature (Dunnicliff, 1988). The quarter-bridge ones are somewhat unstable, but if the temperature is kept constant, they are quite reliable, as can be corroborated in the measurements of load tests (Mendoza et al., 2010).

The strain gauges were fixed directly in the main reinforcement bars of the piers (Figure 7). To place the pressure cell, a steel plate was welded to the tip of the assembly (Figure 8). The plate had cutouts that allowed the concrete to enter, as well as perforations to properly fix the pressure cell. The cables of all the instruments were protected with flexible plastic tubing, leading them to the top of the pile.

Figure 7. Quarter-bridge strain gauge.attached to the steel reinforcement.

4 DESCRIPTION OF THE TEST

A test was carried out on the studied support to evaluate the conditions of both the superstructure (Murià-Vila et al., 2015 and López, 2012) and the foundation when faced with a static lateral force. A pair of plates were installed at the top of the column to apply a lateral load to it, using a crane with a capacity of up to 4900 kN (Figure 9). The lateral action was increasing and was measured with a load cell, perpendicular to the axis of the line and with an inclination of 18° with respect to the horizontal. A maximum effective horizontal load of 533 kN in the west direction was calculated.

Figure 8. Installing the pressure cell at the tip of the steel reinforcement of pier 4.

During load application, quarter-bridge strain gauges were monitored with model P3 manual consoles. The responses of the full-bridge strain gauges and pressure cells were also measured. Readings were recorded every second.

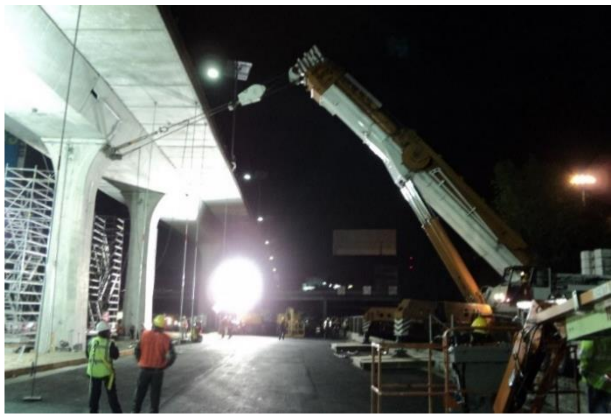


Figure 9. Application of side force by crane.

5 TEST RESULTS

The unit microdeformations recorded by quarter-bridge and full-bridge Wheatstone strain gauges made it possible to quantify the stresses in the concrete, since both were embedded in it. Assuming an elastic material, the following equation was applied:

$$\sigma = E \cdot \varepsilon \tag{1}$$

where:
σ is the inferred stress in the concrete,
ε measured microdeformations,

E modulus of elasticity of concrete, obtained with the following expression, which was supported by experimental tests:

$$E = 4400\sqrt{f'_c} \tag{2}$$

The stresses calculated in the piers, based on the measurements of the strain gauges, are depicted in Figures 10 to 12. It should be noted that the reported measurements assign a zero value to the static load initial condition for the support due to its own weight.

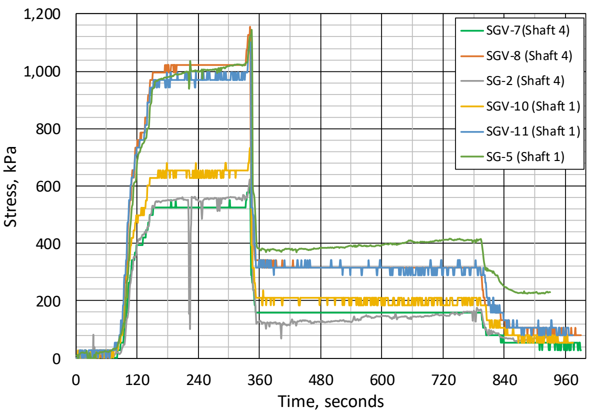


Figure 10. Stress measured in the concrete, one meter from the head of the piers.

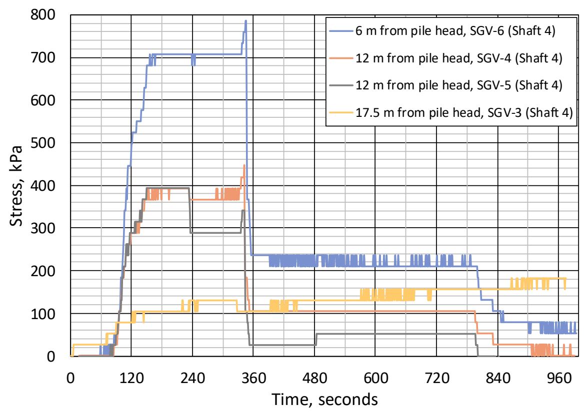


Figure 11. Stress measured in the concrete, at different depths of the piers (6, 12 and 17.5 m).

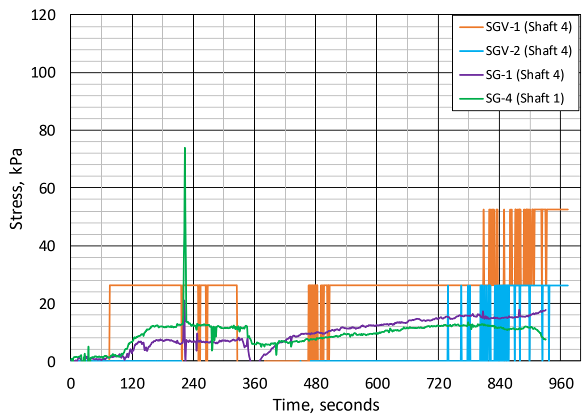


Figure 12. Stress measured in the concrete, one meter from the tip of the drilled pier.

The following stages can be recognized: zero load, gradual load increase, first sustained load (555 kN inclined load, applied to the column), first discharge, second sustained load, second discharge and residual load. The latter corresponds to the remaining load after unloading the column.

As expected, the transmitted load in the piers decreases with depth. To better visualize the measurements, the scale of the graph that shows the stresses measured at the tip (Figure 13) by means of the pressure cell shown in Figure 8, is an order of magnitude smaller than that of the other graphs.

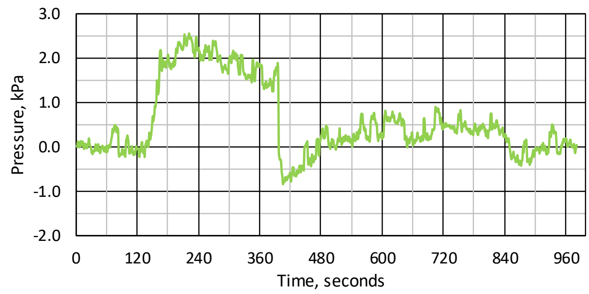


Figure 13. Vertical pressure at the tip of the pier 4.

The resolution of the quarter-bridge gauges is lower than that of the full-bridge gauges, which is more noticeable in the stresses recorded at the tip of the pier, which were very low. In view of this, only the better resolution of the full-bridge strain gauges (SG) allowed us to measure more clearly what happened right at the tip; not so with the quarter-bridge.

Despite being in the same location along the length of the piers, the stress measurements show a small dispersion, which could be related to the plant orientation of the sensors; therefore, it was decided to use an average value of the measured stresses. With this mean value, the load transfer graph was drawn against the depth, and is shown in Figure 14. The maximum load was measured precisely at the junction of the footing with the pier (zero depth) and was 677 kN. Near the tip, the recorded maximum load that was 22 kN (Figure 14) that is, 3% of the maximum load measured at the connection of the pier with the footing. It can be distinguished that despite removing the load at the top of the column, the pier remains charged.

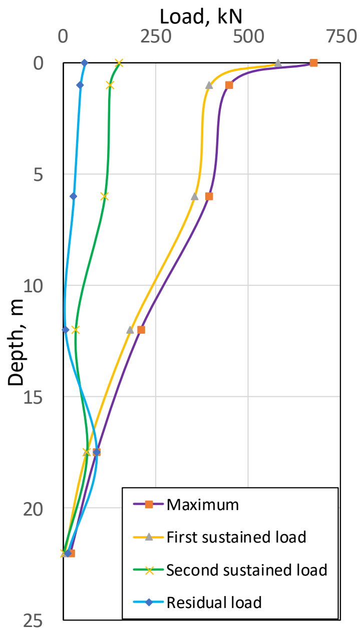


Figure 14. Load transfer curves for different stages of the test.

On the other hand, Figure 15 shows the measurements made with the pressure cells, under the footing. Like the records shown previously, the measurements with the pressure cells start from a zero that corresponds to the static condition, therefore negative pressures are reported.

The pressure cell located in the West (PCW) had a maximum increase of 11.8 kPa, while the pressure cell in the East (PCE) presented a maximum decrease of 3.3 kPa. The measurements show consistency with the physical phenomenon, since the application of the horizontal load in the upper part of the column imposes a turning moment that favors a rotation of the footing, causing the pressure on the west side to increase, and in the east, to decrease.

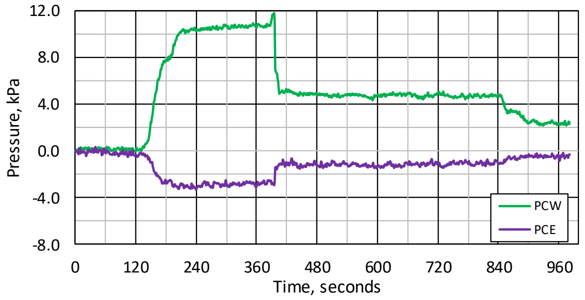


Figure 15. Vertical pressures under the footing.

With the maximum applied load, its inclination and the height of the application point, we find that the moment acting on the footing is 5330 kN-m. According to the measurements, the resistant moment provided by the piers is 4468 kN-m; that is, the piers resist 84% of the applied moment. The remaining 16% comes from the pressures at the base of the footing, as well from the frictional resistance forces that develop on the front and side faces of it. The pressure cells located at the base of the footing recorded very low variations in that vertical pressure. However, it should be noted that such measurements must be taken with some reserve given that the cells were added after the footing-column was placed, although it was intended that the sensitive element of these pressure cells made contact with the footing by means of a mortar with a certain expansion additive. Then this brings some uncertainty in the measured pressures below the footing. Considering the forces measured in the piers and the estimates of the reactions of the footing walls, it is estimated that the response of the base is marginal; it does not contribute more than 10% of the applied moment. The rigidity of the piers in their axial direction then prevails, and that this comes mainly from the skin resistance. It is relevant to mention that very low measured contribution comes from the tip.

The load transfer mechanism in drilled piers under turning moments to the foundation, is then the same as the case in which the application of load is directly applied in axial direction (Mendoza et al., 2016). In both cases the response of the piers comes from the skin friction, and the contribution of the tip is practically nil, at least for workload levels.

Regarding what was measured with the pressure cell at the tip (CPP), it is found that the maximum pressure variation was 2.5 kPa, which leads us to conclude that the turning moment caused by the load in the column, was supported by friction in the lateral skin of the piles, and it was almost imperceptible by the tip of the piers.

6 CONCLUSIONS

The geotechnical instrumentation arranged in a support of the Bicentennial Viaduct proved to be a very convenient means of understanding the load transfer mechanisms between the superstructure and its foundation. The foundation system consists of a precast footing to its column, and four drilled piers, which were post-tensioned to the footing, in order to ensure their union and joint work. The tests presented in this paper correspond to those in which lateral forces were imposed on the upper part of the column of this viaduct, thus inducing significant overturning moments to the foundation. Based on the measurements made, the following conclusions are presented:

Measurements of microdeformations along the piers, allowed to establish that the resistant forces reached in its connection with the footing generated a resistant torque that represents 84% of the applied turning moment.

The vertical pressure registered by the pressure cells under the footing was consistent with the effect of the applied moment; it increased towards the west side, while it decreased towards the east side. However, their values, which are very low, seem to show the effect that these cells were added after the footing-column was placed. It is estimated that the footing base provided a response that did not exceed 10% of the applied moment. The small remaining fraction of the resistant moment was contributed by the frictional forces generated in the side walls of the footing.

Coincident measurements of the forces transmitted by the piers at depth by means of the strain gauges embedded in the concrete, on the one hand, and the pressure cell arranged at the tip of one of the piers, leads us to conclude that, in this case, the contribution of the tip was barely perceptible. Indeed, in a section one meter from the tip, a force was measured representing 6% of the maximum recorded; and right at the tip, the maximum measured pressure was barely 2 kPa.

With the above, the measurements in some other axial static load tests were ratified, where the applied loads are fundamentally supported by the lateral skin of the piers. In the case discussed in this paper, the base of the piers does not contribute to resisting the axial loads generated by the overturning moments.

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