Geotechnical instrumentation in large-diameter access shafts

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ABSTRACT
The Eastern Sewage Tunnel (TEO) will be part of Mexico City´s drainage system. It will be more than 62 km long at depths varying from 32 to 148 m. It will traverse different kinds of soils most of which are the product of depositions in a lacustrine environment. The tunnel required the construction of access shafts having diameters ranging from 12 to 16 m. This paper describes the geotechnical instrumentation installed in the deepest shaft (L-20); it also describes the results of the initial measurements.

RESUMEN
El Túnel Emisor Oriente (TEO) será parte del sistema de drenaje de la Ciudad de México. Tendrá más de 62 km de longitud y profundidades de 32 a 148 m. En su desarrollo atravesará diversos tipos de suelo, la mayoría productos de depósitos en ambiente lacustre. El Túnel requiere la construcción de lumbreras de acceso con diámetros de 12 a 16 m. Este artículo describe la instrumentación geotécnica instalada en la lumbrera más profunda (L-20), así como los resultados de las primeras mediciones.

1 INTRODUCTION

1.1 Background

The so-called Eastern Sewage Tunnel (Túnel Emisor Oriente or TEO) will have an approximate length of 62 km; for purposes of construction and subsequent operation it will be provided with 24 access shafts and an exit portal. The tunnel starts at the intersection of the avenues Gran Canal and Río de los Remedios; it runs almost parallel to the left bank of the Gran Canal following a north-west direction along an approximate distance of 10 km; then it shifts direction toward the north-west at the right bank of the Gran Canal, and it crosses the municipalities of Ecatepec, Coacalco and Tultepec in the Valley of Mexico, along a distance of 20 km.

From this site it diverges from the Gran Canal, crosses at the west of Laguna de Zumpango and traverses the municipalities of Teoloyucan and Huehuetoca with a length of 20 km and then it follows the right bank of Tajo de Nochistongo, crosses the municipality of Melchor Ocampo and arrives at the Exit Portal located at Ejido de Conejos in the neighboring state of Hidalgo. The access shafts will be spaced at average distances of 2.5 km between each other and they will reach depths ranging from 32 m to 149 m, and at the sites where they will be excavated different geotechnical characteristics are encountered. From Shaft L-00 to Shaft L-06, the design grading for the tunnel will be equal to 0.19%, whereas from Shaft L-06 to the Exit Portal it will change to 0.16%.

As part of this project reference will be made of Shaft L-20 that will become the deepest from them all. The geotechnical instrumentation works presented herein were requested by the company COTRISA, S.A. de C.V. and were carried out according to the guidelines and specifications established by CONAGUA in compliance with the instrumentation project.

1.2 Characteristics of Shaft L-20

Shaft L-20 is located in the municipality of Huehuetoca, Estado de México. It corresponds to stationing Km 49 + 607.654 of the tunnel alignment with its center lying at coordinates x = 476,861.310 and y = 2,196,795.830; it will have a maximum depth of 148.204 m with an internal free diameter of 22.06 m. The existing ground surface at the site is located at elevation 2269.385 msnm (meters above mean sea level) (Figure 1). This shaft is currently in the stage of excavation of the core and it was necessary to build a diaphragm wall to a depth of 120 m; segments of the wall are 1.20 m in width and 2.80 in length. After a depth of 120 m and until the maximum depth of 160 m is reached it will be necessary to continue the excavation using shotcrete, anchors and steel rings.

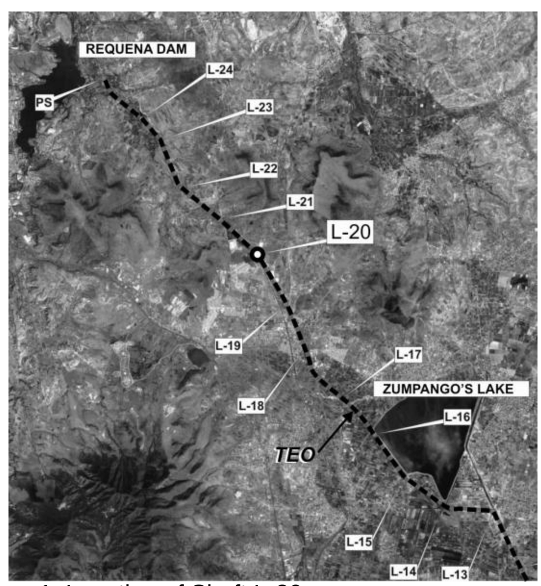

Figure 1. Location of Shaft L-20

2 GEOTECHNICAL CONDITIONS AT THE SITE

2.1 Available geotechnical information

For the geotechnical project of Shaft L-20, borings with continuous sampling were previously made to recover undisturbed specimens for subsequent laboratory testing. Figure 2 presents the geotechnical conditions encountered at the site; a summarized description of the main stratigraphic units is included as follows.

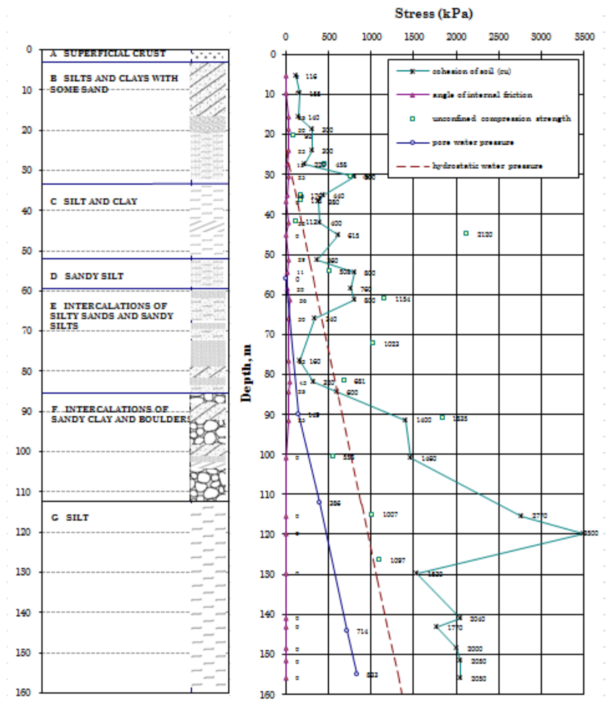

Figure 2. Geotechnical conditions at the site

(A) Superficial Crust. From 0 to 3.10 m in depth; it is constituted by highly plastic clay (CH) with stiff consistency and average Standard Penetration Test (SPT) blow count equal to 15 and an average natural water content of 31%.
(B) Silts and clays with some sand at depths varying from 3.10 to 33.50 m; actually sandy clays and silts (CH, CL, MH and ML), with stiff consistency in which the SPT blow count ranges from 60 to 90 and the natural water content has an average value of 30%.
(C) Silt and clay. From 33.50 to 52.00 m silt and highly plastic clay (MH and CH) with stiff consistency are interspersed, with NSPT of 30 blows on the average and average natural water content of 36%.
(D) Sandy silt. From 52.00 to 59.50 m in depth; silt with high to low plasticity (MH and ML), with variable amount of sand and very stiff consistency, SPT blow count of 90 on the average and average natural water content of 29%.
(E) Intercalations of silty sands and sandy silts, from 59.50 to 85.60 m in depth; alternated layers of very dense silty sands and sandy silts with very hard consistency (SM and ML) can be found where the average SPT blow count was 90 and the average natural water content, 31 %.
(F) Intercalations of sandy clay and boulders. From 85.60 to 112.50 m in depth; this unit is constituted by sandy clays with very stiff consistency (CL), with an important amount of boulders at different depths; in these soils the average SPT blow count amounts to 75 and the natural water content is equal to 40% on the average.
(G) Silt. From 112.50 m to the end of the explored depth (158.50 m), there exists a deposit of highly plastic silt (MH) with very stiff consistency in which the average value of the natural water content is equal to 45% and the SPT blow count varies from 60 to 90.

2.2 Existing piezometric conditions

To develop basic studies of the TEO project, the Comisión Federal de Electricidad (CFE, 2008) was engaged to install at the site of Shaft L-20 a piezometric station labeled PZA-L20 that is constituted by five Casagrande-type open piezometers and a telltale casing, TO-L20. This piezometric station was installed on October 16, 2008 and the last reading was taken on June of 2010, and it is reported in Table 1. Mention should be made that the five piezometers constituting the piezometric station were placed in a single 8.5”-diameter borehole; in a later section a discussion will be presented of some disadvantages related to the installation of this type of stations.

Table 1. Piezometric conditions at Shaft L-20.

Instrument	Depth (m)	Pore pressure (kPa)
PZA-L20-A	155	837.31
PZA-L20-B	144	725.64
PZA-L20-C	111.7	443.49
PZA-L20-D	90	157.98
PZA-L20-E	56	Dry
TO-L20	25.55	Dry

3 GEOTECHNICAL INSTRUMENTATION PLACED AT SHAFT L-20

To monitor the behavior of the subsoil during construction of the access shaft it was necessary to install the following geotechnical instrumentation: a) four vertical inclinometers (IV) at a depth of 160 m and one piezometric station constituted by four vibrating wire piezometers (PCV) at different depths (Figure 3). The depth, type and characteristics of the instrumentation were determined by CONAGUA and the specifications established for its installation were complied with.

The objectives of the placement of the instrumentation were: monitoring of the stability of the excavation through the installation of inclinometers and, control of the hydraulic subsoil conditions during the excavation by means of the piezometric station. The description of the works is presented as follows.

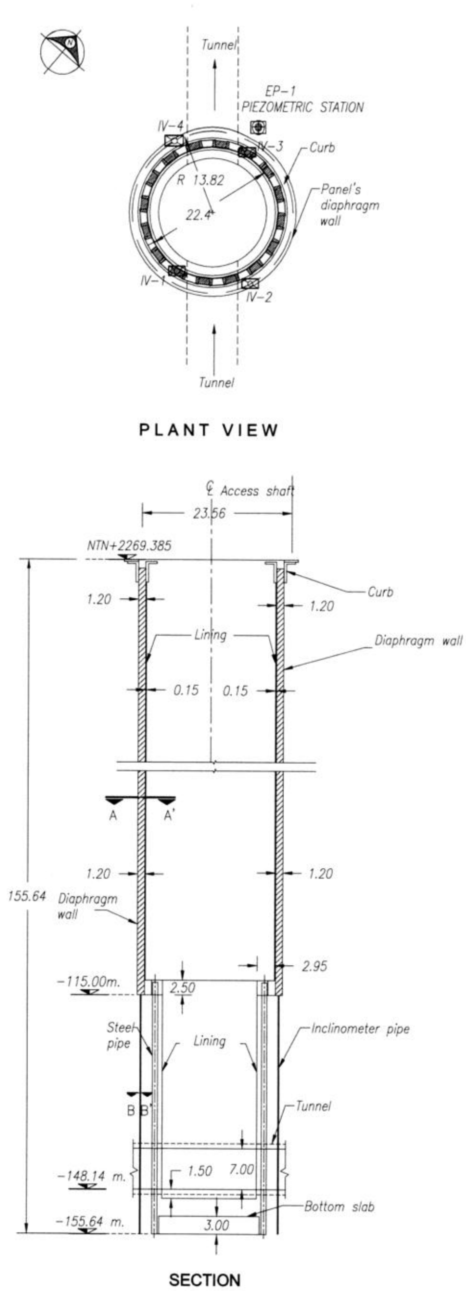

Figure 3. Location and depths of the instrumentation

3.1 Vertical inclinometers

Four inclinometers were installed to a depth of 160 m to monitor the radial movements with respect to the excavation and they were distributed as follows: two of them at the diaphragm wall and the other two in the adjacent natural ground.

3.1.1 At the diaphragm wall

Prior to concrete casting of the segments, the contractor inserted a 3"-diameter PVC casing through the steel reinforcing down to the end of the 120-m depth of the wall. Once the reinforcing cage was lowered and prior to concreting, the inside of the casing was filled with dumped fine sand so as to prevent its collapse. Upon completion of concrete pouring of the circular segments and after the concrete had reached the design strength (35,000 kPa) a boring was made with a tricone bit in three stages: first using a bit of 2 15/16 inches in diameter to remove the sand and part of the 3"-diameter casing and subsequently to drill again with a 4 ½ inches bit to reach a depth of 160 m.

After reaching this depth the inclinometer casing was introduced (ABS-type casing with 73 mm of external diameter and 3.05-m long in each section (Figure 4) placing a steel tip and a bottom plug. For the maneuver of casing sections placement it was necessary to pour plain concrete in the first 1-m long section to act as ballast when lowering the subsequent sections. To facilitate the maneuver of insertion, the inside of the casings was filled with water to lower them under their dead weight. For the purpose of assuring the contact between the borehole and the inclinometer casing a grout mix was injected around the annular space using a proportioning of 15%, 5% and 80% (cement, bentonite, water).

Figure 4. Installation of inclinometer casing.

3.1.2 In the subsoil

Using a similar procedure, to install the inclinometers in the natural ground a borehole with a diameter of 215/16 inches was first drilled and subsequently re-drilled to a diameter of 4½ inches to a depth of 160 m. The installation maneuvers were the same used for the inclinometers embedded in the diaphragm wall. To guarantee the contact between the diaphragm wall and the casing as well as between the soil and the casing, a grout mix was injected around the annular space using a proportioning of 15%, 5% and 80% (cement, bentonite, water) (Figure 5).

Mention should be made that to lower the inclinometer casing to a depth of 160 m it is helpful to fill it inside with clean water so that it moves down by gravity and that a steel cable attached to each section is used to handle it properly (Figure 6). When the two inclinometers for the diaphragm wall started to be installed the excavation of the core of the shaft had reached a depth of 16 m and it was therefore necessary to support the casing with a steel structure anchored to the internal wall of the shaft to serve as working platform for the drilling crew (Figure 7).

3.1.3 Comments

The selection of the proper diameter to carry out drilling of the inclinometer borehole is an important matter. A boring with large diameter simplifies the installation but at a higher cost, whereas a boring with a smaller diameter is cheaper to execute but if not sufficient experience is available to handle the casings a potential risk of failure can be expected in the installation. In addition to this, in past time aluminum casings with a diameter of 3” were used with couplings of the same material that were joined with POP rivets or with metal strips to prevent sliding down of the coupling or otherwise when the coupling was intended to slide it was attached to the individual casing sections with some type of adhesive tape (Figure 8). The maneuvers for its installation were of complex nature particularly when it was required for the couplings to slide. Consequently, the borehole was drilled with a diameter of as much a six inches.

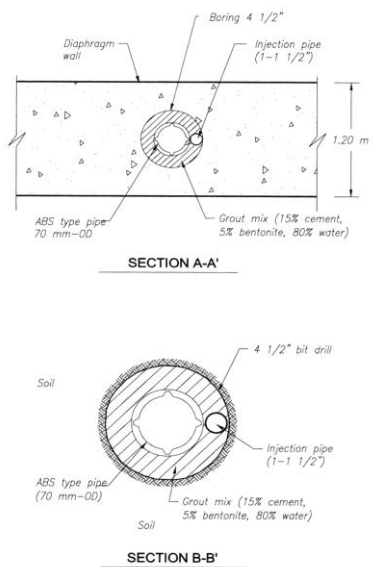

Figure 5. Injection to fix the inclinometer casing

Figure 6. Maneuvers required to install the inclinometer casing

Figure 7. Working platform to install the inclinometers through the diaphragm wall

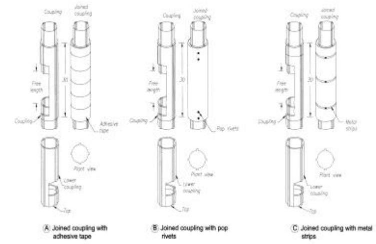

Figure 8. Joints between aluminum casing sections

In the last few years the casings for inclinometers are manufactured with different materials such as aluminum, fiberglass, polyvinyl chloride (PVC) and acrylonitrile butadiene styrene (ABS), being the latter the standard material adopted currently by manufacturers of casings having external diameters ranging from 48 to 85 mm. The unions between individual sections has been also resolved adequately by means of waterproof joints for quick connection and with sliding couplings; these two combinations have expedited to a large degree the installation of inclinometers (Figure 9) and, as a result, drilling with diameters larger than 4½” should be avoided and the practice of still using aluminum casings and couplings joined with straps or rivets should be abandoned, particularly in regions subjected to regional subsidence that induces buckling of the casings with consequent erroneous readings (Santoyo et al., 1981).

Figure 9. Plastic casings and couplings for inclinometers

3.2 Piezometric station (EP)

3.2.1 Installation

To determine the distribution and variation with time of the pore water pressure in the subsoil a piezometric station was installed being constituted by four electronic vibrating chord piezometers (PCV) placed at different depths inside a single borehole. To carry out the installation it was necessary to drill in four different bit diameters: 215/16, 4½, 57/8 and 8½ inches. After the maximum, depth of 160.5 m had been reached the boring was flushed several times with clean water until the water emerged clear and almost sediment free. Prior to the start of the installation initial readings were taken on the open-type piezometers. The four piezometers were fastened to a column of galvanized steel casing with a diameter of ¾” and after being fully attached the piezometers were lowered and at the same time saturated with clean water to have an assurance of their proper operation, introducing them carefully until reaching the respective depths indicated by the project (Figures 10 and 11).

Figure 10. Saturation of piezometers prior to their installation

Figure 11. Installation of vibrating wire piezometers

After the piezometers had reached their intended depth, sand filters, seals with pellets of bentonite and grout mixes (cement-bentonite-water) were placed from the bottom up with the dosage for the mix being 15%, 5% and 80%, respectively. The thickness of the filter was of 2 m, that of the bentonite pellets of 1 m and at the grout mix it varied from 5.5 to 13 m (Figure 12).

During all the time spent in the installation of the piezometers until its completion readings were taken of the pore water pressure until reaching the stabilization pressure of the instrument; it became particularly important to place the last 101 m of the bentonite seal in increments of 20 m each to prevent the generation of any excess pressure in the piezometers and not to exceed their range of operation.

3.2.2 Comments

In our country there prevails the practice of constructing piezometric stations (mostly for open-type or electronic piezometers) using two different approaches: the placement of piezometers in a single borehole or the use of independent borings to install each piezometer. As it was mentioned in paragraph 2.2, at the site of Shaft L-20 a piezometric station was installed in October of 2008 (CFE, 2008) with five piezometers inserted in a single 8.5" diameter borehole and placing a series of filters of different permeability and different thickness ranging from 25 cm to 4.0 m, as well as seals with bentonite pellets with a thickness of 20 cm and seals with a grout mix of water and cement in average thicknesses of 80 cm, to finally separate each piezometer with a fill material constituted by cement-bentonite-water (Figure 13). Placement of this complex sequence of materials that constitute the filtering chamber of the piezometer as well as of the isolating seals is actually very involved for various reasons: the diameter of the borehole, the number of piezometers to be installed and the final depth of each of the devices. All of these materials have to be installed by a field crew with one supervisor, oftentimes with insufficient capability and experience, with the enormous risk of leaving defective seals that propitiate interconnection among the piezometers, principally because of the close distance existing between the casings that constitute the piezometers themselves.

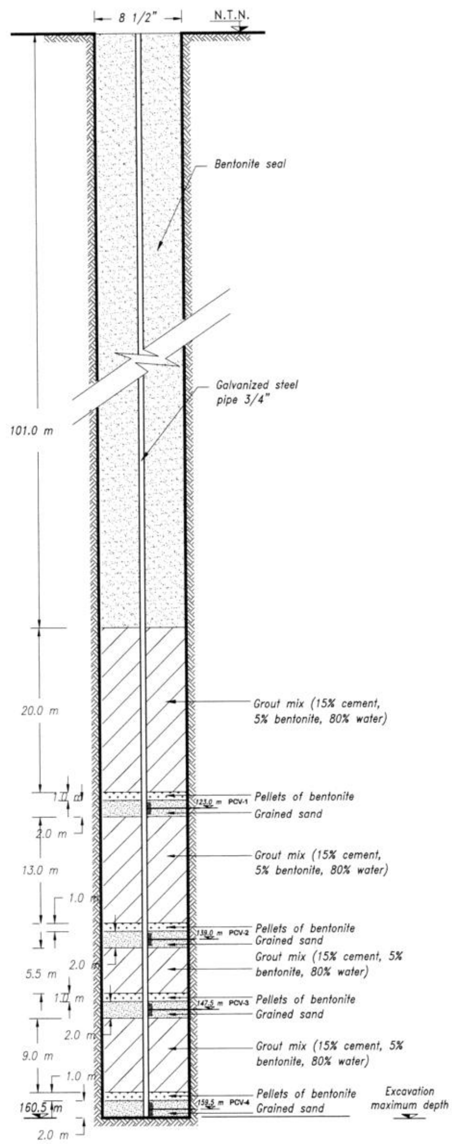

Figure 12. Station with vibrating wire piezometers installed at the job site

To prevent this type of errors the best option is to drill independent borings in diameters not to exceed 4"; this would facilitate enormously the installation and the risks of interconnection would be minimal.

If attention is paid to Figure 12 corresponding to the recent installation of the four vibrating chord piezometers (PCV) in a single borehole having a diameter of 8½ inches, although the number of filters and seals is smaller, the risk of committing mistakes during the installation is not actually reduced. Placement of filters and seals at great depths becomes a difficult task and there exists the possibility of exceeding the limiting pressure of the piezometers during injection of the cement-bentonite-water grout mix for the purpose of isolating the filtering chambers of the piezometers and to place the last section until reaching up the natural ground surface.

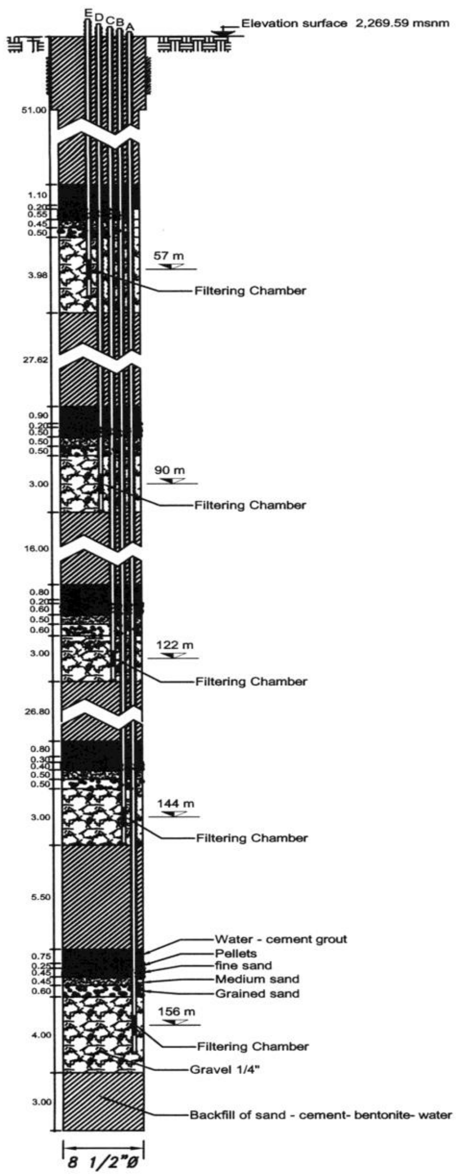

Figure 13. Station with open-type piezometers existing at the site (CFE, 2008)

4 RESULTS

4.1 Inclinometers

For the interpretation of the results obtained from measurements made with the inclinometer the following convention of signs was adopted (Figure 14): direction A+ toward the excavation and direction A- towards outside the excavation. The configuration of the horizontal displacements in terms of depth for directions A+ and A- are presented graphically in Figures 15, 16 and 17, whereas the graphs plotted in Figures 18, 19 and 20 depict the horizontal displacements in the full depth for the perpendicular directions B+ and B-.

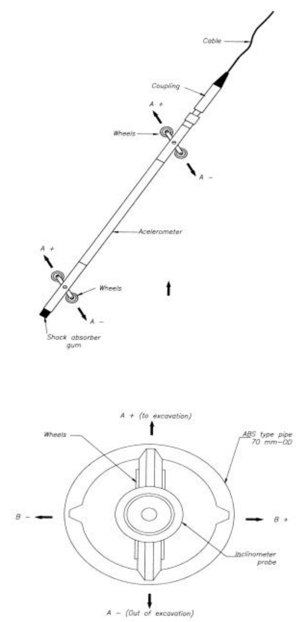

Figure 14. Sign convention for inclinometer readings

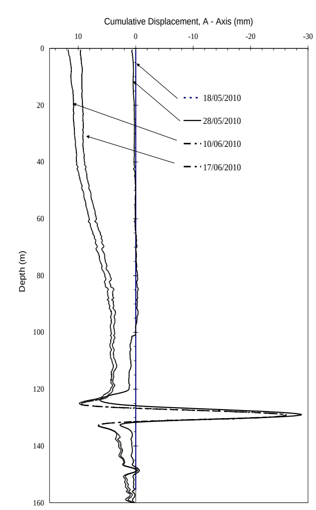

Figure 15. Horizontal displacements (A+, A-) at inclinometer IV-2 in the ground

It can be observed that in the case of inclinometer IV-2 installed in the ground maximum horizontal displacements of up to 10 mm are presented at the surface oriented toward the excavation with a trend to decrease down to a depth of 123 m where a change of direction is observed with values of 29 mm toward the outside of the excavation. At inclinometer IV-4 also placed in the ground maximum horizontal displacements are also observed toward outside the excavation with a value of 5 mm down to a depth of 71.5 m, with a trend to decrease until the maximum depth is reached.

For the case of inclinometer IV-3 at the diaphragm wall, minimum displacements of 3 mm are observed towards outside the excavation that probably correspond to the settling down of the inclinometer casing inside the borehole and to the accuracy of the measuring instrument; after a larger number of reading is available it will be possible to try to define a definite trend of the movement, associated if applicable to the excavation procedure.

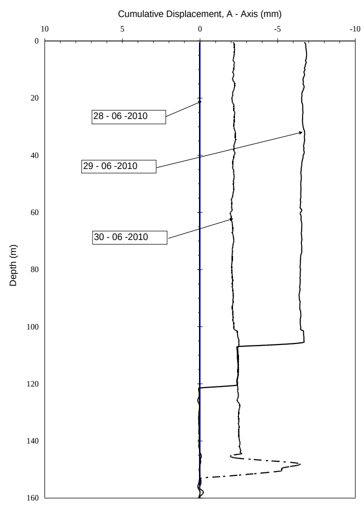

Figure 16. Horizontal displacements (A+, A-) at inclinometer IV-3 in the diaphragm wall

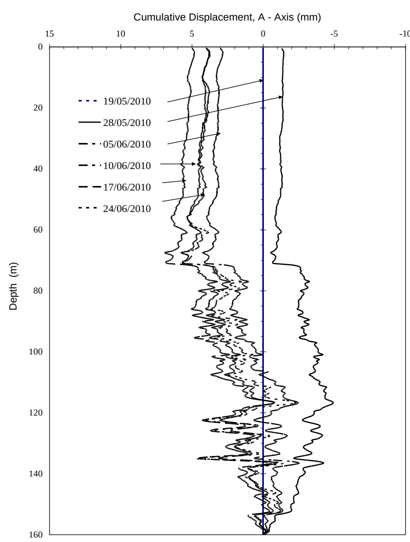

Figure 17. Horizontal displacements (A+, A-) at inclinometer IV-4 in the ground

Graphs corresponding to inclinometer IV-1 installed at the diaphragm wall are not reported because of problems developed during its installation that made it necessary to install it again, and values of the measurements are not still available.

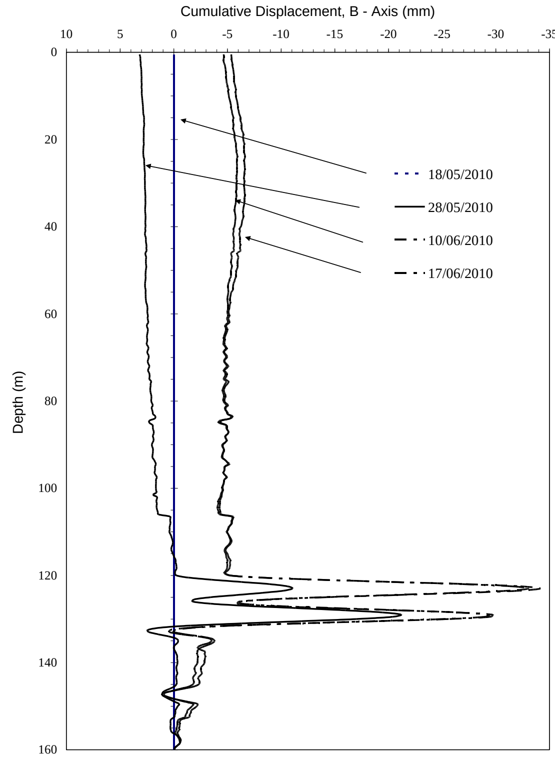

Figure 18. Horizontal displacements (B+, B-) at inclinometer IV-2 in the ground

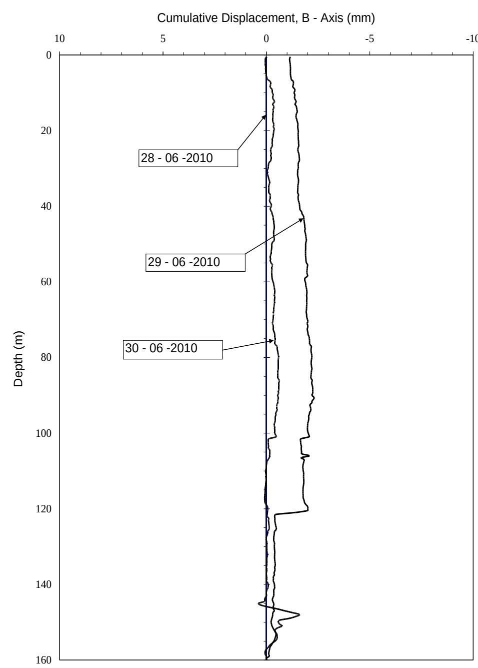

Figure 19. Horizontal displacements (B+, B-) at inclinometer IV-3 in the diaphragm wall

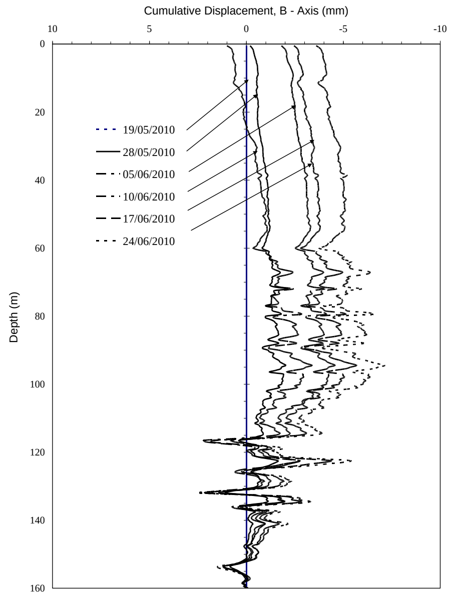

Figure 20. Horizontal displacements (B+, B-) at inclinometer IV-4 in the ground

4.2 Piezometric station EP-1

Since the installation of the piezometric station readings were taken at the four vibrating chord piezometers until the stabilization pressure that corresponds to the hydraulic pressure existing at the site was reached. Figures 21 and 22 show the variation of pressure and temperature as a function of time; the values of the pore water pressure as of July 15, 2010 are presented in Table 2. Both graphical representations are compared in Figure 23.

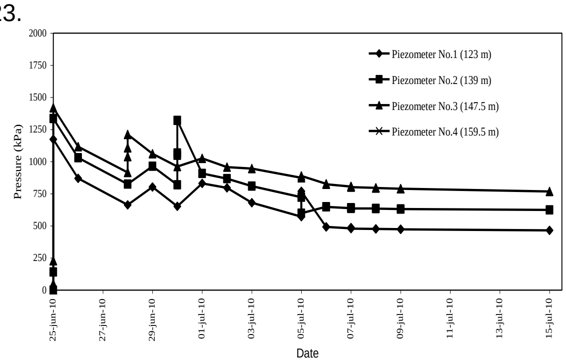

Figure 21. Pore water pressure measured at the station with vibrating wire piezometers

Table 2. Pore water pressures measured at the station with vibrating wire piezometers on July 15, 2010.

Instrument	Depth(m) Pressure (kPa)	Pore pressure (kPa)
PCV-1	123	47.9
PCV-2	139	64
PCV-3	147.5	78.6
PCV-4	159.5	109

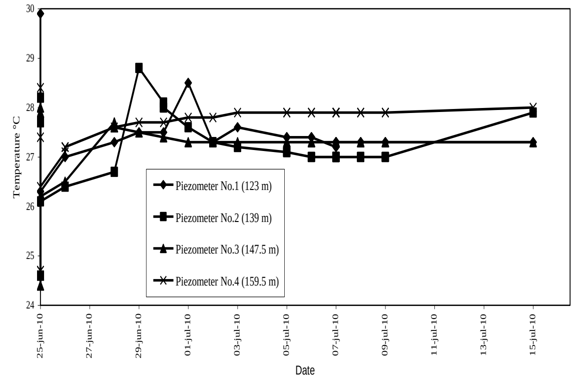

Figure 22. Variation of the temperature at the station with vibrating wire piezometers

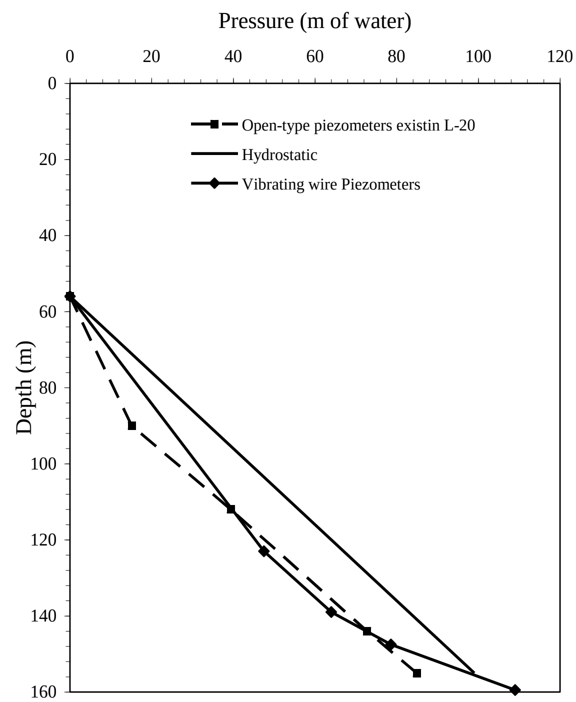

Figure 23. Comparison of the pore water pressure values measured at both piezometric stations

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