Pumping effects on deep excavation in the Valley of Mexico clay deposits

Les effets du pompage sur une excavation profonde dans la vallée des dépôts d'argile de Mexico

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SYNOPSIS: This paper presents an account of an experimental stretch constructed on Line 6 of Mexico City's subway (Metro), in which a comparative study was made of the behavior monitored at an excavation for a box-type tunnel, shored with cast-in-place concrete walls (Milan diaphragms); part of the constructions was performed with gravity pumping prior and during the excavation and the rest by resorting to intermittent local dewatering aimed at evacuating groundwater seeping towards the bottom of the excavation and concentrating it in small temporary sumps. By means of a system of piezometers installed at different depths, it was possible to monitor the groundwater hydrostatic level during the excavation stages. From the results thus obtained, together with the application of certain theoretical approaches, an interpretation of the hydrodynamic phenomenon was advanced and a theoretical method was proposed to calculate the pore-water pressures induced by unloading of the ground during the excavation process. The study showed that in cases similar to the one described, the advantages likely to depelop from the effects of the prior pumping (predrainage) do not justify the excessive cost of this operation.

INTRODUCTION

During the execution of relatively deep excavations in the soft clay deposits of the Valley of Mexico, it is quite common to resort to deep groundwater pumping prior and during the construction operation with the apparent purpose of improving the general equilibrium conditions of the soil mass and to eventually benefit from the increased efficiency of the heavy excavation equipment utilized.

There exist, however, some uncertainties about the mechanism affecting the variations of the pore-water pressures, not only when this procedure is complied with but also when such type of pumping is omitted. At the same time, discrepancies exist on whether or not the potential benefits derived from pumping justify its cost.

In order to clarify at least part of the uncertainties involved, an experimental stretch was constructed along part of Mexico City's subway Line 6, at present being built, conceived as an underground box-type tunnel in which the excavation walls are braced againts cast-in place diaphragm walls. In one part of said stretch, routine construction procedures normally applied in this type of projects were adopted among which mention should be made of the installation of a series of well points along the excavation longitudinal axis which start pumping out the groundwater one week in advance of the excavation start-up and then keep on operating continuously until the concrete floor slab is poured.

For the sake of comparison, in the remaining part of the experimental stretch such type of pumping procedure was omitted during the entire time the excavation lasted, without changing at all the construction procedures followed in the first case; the objective of the paper presented herein is to demonstrate the results thus obtained in what refers to the behavior monitored in both parts of the stretch, as well as to provide an interpretation of the criteria developed so far to explain the variations of the pore-water pressure induced during the constructions process, as well as the practical type of implications which might derive from the utilization or omission of groundwater pumping.

DESCRIPTION OF THE EXPERIMENTAL STRETCH

Location and stratigraphy

The experimental stretch under study corresponds to part of Line 6 of the subway system (Fig. 1) and it is located from a geotechnical point of view in the so-called Transition Zone, in between the zone of lacustrine clay deposits (Lake Zone) and the firm deposits of the Hill Zone in the northern part of Mexico City. The soil profile determined at the project site, as well as its index properties and the variation with depth of the shearing strength determined from electric static cone penetration tests and with Torvane are presented in Fig. 2; generally speaking, it can be observed that the upper 6 m in depth correspond to a superficial crust formed by recent deposits characterized by silt and clay strata which have been subjected to the effects of drying and with a cone penetration resistance varying from 2.5 to 15 kg/cm^2 (0.2 to 1.47 MPa) and a natural water content in the range of 50 to 100 per cent.

Underlying this uppermost deposit and down to the maximum explored depth of 20 m, there exists a succession of clay layers interbedded with strata of sandy silt and silty sand; the natural water content of the clay deposits fluctuates from 200 to 400% whereas for the silt layers there is a mean value of 100%. The cone penetration resistance ranges from 2.5 to 10 kg/cm^2 (0.24 to 0.98 MPa) except at the

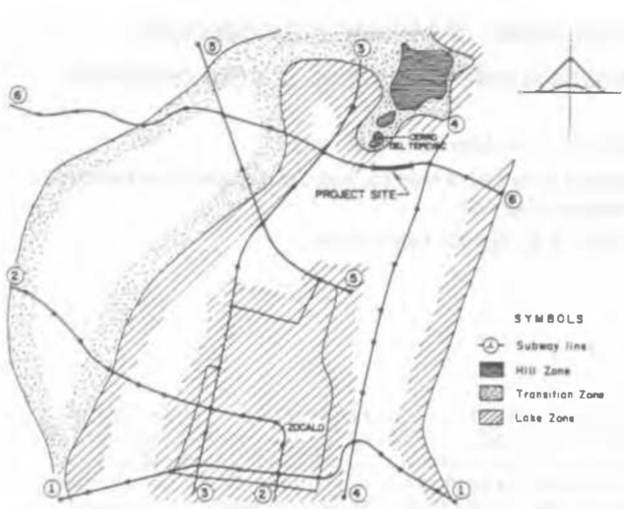

Fig. 1 Stratigraphical zoning of Mexico City

various thin intebeddings of very stiff soils, were this resistance exceeds widely the above values; from measurements taken with the Torvane the undrained shear strength was found to vary from 3 to 6 t/m^2 (29 to 59 kPa) with a weighted average of 4 t/m^2 (39 kPa). As it will be shown further on, the zone has been subjected during a long time to lowering of the groundwater table induced by deep pumping at a regional level, a fact that has affected the consolidation pressures of the clay deposits located at depths in excess of 10 meters.

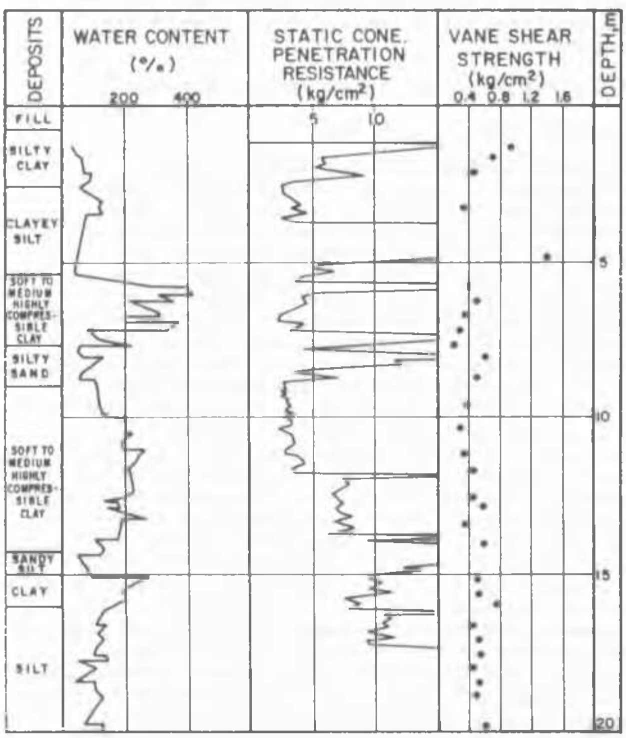

Fig. 2 Profile and soil properties

Geometry

In Fig. 3 it is shown the typical cross-section of the box-type underground structure, with its corresponding dimensions. The length of the experimental stretch was divided into two sections, identified as A and B and they are also presented, each section being about 60-m long. At section A the excavation proceeded once the well points installed along the longitudinal axis at every 10 m, became operative approximately one week in advance; it should be mentioned that the tip of the well points was placed 2 m under the bottom of the excavations.

On the other hand, at section B all the well points were omitted and only local dewatering from sumps was required during excavation.

Instrumentation

With the aim of monitoring the development of pore-water pressures in the ground, not only under the bottom of the excavation but also in its surroundings, a system of piezometers was installed down to depths ranging from 10 to 19 m as depicted in Fig. 3; in addition, observation wells of the table were advanced on the outside of the diaphragm walls. Piezometers utilized were of the pneumatic type, because of the low permeability of the soil deposits. Piezometric records were made at time intervals varying from two to three days prior to the start of the pumping operation and as the excavations progressed and they lasted until aproximately one month after it was completed.

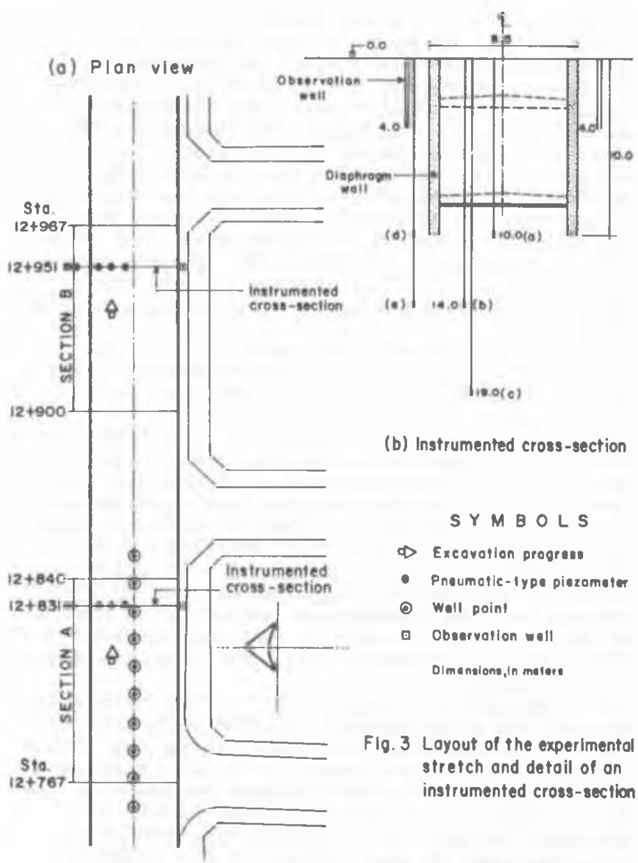

Fig. 3 Layout of the experimental stretch and detail of an instrumented cross-section

Construction procedure

For the sake of simplifying the interpretation of the measurement records performed to be discussed in the subsequent chapter, it has been deemed pertinent to present in what follows, a brief description of the construction procedure which has been established for conducting the excavation for a typical box-tipe tunnel for the Metro lines. To do this, reference is made to the diagrams shown in the upper part of Figs. 4 and 5, corresponding to sections A and B, respectively, which are self-descriptive; the sequence of the different stages covered by the construction procedure can be summarized as follows:

a) Construction of mouth walls to serve the purpose of guiding the excavation of the trenches that will contain the diaphragm walls. Bentonite slurry is used to stabilize the trenches.

b) Starting of the pumping operation along of 20 m preceding the zone to be excavated, six days prior to the beginning of the excavation.

c) The excavation proceeds in lengths of 7 m with the side walls sloping 0.5 (horizontal) to 1.0 (vertical). During excavation transversal struts are placed at three elevations, measured with respect to the upper level of the roof slab. The second tier of struts is removed 24 hours after the third one is installed.

d) Once the bottom of the excavation is reached, work proceeds on the placement of a lean concrete layer 10-cm thick in no more than four hours. Pumping is disconnected at this stage.

e) Within the next two hours the floor slab is poured in no more than 36 hours and the third tier of struts is removed 24 hours after casting.

f) Installation of the precast elements for the roof slab and construction of the compression pad of such slab. When the pad reaches its ultimate strength the first tier of struts can be removed.

g) When the roof slab attains the specified strength, the fill necessary to reach the ground surface can be placed.

h) The next excavation stage may proceed once the pouring of the floor slab corresponding to the previous stage is started.

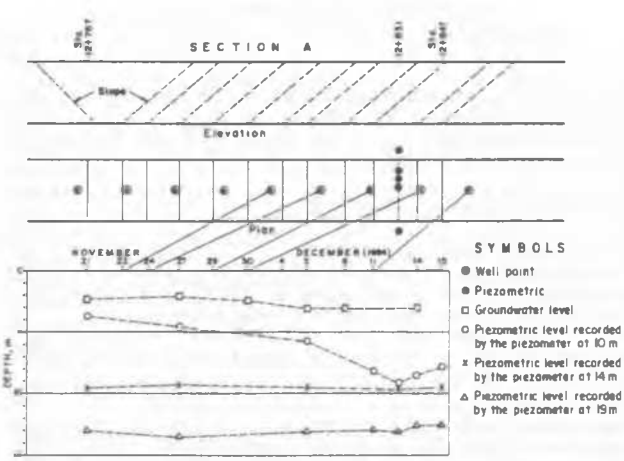

Fig 4 Section A (with pumping)

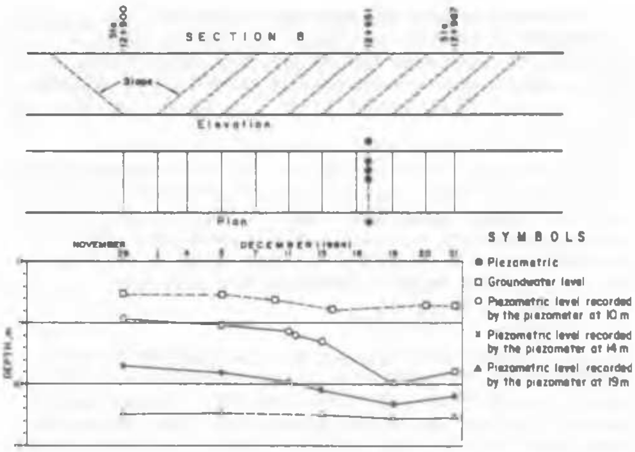

Fig. 5 Section B (without pumping)

MEASUREMENT RECORDS

The results of the measurements made at the piezometers installed at sections A and B are presented in Figs. 4 and 5, corresponding to the excavation stages with and without pumping, respectively.

The layout of the instrumented cross-section A is depicted in Fig. 4 as well as the plan view and elevation of the successive excavation stages and position of the forward slopes and the conclusion dates of each stretch. The location of the well points installed is also shown and a line joining them with the time scale indicates the date when pumping started at each of the wells.

If the initial position of the water levels recorded at the piezometers installed under the excavation limits at depths of 10, 14 and 19 m is analyzed, a groundwater lowering effect can be inferred which situates such levels at depths of 3.8, 9.5 and 13 m, respectively. As time proceeds and upon commissioning of the successive well points and while the excavation progresses, the piezometers register a drawdown of such levels, being most conspicuous at the piezometer tip 10-m deep and hardly perceptible at the two others; the greatest drawdown is recorded during the day when the excavation reaches the instrumented cross-section. At that moment, the groundwater elevations correspond to 9.2, 9.8 and 13.2 m, respectively; a recovery of such levels can be then observed being more relevant at the shallowest piezometer. During the same period the phreatic level on the outside of the box section oscillated from 2.5 to 3 meters.

The sequence of events occurred at section B has been plotted in Fig. 5 in which no deep pumping exists. It can be observed that initially the piezometers mounted at depths of 10, 14 and 19 m like is section A, show drawdonws that affect the position of the groundwater level found at depths of 4.5, 8.5 and 12.5 m, respectively; as it can be appreciated, these values are quite similar to those corresponding to section A. As the excavation process advances, such levels are lowered down to depths of 10.0, 11.5 and 13.0 m respectively when the excavation crosses the instrumented cross-section. In correspondence with section

A, the shallowest piezometer reflects the strongest lowering of the water table and a subsequent recovery is observed. The phreatic level outside the excavation boundaries during this period fluctuated from 2.5 to 3.5 m in depth.

When both cases are compared, it can be concluded that the evolution of the piezometric drawdowns as the excavation progresses shows a similar trend regardless of whether or not gravity pumping exists; additionally, the groundwater table drawdown was more conspicuous at the section where pumping was omitted.

THEORETICAL APPROACHES

Let us assume a rectangular loading of infinite length, i.e. a uniformly distributed load of finite width and infinite length, acting upon the surface of an elastic medium. The solution developed by Terzaghi and Carothers (1) for the principal stresses is as follows (Fig. 6):

$$\sigma_1 = \frac{p}{\pi}(\alpha + \sin\alpha)\ ;\ \sigma_3 = \frac{p}{\pi}(\alpha - \sin\alpha) \quad (1)$$

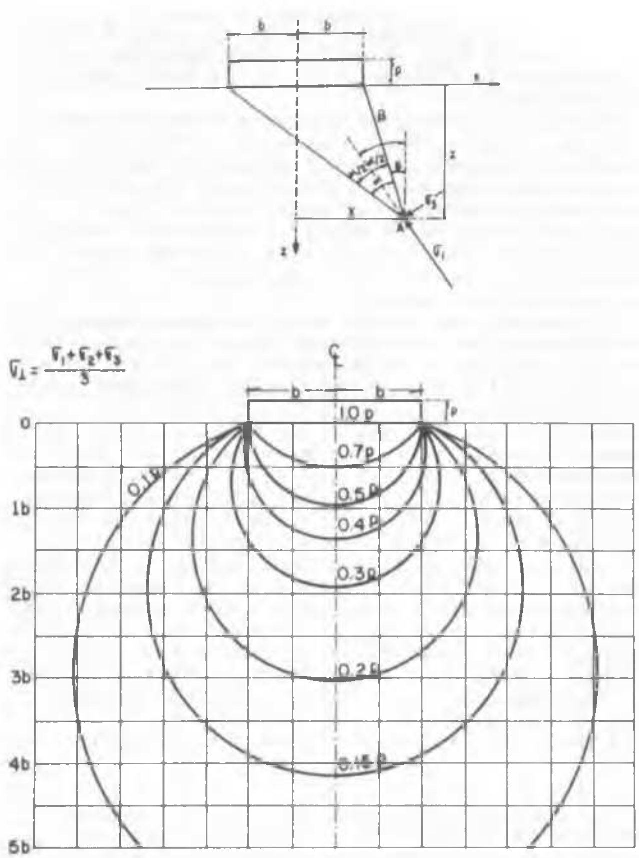

Fig. 6 Isotropic stresses under a rectangular load of infinite length

Assuming that the soil is constituted by saturated clay, the value of Poisson's ratio is 0.5 and for the intermediate principal stress σ_2 the following equation can be written:

$$\sigma_2 = \frac{\sigma_1 + \sigma_3}{2} \quad (2)$$

In this expression the isotropic stress σ_i is equal to:

$$\sigma_i = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3} = \frac{\sigma_1 + \sigma_3}{2} = \frac{\alpha}{\pi} p \quad (3)$$

Equation 3 can be applicable to the case of a rectangular unloading of infinite length, i.e., an excavation of finite depth and width and infinite length. In this latter case, the unloading boundary is the bottom of the excavation and the magnitude of the unloading is equal to the total pressure at the elevation corresponding to the excavation bottom.

Pore-water pressure induced by either loading or unloading conditions as previously establised consists of three components for the most general case of preconsolidated clays (2):

$$\Delta u = \Delta\sigma_i + [\alpha\,\sigma_c \frac{\sigma_c}{\sigma_e} - \alpha(\sigma_e - \sigma_c)]\ y \quad (4)$$

where σ_c and σ_e are, respectively, the isotropic consolidation and equivalent consolidation pressures, α is a pore pressure coefficient and "y" is the sensitivity function. In eq. 4 the sensitivity function "y" is such that far from the failure it is small and close to it increase up to a value equal to unity during the failure state. The meaning of this is that at a project where the safety factor (SF) is higher than 3, it can be assumed that the pore pressure is defined only by the isotropic component:

$$\Delta u \doteq \Delta\sigma_i \quad (5)$$

Therefore, for the cases of rectangular loading or unloading conditions with infinite length, if SF > 3, from eqs. 3 and 5 it can be written:

$$\Delta u = \frac{\alpha}{\pi} p \quad (6)$$

which if applied to the case of an excavation may provide the magnitude of the pore pressure decrements in the satured clays.

Mention should be made of the fact that the safety factor for this does not refer to the general failure of the project in itself, but rather to the specific safety factor of the points under consideration, that is to say, to the ratio between the shearing strength and the shear stress induced by the structure on a particular point.

The isobars or contour lines of equal decrease of pore pressure correspond to arcs of a circle with a chord equal to the width of the bottom of excavation (Fig. 6).

INTERPRETATION OF RESULTS

Table 1 presents a summary of the results corresponding to the piezometric drawdowns recorded at the experimental stretch as well as the values from the theory represented by eq. 6, for the case of an 8-m deep excavation, the unloading of which is equal to p = 10.4 t/m^2 (102 kPa) at the depths and location of the different piezometers installed. After comparing both sets of values it can be derived that save for the deepest piezometers, the agreement between the theoretical and the measured values, although roughly approximate, can be considered rather good. At the deepest piezometers the agreement between both sets of results is far less than satisfactory, a fact which still remains to be clarified; it can be attributed to a certain time lag in the response of the piezometers when dealing upon with rather low pressures, and perhaps to nonhomogeneous conditions reflected by the presence of pervious lenses contribute to the expeditious dissipation of the pore-water pressure.

Table 1.- Summary of results

Point see Fig. 3	Location	Pressure decrement t/m^2 (kPa)		
		Theoretical	Measured	
			Section A see Fig 4	Section B see Fig 5
a	- 10 m, axis	7.0 (68.6)	5.4 (52.0)	5.5 (53.9)
b	- 14 m, axis	3.5 (34.3)	0.5 (4.9)	3.2 (31.4)
c	- 19 m axis	2.0 (19.6)	0.5 (4.9)	0.5 (4.9)
d	- 10 m, 6.5 m to the left	1.4 (13.7)	1.6 (15.7)	0.4 (3.9)
e	- 14 m, 6.5 m to the left	2.0 (19.6)	1.0 (9.8)	1.7 (16.7)

Mention should be made of the fact that in the comparative analysis described above no appreciable differences were observed in the piezometric variations of sections A and B; in other words, the effects of groundwater pumping in the well points did not become manifiest in the piezometric drawdowns.

CONCLUSIONS

From the above results the following conclusions can be derived:

1. From a hydrodynamic point of view, the experimental stretch carried out did not show appreciable differences in the evolution of the piezometric levels upon performing the excavation either with or without well points.
2. It has been considered that the theoretical approach proposed to calculate the pore-water pressure variations induced by the unloading conditions during excavation through saturated soils is reasonably supported and that its application led to the obtention of results in fair agreement with the direct measurements recorded.
3. Even though the purpose of this paper is not to discuss the needs and benefits of resorting to well points, it should be mentioned that when excavation similar to those reported herein are encountered, it is very important not to rely in misinterpretations of the results by attributing to the pumping operations some effects that are no related to them, because in cases like the one under study it became evident that the piezometric drawdowns occurring during the excavations were independent of the deep well point pumping (3 - and 4).

ACKNOWLEDGEMENTS

The authors wish to express their gratitude to the authorities of Comisión de Vialidad y Transporte Urbano (COVITUR) for their unlimited support received during the carrying out of this research. Thanks are also given to Rioboo, S.A. the company in charge of supervising the construction of Line 6 where the experimental stretch was built.

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