Stress and Strain besides a Circular Trench Wall

Poussées et Mouvements d'une Paroi Moulée Circulaire

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SYNOPSIS. To start the works on the new Sevilla (Spain) subway a circular shaft has been excavated. To support the shaft an slurry trench wall has been constructed. Two of the wall moduli have been instrumented. Field instrumentation in each modulus consists of one inclinometer, pressure cells to measure earth pressure and circunferential stresses in the wall. Besides the two instrumented moduli, eight piezometers were set around the wall. The measurements and its interpretation is divided in two phases. In the first part the behaviour of the soil and the wall were studied during the excavation, concrete pouring and concrete hardening of the slurry trench wall. In the second part the circunferential stress the earth pressure and the movements during the excavation of the shaft were recorded. The paper gives account of the instrumentation and the measurements obtained, and discusses its interpretation, from the initial soil condition to the end of the excavation.

INTRODUCTION

The design of the future subway network in the City of Seville (Spain) raised up some problems from the geotechnical print of view as well as construction ones. In this regard, the Ministry of Civil Works initiated the subway ("Metro"), project with the construction of a 25 m diameter shaft for future conections with subway stations. This shaft accomplishes a twofold purpose: feasibilities studies for the utilization and behavior of slurry trench walls excavated with bentonite, and analysis of geothecnical charac-teristics of soils affected by shaft excavation operations, from surface clown to the level of underground hanling.

Previous to shaft sinking, a circular cast-in situ wall, 80 cm thick, was excavated with a Kelly shovel (independent valves type). Bentonite slurry was used for trench supports. The cast-in situ wall was assembled in 24 panels, forming a 26.75 m diameter circular area. Those panels, 3.40 m long, reached a depth of 34 meters. During the excavation inside the closed circular zone, a reinforced concrete inner lining was constructed. Inner lining thickness varies with depth (from 40 to 80 cm).

Within two of the panels, several measuring devices were installed to determine stress and displacement patterns in the slurry trench wall. Inclinometers were also used in the ground nearby the instrument-controlled panels. In this study, an analysis of results obtained during the instrumentation program is presented. Conclusions are discussed as well.

SOIL DESCRIPTION

Before panel construction, several boreholes were drilled for exploration purposes. That information together with that one provided during shaft sinking made able to define the following geotechnical soil descriptions:

Level A - Sandy silt cuaternary formations with silty clay and fine sands partings. Thickness reaches 13 m. Upper layers of this formation show a relatively recent silty filling materials. SPT tests give values ranging between 27 and 43 b/ft.

Level B - This is a quite thick layer (13 m thickness) formed by cuaternary granels. In this level SPT test were carried out, all of them gave values of 60 blows in 10 cm penetration.

Level C - "Blue Marls" formation. This level is constitued by a bluish-gray plastic and fissured clay layer. Within the clay layer, various thin lenses of sandy silt and highly carbonated clay can be observed, reaching 20 cm thick.

Table 1 shows grouped characteristics of the most relevant geotechnical properties of each of the above mentioned ground levels; i.e. depth, granulometry, plasticity, shear strength, deformability, etc. During the one year observation period, ground water table varied between 3.50 and 5.50 m, depending on weather season, rain fall, etc. An average value of 4.50 m. deep can be taken as a fairly representative figure.

TABLE 1.- SOIL PROPERTIES

LEVEL	DESCRIPTION	DEPTH (m)	GRAIN SIZE n°4	GRAIN SIZE n°40	GRAIN SIZE n°200	ATTERBERG'S LIMITS W_L	ATTERBERG'S LIMITS W_p	WATER CONTENT (%)	BULK DENSITY t/m³	SHEAR STRENGTH c' (kg/cm²)	SHEAR STRENGTH φ'(°)	VERTICAL DEFORM MODULUS (kg/cm²)
A	SILT AND FINE SILTY SAND	0-15	[illegible]	80	34	23* to 43	4* to 21	15 to 30	1.85	0-4	30 to 32	190**
B	GRAVEL	15-25	58	18	8	-	-	7.7	2.10	-	-	260
C'	BLUE MARLS	>25	100	99	93	52	22	26	2.05	1-2	30	1500

* CLAYLY-SILT FRACTION
** VERTICAL PLATE TESTS

INSTRUMENTATION PROGRAM

As a starting point for the planned study, the following objectives were established: a) Behavior analysis of the slurry trench wall to be installed in the blue marls ., wherein the subway tunnel is proposed to be excavated. This analysis involves ground pressures variations on the wall as shaft sinking progresses and deformation pattern within the concrete. b) Deformation analysis during construction and shaft sinking of the ground nearby the cast-in-situ wall. This analysis is to quantify, somehow, the effects of construction, of the type here used, on surrounding buildings. c) Geotechnical properties studies of the affected grounds, mainly the blue marls , by means of "in-situ" tests, carefull sampling, etc. d) Observation of effects of the inside area excavation on the outer ground water table.

With the above mentioned objectives, it was decided to monitor two slurry trench wall panels (numbers 12 and 24) (fig. 1), installing in each of them the following described instruments:

1º) Seven hydraulic cells for earth pressure measurements (Gloetzl, 20 x 30 cm). These cells were installed by two different methods. In the first method the cell is attached to a hydraulic jack which is then clamped to the reinforcing steel cage. Once the screen has been placed, the jack is activated to confine the cell against the soil. In the second method, the cell is fitted in the inside of a metallic frame (60 x 90 cm) which is then jointed to the reinforced steel cage. Its task was to materialize around the cell some carvings, in such a way that stresses in the concrete at 6-8 cm from the soil, were practically the same as in the soil-wall interface.

2º) Five hydraulics cells to measure the average circunferential stress in the cast-in-situ wall. The cells are Gloetzl type (10 x 20 cm) firmly fastened to the structure and with no allowable pay.

3º) Seven KIOWA SR-10 electrical extensometers, Carlson type, to measure average circunferential strain in the concrete. Another specially designed extensometer separates and detects creep strains

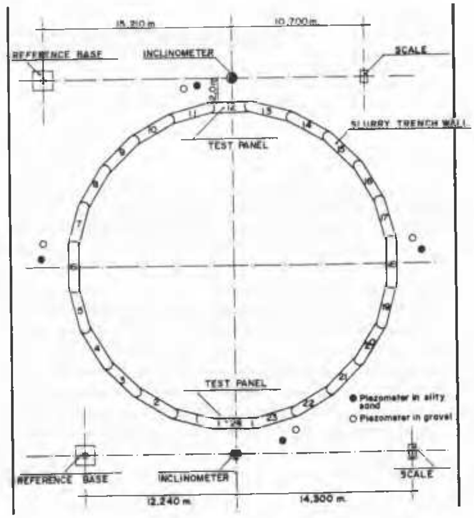

Fig. 1 Location plan with instrumentation positions

only. It permits to define circunferential stresses due to earth pressure, once concrete elastic modulus has been evaluated.

4º) One optical inclinometer was located at 2.5 m. from the outer face of the slurry trench wall, in order to measure horizontal soil movements at different levels. The inclinometer was 11 m long.

5º) Eight slotted-type piezometers (fig. 1). Four of them were placed with slots facing level A silty sands and the remaining ones in the level B gravels. This distribution was planned to monitor piezometer level variations with time along construction duration, excavation sequence influence, etc.

The installed inclinometers, were designed, calibrated and field tested by the "Laboratorio del Transporte y Mecánica del Suelo" at Madrid. The inclinometers are able to detect movements of points at depth relative to head with an accuracy of less than 0.5 mm. They are made out of PVC tubes, with 170 mm. inner diameter, and assembled using 2 and 3 m. lengths. Tubes ends are fitted for a better cement gweing. In each of the tubes and along two opposite diameter generatrixes two rows of microlamps have been fixed.

Once the inclinometer has been installed, the microlamps position is determined by means of a level and optical plumb and measured at the head by a scaled protactor (fig. 2). It was also necessary to measure absolute horizontal head movements. These measure ments were achieved by using a tachymeter set up at a station and shooting a flag attached to the inclinometer head. Backward shots were made to a stable point located at a convenient distance, as can be observed at fig. 1.

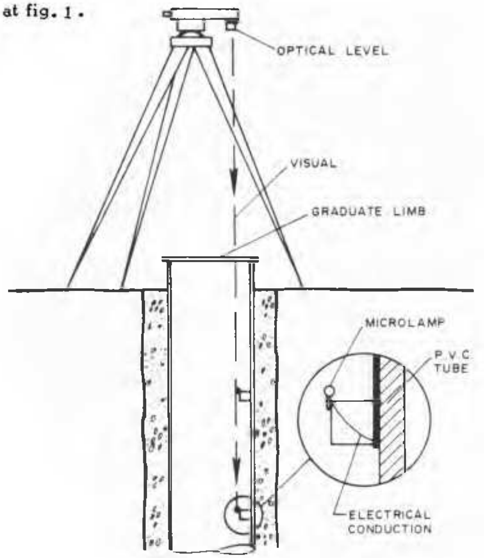

Fig. 2 Lay-out of the inclinometer features of operation.

In this manner microlamps horizontal movements, i.e. soil displacements, referred to inclinometer head can be detected, and subsequently absolute soil movements, once shaft radius and head movements have been implemented in the analysis.

In short, we can say that the installation of inclinometers is composed of the following operations: a) Mounting tubes in one unit with a definitive length. b) Drilling a borehole and tubing it. c) Lifting the inclinometer with an extra prop (steel pipe). d) Placing the inclinometer vertically in the borehole and joining its bottom to a dead weight. e) Fitting the inclinometer inside the borehole. f) Withdrawal of tubing and proceeding to bentonite-cement infection.

Readings from all instruments were recorded periodically, from October 1974 to August 1975. However the following phases can be differentiated as construction progresses:

1st Phase.- Instrumenting two panels and surrounding ground as well as corresponding measurements during job execution (October to December, 1974).

2nd Phase - Taking readings inside the shaft, during ground excavation, sampling "in-situ" tests, etc. (January to September 1975).

FIRST PHASE

Once the reinforcing steel cage was placed, several cell readings were taken to measure pressures and stresses at different staves along the construction of panels: a) After steel cage installation has been completed (without concreting, neither activation of hydraulic jacks, that were attached to the steel structure). b) After hydraulic jack activation. c) During concreting and as concrete levels up cells location. d) Few hours after concreting, etc.

In fig. 3 panel 24 pressure on cells variations can be

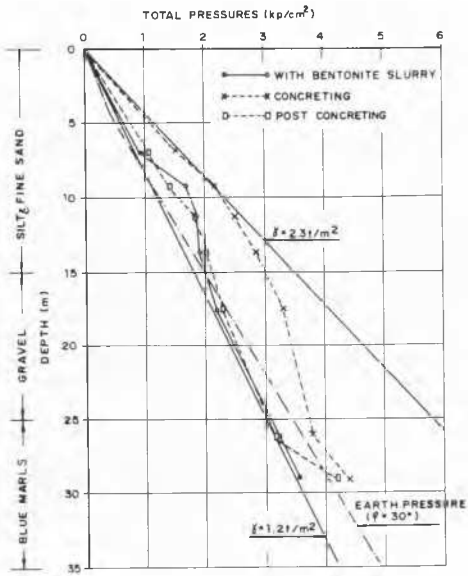

Fig. 3 Observed total pressures

observed. Pressures were taken us depth in three stages, as follows: With cells in their ultimate location, but only with bentonite inside the panel; during concreting (shown pressures correspond to the maxima recorded during this operation) and after concreting (approximately 24 hours later). In panel n. 12, results were similar (differences less than ± 5 %).

For clarification, theoretical total pressures have been also plotted. These pressures correspond to the case in which panel is completely full of bentonite (with a maximum slurry density, γ_1, of 1.2 Tn/m^3); with fresh concrete (density would be 2.3 Tn/m^3). Total pressures plot were performed in order to study pressures values at the two limit situations that contain the above mentioned phases. As can be observed, for practical purposes, all measured pressures are included between those two limits.

Also a straigh line is displayed representing stabilized pressures, in the case of homogeneous soil and internal friction angle of 30º (lower value for the soil). These pressures, e_{tz}, at the upper part of the ground are lower than those assumed for the bentonite, e_{lz}. This is thought to happen because of average ground water level is located at 4.5 m. and bentonite reaches up trench surface (fig. 3).

Comparing the above two values e_{tz} and e_{lz}, we can see that the former is lower than the latter with a difference up to 8 m. depth, at this depth both values are levelled up. Variation in internal friction angle, φ', does not produce much effect. It φ' value were 25º or 35º instead of 30º the levelling up point would change approximately from 7 to 9 m.

During the phase of only bentonite in the trench, measured pressures are very similar to the theoretical ones of $\gamma_l = 1.2$ Tn/m^3, except in cells situated at 9 and 12 m. depth, where they are slightly higher.

Upon concreting, pressures raise up to nearby line $\gamma_h = 2.3$ Tn/m^3. Down to 12 m. depth, the measured and theoretical values are practically identical. Differences are enhunced as depth increases. Pressures measured at greater depths correspond to a maximum fresh concrete head that oscillates between 13 and 15 m. Because of the rheological characteristic of concrete, it can be considered that a silo effect take place inside the trench as concreting comes up; the result is a pressure drop at the deeper zone (about 30 %).

In the next stage, say, at least 24 hours after concreting, pressures decrease respect to values measured during concrete pouring, but they stabilize, in general, at values of the same order of magnitude, t'hose of the bentonite phase. The mean value of pressure differences between both stages is of the order of $\pm$ 0.15 Kg/cm^2 at the test panels.

In the following construction stages, i.e. excavation of neighbour panels, concreting, etc. it is observed that upon construction of adjacent panels pressure values build up slowly in relation to the initial ones, although then they fall dawn and get stabilized at a value slightly higher than initial, when the slurry trench wall works is completed.

The measurements recorded by the cells installed in the center of reinforcing steel cage to measure circunferential stresses, follow a similar trend that the earth pressures cells. In the readings corresponding to the final completion of the cast-in-situ wall, very low values began to be recorded by the highest located cell in panel n.24. These abnormal low values continued till the end of the works. It was believed that it was due to either the cell get out of order or the joint between panels 1 and 24 could have been left poorly jitted, leaving a some kind of discontinuity in this zone of the wall.

In the First Phase the inclinometers movements were carefully measured. If the analyze closely panel n.12 inclinometer, it can be observed (fig.4) that upon panel excavation, ground movements take place outwards, i.e., away from excavation. Upon completion of this excavation, maximum horizontal movements (at 8 m. depth) reached 5.4 mm., whereas head moved less than 1 mm. Movements were carried on, building up even after panel concreting completion, at that time maximum displacement reached 7.5 mm. During the following stages (adjacents panels excavation, concreting, etc.), movements did not show any relevant changes. In the other panel (n.24), the observed phenomenon was identical, except for the maximum horizontal movement that showed sligthly lower values (about 5 mm.).

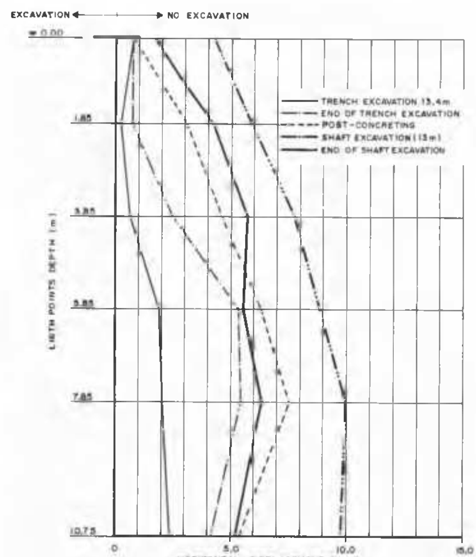

Fig.4 Observed lateral displacements (Inclinometer n.12).

Along this First Phase, measured piezometers levels indicated maximum variations ranging between 4.5 and 5.4 m., yielding 4.5 m. value in the safe size as concerning to comparison between earth and sherry pressures . The piezometric difference between gravels and silty sands zones were really insignificant (of the order of 20 cm. as maximum).

SECOND PHASE

As it has already expressed shaft excavation was carried out in such a way that the cast-in-situ wall, was built up by means of a lining or reinforced concrete reinforcement rings 0.40 m. thick in the upper part and 2.80 m. high.

In panel n.12 (fig.4), and in relation to the First Phase, absolute horizontal ground displacements were increasing as shaft sinking were progressing, leading to a deformation profile that parallels the are obtained at the end of the First Phase. Movements were of higher values in the lower zones (at 8 m. depth) raising up to 10 mm. displacement. Upon excavation ending, movements were similar to those at the end of the First Phase (± 1 mm.).

In panel n.24 and in the excavation process, a ground movement took place towards shaft inside, this displacement was mainly observed at 7.0 m. depth, with an increase of 3 mm. upon reaching 18 m. excavation. Afterwards, the inclinometer get back to the situation held at the end of the First Phase.

In short, this is to say that in both inclinometers and during shaft excavation inwards and outwards displacements are observed with maxima of 3 mm. At the end of the excavation operations and once several readings were taken at the beginning and at the end of the day, the situation in the inclinometers were practically the same that at the end of the First Phase.

Total pressures acting on the wall were also recorded along this second Phase. All these measurements have been compiled and results drawn in fig. 5 for

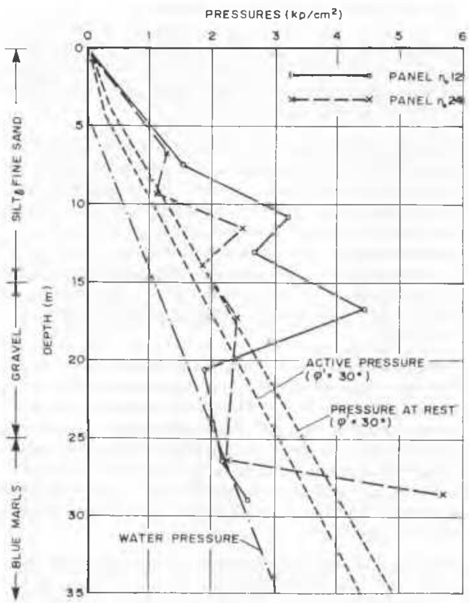

Fig.5 Total earth pressures on the slurry trench wall (end of phase n.2)

the two test panels all along the depth. In the same figure, and for comparison purposes, total theoretical pressures have been represented for an homogeneous stratum with a unit weight 1.8 in the dry zone and submerged unit weight 0.9 below the ground water level (sited at 4.5 m depth). Active pressures and at rest have been represented for internal friction angles, φ, of 30º. Pore water pressures have been also depicted.

In panel n. 12 and down to 20 m depth, total pressures came out to be slighty higher than theoretical ones, whereas beyond that level they become substantially lower; in such a manner that they become of the same order of magnitude that those corresponding to water. Values lower than the average of measurements correspond to cells placed with hydraulic jack.

Upper zones measured pressures correspond to earth pressures with internal friction angle below 15º, which does not seem to be acceptable when comparing with results from penetration or laboratories tests.

In panel n.24, measurements get closer theoretical values. Down to 20 m. depth measured values correspond to earth pressure at rest of the order of 20º or higher. However, in the lower part, at 27 m. depth, observed pressure is practically of the same order that water pressure, whereas at 29 m. pressures increase noticeably above average. Similarly that in panel n.12, cells placed with hydraulic jack display lower values than those ares fitted with the frame, when compared to the averaged pressures values.

In respect to circunferential stresses, they were determined by means of hydraulic cells utilization as well as by extensometers.

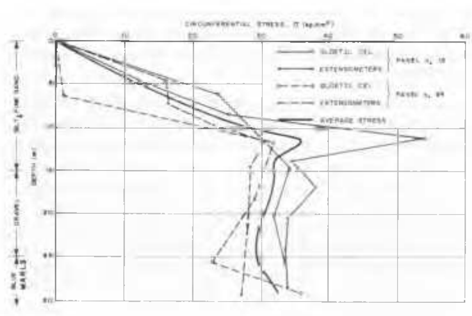

Fig.6 Circunferential stresses measured in the slurry trench wall

Fig. 6 shows the stresses obtained as function of depth (Panels n.12 and 24, respectively). It includes measurements taken with Gloëtzl cell and those detected by electrical extensometers as well. The displayed values correspond to stresses reached in each and every point, once shaft sinking was completed. Also a straight line corresponding to average pressures computed with all available measurements, is included.

In fig. 6 it can be observed, in general, that the order of magnitude of stresses calculated with the two measuring systems is similar, except for the extensometer located at 11.5 m. depth. At this point the measured stress is substantially higher than those corresponding to hydraulic cells as well as all the remaining measurements (approximately 57 Kg/cm^2. In all the others instruments no stresses above 38 Kg/cm^2 were recorded. As already mentioned a malfunction is observed in the hydraulic cell of the panel n. 12 upper part. Below 12 m. depth, stresses remain practically stabilized, showing values ranging from 30 to 34 Kg/cm^2.

In this second Phase, piezometers levels changed between 3.5 and 5.5 m., depending on seasons. Shaft sinking did not affect at all in those fluctuations, which were only influenced mainly by rainfall in the area.

ANALYSIS OF RESULTS

Although it is rather difficult to justify theoretically, in a cuantitative manner, the movements which took place during the First Phase of the excavation, a cualitative explanation is given.

The displacements recorded when the concreting operations were finished ranged from 4.5 to 7.5 mm (11 m. depth). The pressures due first to the slurry fluid and then to the concrete are higher than the earth pressure at rest. Consequently the displacements are towards outside the shaft. After the hardering of the panel, the lateral soil displacement are practically zero. Esencially, the total soil deformation is mainly produced during the trench excavation and the concreting operations. This movement is not influenced by the nearby panels construction.

Fig. 7 shows the calculated displacements of the soil, due to the concreting operations of the corresponding panel, according to the following hypothesis: 1ª) The soil is a elastic half-space, with a Poisson's ratio ranging from 0.3 to 0.5 . 2ª) The concrete acting as a fluid, of 2.3 density(the silo effect is not taken into account). 3º) At the depth corresponding to the cuaternary layers, the Mindlin's solutions may be used for calculations of soil deformations originated by the concrete pressures.

A comparison of the theoretical and field values have been done (fig.7). It can be concluding that the soil subjected to lateral concrete pressures, have an elastic behaviour. The bulk deformation modulus ranges from 950 to 1270 Kp/cm^2. This value corresponds to that obtained in vertical plate tests.

For the upper cuaternary soils the vertical deformation modulus is 200 Kp/cm^2. At Spain, the ratio

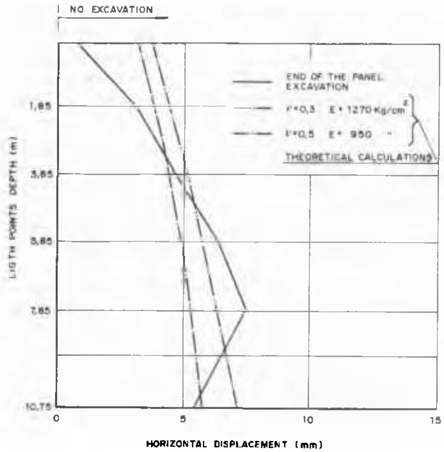

Fig. 7 Comparison between field measurements and theoretical calculations.

between the vertical and horizontal moduli ranges from 1.5 to 4. In the case analised this ratio varies from 4.7 to 6.3; i.e., the moduli ratio is sligthly higher than the normal values. The dryer condition in the upper cuaternary soils (4.5 m. depth) could justify this difference.

The final deformations which occur during the Second Phase of the construction are supposed to be due to thermal differences between the end of the cast-in-situ wall construction (December, 1974) and the end of the shaft excavation (July, 1975). The average annual temperature of Sevilla City is 17ºC. In summer time, the average temperature is 28ºC. The expansion of the concrete wall, due to thermal difference, give place to displacements outwards the shaft.

An evaluation of this lateral movements is made with the following basic hypothesis: 1ª) The concrete temperature is 6ºC higher than the environement temperature. 2º) For slow increments of temperature, the concrete deformation modulus is the 33% of the normal value, i.e., 100.000 Kp/cm^2. 3º) The horizontal soil deformation modulus is 900 Kp/cm^2. 4º) The thermic linear expansion ratio is $0{,}5\text{-}2 \times 10^{-5}$.

The recorded values for the displacement in this phase vary from 2 to 3.5 mm. These values are in good agreement with their obtained theoretically (1-4 mm), for an elastic ring constrained laterally by the ground, subjected to thermal expansion. (The corresponding increment of the earth pressures on wall vary from 1 to 4 T/m^2).

Total pressures on the circular wall, as already indicated, between 8 and 17 m. surpass the theoretical pressures, and even they are doubled in panel n.12. It might be thought that it is due to mistaken readings or to cells malfunction. The numerous measurements taken, the gradual increase of those pressures upon inside shaft sinking, measuring instruments check-ups, etc. proved, in our judgement, the inadequacy of those hypothesis. If a value of 4 T/m^2 (hypothetical thermal increment of earth pressure) is descointed to the average field pressures, the result corresponds fairly well to the active or at rest earth pressure, assuming either φ'=20º or φ'= 25º. The values obtained in the laboratory for the internal friction angle of the cuaternary soils are in this range.

Regardless the above considerations, stresses prevailing in the concrete can be, as a whole compared with those that could be expected from measured pressures. If we assumed that wall informations are negligible in relation to diameter, it could be admitted with enough approximation (since that pressure gradient is not very high: less than 1 Kg/cm^2 in 5 m. depth) that the relationship between earth pressures, p, and stresses acting on the concrete, σ , is controlled by the to-called "pipe formula": $\sigma/p = r/e$, being r circular wall radius and e its thickness.

For the above radius, the value 12.57 m was adopted. But, insomuch the initial thickness of the wall, 0.80 m., varies as inner lining is built, it was accepted the criteriar of taking e = 0.80 m in the upper zone down to the first lining ring (6 m depth). In the next zone where the lining is 0.40 m. thick, the value of e was taken as 1.20 m. In the lower zones, the criterion was to adopt the total thickness (wall plus linings) all the war down to one ring (2.8 m) below the zone with that mentioned final thickness. In such a manner come up the law named: "theoretical relationship", so denominated in fig. 8.

This procedure of determining the values σ/p along the wall might be very questionable, insomuch pressures are acting with a certain magnitude from the site where thickness is only the initial (e=0.8 m) down to the bottom of the excavation ($e_{máx}$ = 1.6 m), for this reason the choice of the mentioned values ought to be considered as a lower limit for σ/p. The upper limit can be taken as the value corresponding to e = 0.8 m. In fig. 8 is also shown the average value for σ/p, which was determined using all the available experimental values. The range of variation of the measured have been drawn as well.

If the above mean value is compared with the theoretical are could be observed a considerable similarity. Indeed, in the upper zone (down to 20 m depth), the mean value decreases from 15.5 (as compared with 16.7 theoretical) to 10.2 (11.2 theoretical).

Therefore, we can say that the comparison between theoretical and field values indicates that σ/p in fact, varies gradually, from values corresponding to the upper limit to those of the lower one, although with no total adjustment to the theoretical value of the latter.

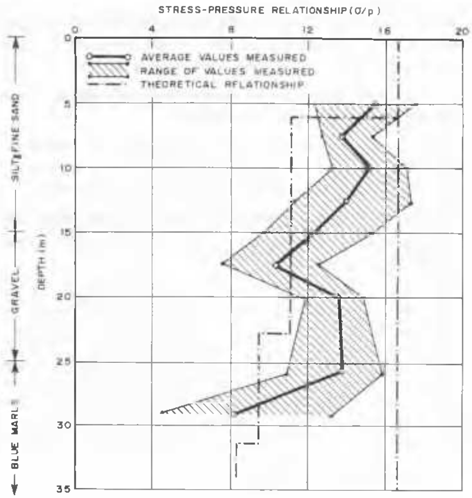

Fig. 8 Stress-earth pressures relationship at end of shaft excavation.

It indicates that the strict theoretical relationship (lower limit), as already pointed, is not quite adequate. The real σ/p parameter is very much conditioned by the initial wall thickness. It is believed that because of lining construction method, it will not start to work inmediately, being the wall the one that starts working from the very beginning. To achieve that, the stresses will not be uniformely distributed through all the wall and lining thickness. Nevertheless, the use of the pipe formula could allow, taking into account initials and finals thicknesses to shorten circunferential stresses with enough accuracy.

CONCLUSIONS

1ª) The execution of the sluny trench wall, even in granel zone, can be carried out without major difficulties, the later seepage is not excesively considerable.

2ª) In the proximity of the wall and during panel construction the horizontal ground deformations reached up to 7-8 mm at 7 m depth. Those movements are originated by the high pressures that the inside panels materials exert as compared with those ones exerted by the ground.

3ª) During inside shaft excavation, new ground movements are observed. They are thought to be due to thermal variations.

4ª) After shaft sinking, the average total pressures in the cuaternary zones are greater than expected which could be caused by thermal strains.

5ª) The two methods of circunferential stress measurements gave similar results.

6ª) The pipe formula allows to shorten the circunferential stresses, but always taking into account the initial and final thicknesses of the shaft wall.

7ª) Shaft excavation did not have, for practical purposes, any repercussion on the ground water table prevailing in the area.

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