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# THE CONSTRUCTION OF STATIONS AND TUNNELS BY SLURRY TRECNCH, METHOD IN THE MADRID METRO EXTENSION

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**ABSTRACT**

Recently a new Extension of the Madrid Subway (Spain) is carried out (1995-2003). In 8 years more of 105 Km of tunnels are builded by several procedures: Traditional non-mechanized excavation, precast linings by E.P.B machines, cut and cover system, etc. In this paper the basic criteria adopted for the design of tunnels and stations built with the cut and cover method (reinforced concrete continuous and discontinuous walls) are analysed. Initially, to this purpose the geotechnical properties of the Madrid ground are reached, in order to establish a group of unified desion geotechnical parameters. The hypotheses made for the numerical analysis of in situ walls are shown (calculation model, water pressures, etc), together with the installed instrumentation and the obtained results.

## INTRODUCTION

In the 1995 to 2003 period the Madrid (Spain) Regional Government built the Underground Extension providing services for more than 105 km of double track and 70 stations as well. All of those stations –with two excerpts- were built by cut and cover or Milan's method, as well as around 8 km. of tunnels. All those works were made through massive use of machinery to build continuous wall –or pile discontinuous walls- which made possible the building of nearby near of 400.000 $m^2$ of this type of element for earth retaining structures. This technology is also used in provisional works: tunnelling shafts, access ramps, etc.

As consequence, the Works Management decided to unify the design systems for continuous walls in such a way that:

- Soil types in Madrid should be analysed and groups or predetermined types of strata should be defined, each with easily defined geotechnical indexes.
- Numerical models for the wall calculation should be selected.
- Geotechnical parameters should be defined and recommended for each stratum type, suitable to the calculation model.
- Additional calculation criteria were established (maximum movements, embedded lengths, etc.)
- Also for to measure the behaviour of these structural elements, a special auscultation and control has-been designed.

## GEOTECHNICAL CHARACTERISTICS OF THE MADRID GROUND

The strata constituting the zone where the Madrid City and its surroundings are placed is included within three main groups (Escario et al, 1981; De la Fuente y Oteo, 1985):

- The quaternary soils, both of natural origin (alluvial) and man made origin (rubbish filling, excavation products, anthropic fills, etc). They belong to soft and weak materials and with diverse granulometry. Generally the anthropic fills are a collapsible material.
- The Miocene materials, constituted of hard stratified fissurated clays, with grey-brown, green and dark grey coloration, they have some sulphates and alternate with gypsum and marl gypsum. They are placed in the South and Southeast of Madrid. They include smectite materials (specifically, sepiolitics) and normally they are expansive. The contact between these materials and the pliocene ones took place through a lateral facies change.
- Pliocene materials, constituted of arkosic type sediments, yellow to brown colour, coming from erosion of granite and gneiss rocks of Guadarrama Sierra, near to Madrid. They belong to materials with different fines contents (7 to 85%) that have been deposited as rolled and that appear intercalated between themselves. They have high consistency and are partially cemented by quartz and feldespar crystals (De la Fuente y Oteo, 1985). This arkosic ensemble is normally called Facies Madrid.

Principally, the Underground Extension es carried out into the pliocene soil kinds. Only the 10% of the boiled length has been excavated in the miocene materials. In the Pliocene materials it is normal to distinguish some geotechnical units: a) "Miga" Sand (with less than 25% fine content). b) "Tosquiza" Sand (25-40% fines). c) Sandy "Tosco" that is a sandy clay (40-60% of fines). d) "Tosco", that is a rigid clay, with a fine content ranging from 60 to 80%. The Miocene materials are constituted by hard fissured clays, locally called "peñuelas", with different sulphate rock and gypsum contents. The granulometry of these materials is show in fig. 1. The mentioned differentiation is usual in Madrid, but it must be mentioned that, some times, the "Tosquiza" Sand and Sandy Tosco" are grouped due to the fact that the granulometric and plasticity difference is very small. Fig. 2 shows the plasticity of all these formations, in which it can be appreciated that Pliocene materials have from low to medium plasticity (except some samples of "Tosco", that can range up to a level of liquid limit of 60), whereas the miocene "Peñuelas" are of high real clays, although some times are of low density (the sepiolite clays have a dry density ranging between 800 and 1200 Kg/m$^3$).

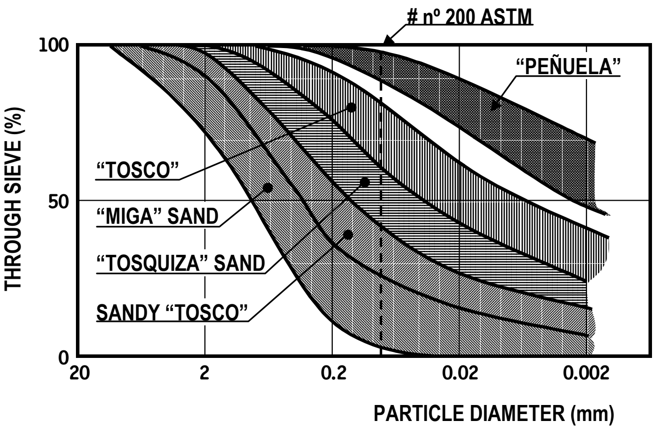



*Fig.1.- Granulometry of the Madrid Soils*

The unconfined compressive strength have the tendency to increase with the fine content. Even the Miga Sand (with fines in the level of 10%) allow us to shape test_samples with strengths about the orden of 0,1 – 0,2 MPa. The most cemented materials ("Toscos") can unconfined approach strengths up to 2,0 MPa. In the Miocene materials the clay parts can also have high strengths (1-2 MPa), nevertheless the gypsum can double these values.

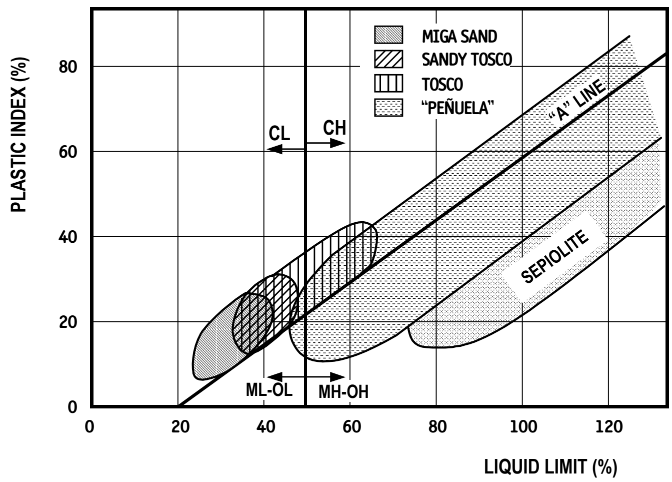



*Fig. 2.- Casagrande´s Plasticity Chart of the Madrid Soils*

The deformability of Pliocene materials is normally low; this is due to the cementation introduced by the feldespar and quartz crystals. The deformation moduli abtained are very high. For instance, 800 - 1200 MPa in the Miga sand. Nevertheless those values belong to a stress path that assumes an increase of compression of the vertical stress and the horizontal stress. Under the case that it would be needed to calculate the produced subsidence when excavating a tunnel: a reduction of the vertical stress is induced, in such a way that there is a real extension on thetunnel crown. In this case the moduli shold be used, those are one half or one trhird of the moduli in compression ( Table1).

| SOIL | BULK DENSITY (Kg/m$^3$) | UNCONF. COMPRES. STRENGTH (MPa) | COHESSION c' (KPa) | INTERNAL FRICTION Φ' (º) | DEFORM. MODULUS (SUBSID.) e (MPa) | POISSON'S RATIO, ν | LATERAL REACTION COEFF. K(t/m$^3$) |
|---|---|---|---|---|---|---|---|
| Anthropic Fill | 1.800 | 0-0,1 | 0 | 28 | 8-10 | 0,35 | 2.000 |
| Compacted Fill | 2.100 | 0,1-0,2 | 20 | 34 | 100 | 0,28 | 8.000 |
| Alluvial | 2.000 | 0-0,1 | 0 | 32 | 10-15 | 0,32 | 5.000 |
| Quaternary Sands | 2.000 | 0 | 0-5 | 34 | 30-60 | 0,30 | 8.000 |
| Miga Sand | 2.000 | 0-0,3 | 5-10 | 35 | 55-75 | 0,30 | 12.000-20.000 |
| Tosquiza Sand | 2.050 | 0,2-0,4 | 10-15 | 33 | 80-100 | 0,30 | 15.000-20.000 |
| Sandy Tosco | 2.080 | 0,3-1,0 | 20-25 | 32,5 | 130 | 0,30 | 25.000-35.000 |
| Tosco | 2.100 | 0,5-2,0 | 30-40 | 30 | 150-180 | 0,30 | 30.000-40.000 |
| Tosco with high plasticity | 2.060 | 1,0-2,0 | 40-80 | 28 | 200 | 0,28 | 40.000 |
| "Peñuelas", grey and green | 2.000 | 0,7-2,5 | 50-60 | 28 | 200 | 0,28 | 35.000-50.000 |
| Green "Peñuelas" with gypsum | 2.100 | 1,0-4,0 | 50-80 | 30 | 250 | 0,27 | 40.000-55.000 |
| Softennied "Peñuelas" | 2.000 | 0,2-0,4 | 0-10 | 28 | 10 | 0,35 | 5.000 |
| Micaceous Miocenic Sands | 2.100 | 0-0,1 | 5-10 | 34 | 50 | 0,30 | 10.000 |
| Sepiolite | 1.600 | 0,2-0,3 | 20 | 28 | 300-500 | 0,28 | 20.000 |
| Carbonated levels | 2.200 | 1,0-2,0 | 150 | 32 | 600 | 0,25 | 80.000-100.000 |
| Gypsum | 2.300 | 3,0-5,0 | 70-100 | 28 | 400 | 0,26 | 60.000 |

In fig. 3 the pressuremeter moduli obtained in several boreholes made along the Extension of Madrid Underground (Rodríguez Ortiz, 2000) are shown, in comparison with the extension deformation moduli (i.e., that recommended for subsidence analysis). It can be checked that the extension moduli (mainly deducted though real settlements measured in several points) are in the order of magnitude, or are a little bit higher, that the values of the average pressuremeter moduli measured. (Remember that deformation modulus in charge is in a range about the double that the extension one). This indicates that the pressiometric test has the trend to underestimate – in those materials with some cementation – the compression deformability for the Madrid Pliocenic and Miocenic grounds. Probably this is due to the alteration introduced by the borehole excavation prior previous to carrying out the pressuremeter test.

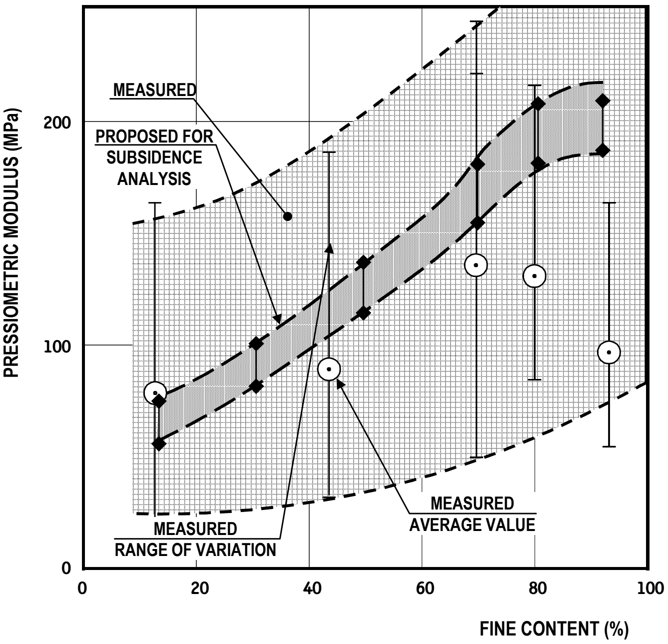



*Fig. 3.- Measured Pressiometric Modulus as a Function of Fine Content*

HYPOTHESES TO THE CONTINOUS WALLS DESIGN

The theoretical model for cast in situ walls (or discontinuous pile walls) was the type which allows the introduction of soil – structure interaction, i.e., it takes into account the limit values for active and passive earth pressures and the watter pressures, taking into account the field wall stiffness. To this purpose, two calculation models have been considered:

- Those that consider a ratio displacement – earth pressure of polygonal type (like those that use similar schemes showed in fig. 4). In these methods it is necessary to establish the active and passive pressure limit values, this is obtained from the soil effective density and its cohesion, c', and the effective friction angle, φ' and the classic formulations from Coulomb or Rankine. Additionally it is necessary to define the soil deformation modulis (strictly for both the active and passive cases, but only one mid value is normally used) or a ground lateral reaction coefficient $K_H$, which defines the inclined line included in the fig. 4. This model is equivalent to considering that the wall rests over horizontal springs (on both sides), with a certain deformability defined by $K_H$ and limit values (active pressure and passive pressure). There are several commercial computer programs that reproduce this model. In our studies the RIDO and the DEVOLA codes have been used.

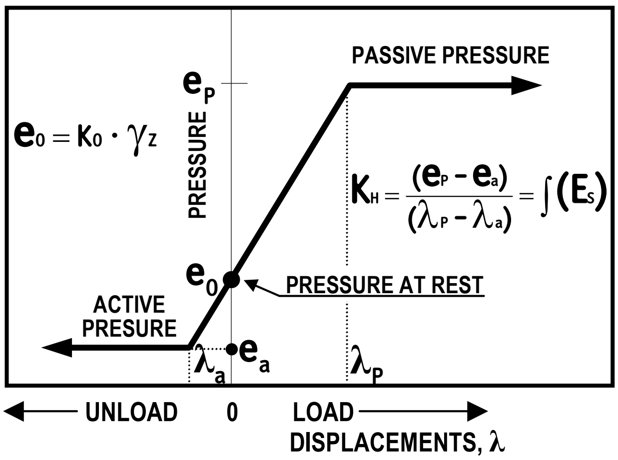



*Fig. 4.- Relationship Between Earth Pressure and Displacements for the Cast-in Situ Wall Analysis*

- Those who consider the wall and all the surrounding terrain, through codes of finite elements, like the RHEO-STAUB or the PLAXIS code. Those codes allows us to consider the soil – structure interaction as a whole and not to concentrate them in springs, this procedure allows us to introduce the arching "effect" that can be produced in the backwall, to calculate the settlements produced by the excavation near and far from the walls. Those any sophisticated systems are used for special analyses

In the Extension of Madrid Underground it was decided:

- To use mainly the RIDO model (and DEVOLA in several cases) for the wall calculations, i.e., to evaluate its movements, bending moment and shear stresses.
- The coefficients of active and passive earth pressures are stabilished by the Rankine equations, assuminy nule the friction between the soil and the walls.
- The pressure at rest, Ko, in tertiary materials was considered equal to 0.8
- The excavations should be reproduced following the construction phases (partial excavations, struts or slabs placement, introducing their deformability, etc).
- In the provisional phases it can be accepted for the wall steel reinforcement a safety coefficient between 1,25 and 1,40 (In fhe basis of Spanish Regulations for provisional situations). For the definitive situation calculation (total excavation, plus cover, plus definitive struts, plus invert) it is necessary to use the higher coefficient of 1,60 (as required by the Spanish Regulation for the reinforced concrete). Generally the most predominant phase was the provisional one with the maximum excavation but without construction of top slab or invert. In the last phase, often the concreting of the invert, a variation of the phreatic level and variation of the upper soil cohesion are introduced.
- Taking into account the alternation of permeable layers ("Miga" sand or "Tosquiza" sand, with permeability coefficients in the level of $10^{-2}$ a $10^{-4}$ cm/s, see comments) and impermeable

(“tosco”, with a permeability coefficient in the level of $10^{-7}$ cm/s) the Madrid phreatic levels are not continuous, but can be said to be “perched water tables”, i.e. water streams that pass around sandy zones and which remains completely separated by highly impermeable strata. For this reason it is not logical to consider continuous phreatic levels making pressure against the walls (the basic phreatic level, continuous, in Madrid is near 70-90 m deep in the centre of the City). Therefore the wall design it was undertaken with water pressure laws as shown in fig. 5, assuming that the clay level had a minimum thickness of about 4-5 m

- The deformation modulus for the wall concrete was assumed equal to 30,000 Mpa. It was necessary to provide a characteristic strength after 28 days of 25 MPa (and an Abraams cone of 16-20 cm).
- The maximum horizontal displacement obtained in the calculations cannot be higher than 20 mm (except under some cases where due to the buildings proximity it was limited to about 12 mm). In provisional and exceptional cases, without nenby mildings, a horizontal maximum displacement of 40 mm has been assumed.
- The minimum wall embedding in the lower terrain should be of 3 m and a maximum of 6 m. Under the case that the embedment should take place in terrain having a predominance of gypsum (in fact a rock, even considered soft, due to the penetrating difficulties), an embedment of 1.5 m was allowed.
- The excavation depth to place struts or slabs was assumed to be 0.5 m under the element to be installed. In the case of curved invert, the excavation calculation depth considered was that corresponding to the average depth of the invert.

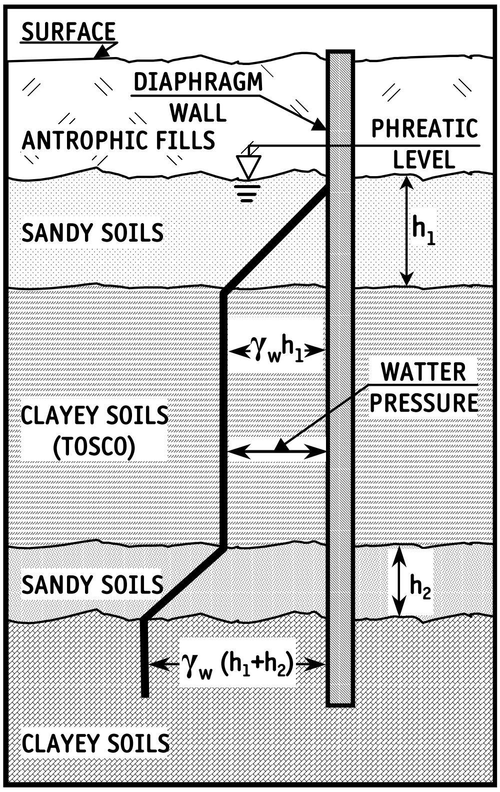



*Fig. 5.- Hypothesis On The Wqtter Pressure In Soils With Different Permeability*

## CALCULATION PARAMETERS

One of the most important problems around all this simulation was to determine the effective parameters for the shear strength (cohesion, c’ and internal friction angle, φ’) representing properly the different ground types considered.

The parameters were initially fixed by the authors of this paper on the basis of several personal experiences and to the data previously published (Oteo and Moya, 1979, etc; Escario and others, 1981; De la Fuente and Oteo, 1985). But, from the beginning of the Underground Extension, several boreholes and tests (traditional and special ones) were performed in order to try to estimate properly the c’ y φ’values, and to confirm the hypotheses developped.

Fig. 6 shows the distribution frequency of the effective cohesion distribution – determined in triaxial tests, with previous consolidation– in several ground types (“Tosquiza sand” and “sandy Tosco” have been grouped together because is similar). For the “Miga sand” case there is a strong accumulation around the range of 5 to 10 KPa (exactly the adopted values) whereas for the “Tosco” the values are grouped place around 15 and 50 KPa (the adopted values are 30-40 KPa). For intermediate materials they provide high frequencies around 10 and 30 Ka (covering about of the 65% of the cases), it was recommended a range of 10-25 KPa. As can be seen, the recommended values correspond to an accumulated frequency of 60-80%.

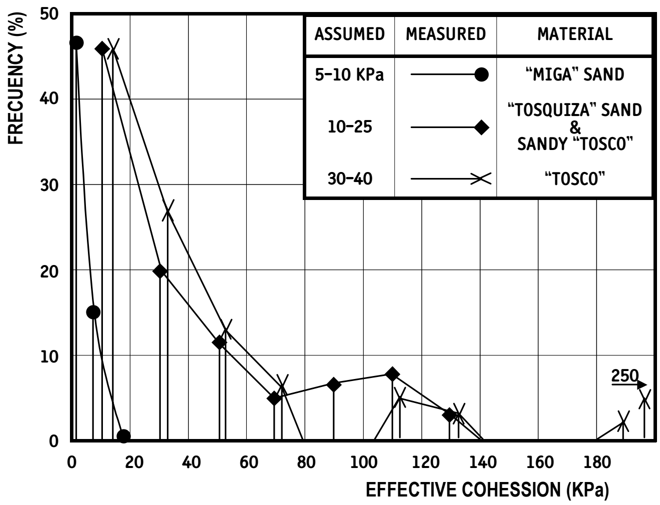



*Fig. 6.- Range of Variation of The Effective Cohesion*

In fig. 7 the obtained variation is represented when determining the internal friction angle φ’, in relation to the fines content (the quartz proportion increases as the fines content decreases, it can be of the level of 50-65% for the “Miga sand” and a level of 40-50% for the “Tosco”). In these mid value determinations, results triaxial tests and simple shear as well as direct shear (cell of 6 x 6 cm) have been included; the latter have a tendency to reduce the medium value. The proposed values are more in accordance with those of simple shear and triaxial (fig. 7).

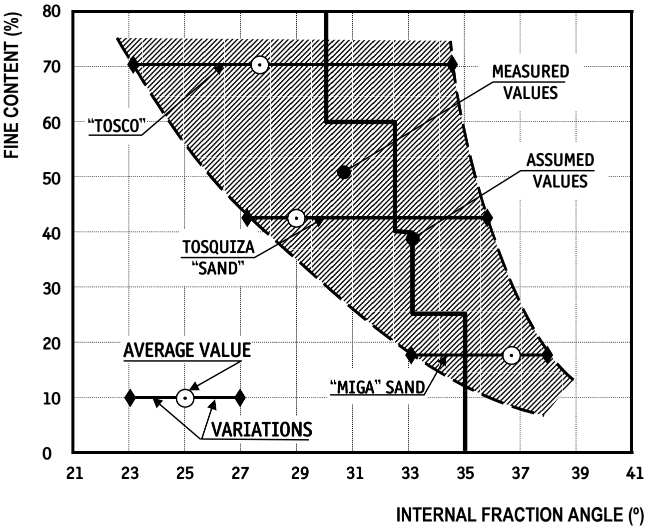



*Fig. 7.- Relationship Between The Fine Content and the Internal Friction Angle*

Finally, fig. 8 shows the values adopted for c' and φ' in relation to the fine content, for all Pliocenic and Miocenic materials, which practically constitute a continuous variation with the fine content. In this figure the lower values for the cohesion are for depths up to 10 m and the higher values correspond to bigger depths. The set of parameters adopted for the calculation of all the walls used in the last two extensions of Madrid Underground (1995-2003, that means more than 450.000 m of wall in tunnels an stations) are listed in the attached table 1.

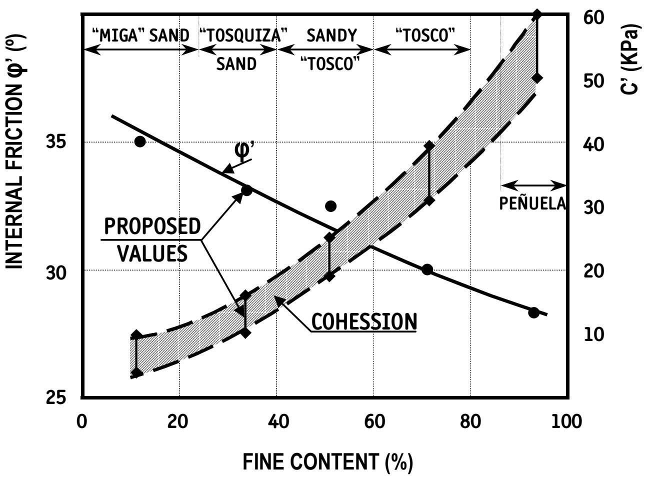



*Fig. 8.-Shear Strength Assumed for the Cast-in Situ Wall Desing*

INSTRUMENTATION

To check the validity of the assumed hypothesis an instrumentation and monitoring programme was established. To this purpose all the stations were provided with instrumentation , at least in two sections with the typology indicated in fig. 9. Additionally also was instrumented (in similar way) tunnel sections made with the shelter of cast in situ walls and shafts (or some station) made with discontinuous pile wall´s. In this last case only inclinometers were installed.

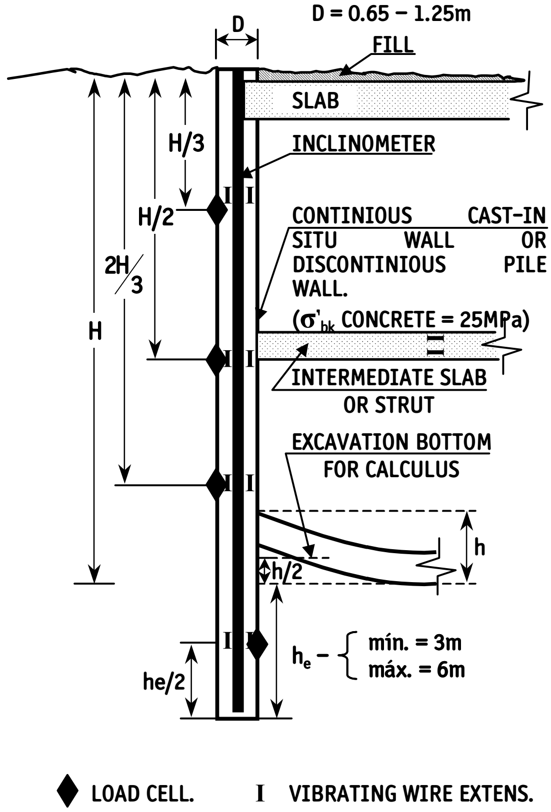



*Fig. 9.- Instrumentation In The Wall For Stations And Line Tunnel*

In addition, settlement references were installed in the ground (rod extensometer 1.5-3.0 m deep), in order to measure the movements near to the wall. Also, little milimitric rules were situated on the building facades

In fig. 10 and fig 11 the results obtained in typical cases are presented. Generally, it can be checked that the measured movements at the end of excavation are somewhat lower than the calculated ones, which means that there is–within this methodology – a determined safety coefficient covering the horizontal movements (in the level of 1.1-1.6), which is considered as very convenient against the displacements that can be experienced the proximity of structures. The curvature types measured and calculated are similar, so in general it can it can be considered that the real safety coefficient in terms of wall bending moments (in service design bending moment divided by the bending moment deduced from the measurements of flection) is in the range 1.5 – 1.7, which is in line with the value of the factor of safety used in the design of stell frames.

EXECUTION OF CAST-IN-SITU-WALLS

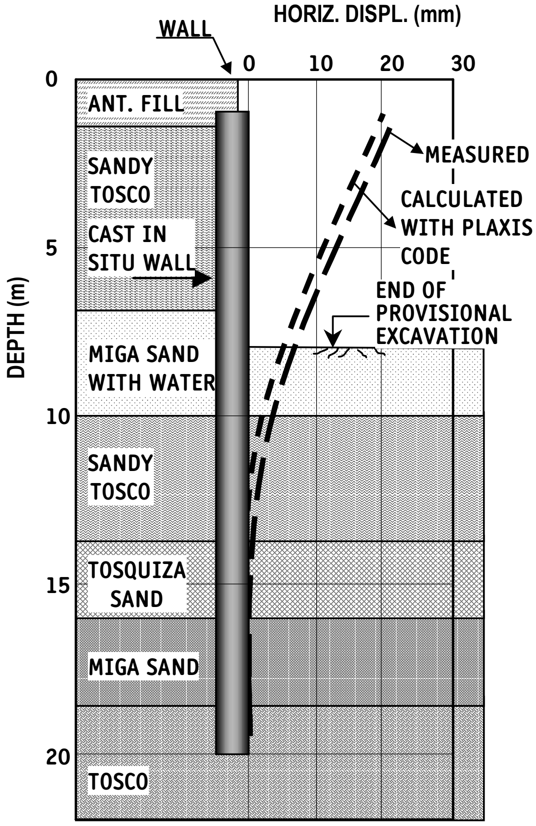
For the execution of all these walls it was used a total of 35 equipment of different marks and characteristics, of the type semiguided with drive spoons. In the zones of gypsum rock it was necessary to use trepan for the embedding by it was autorized to diminish the minimum embedding to 1.50 m (Two stations in the

Extension of Line-1, in the zone of Villa de Vallecas). The tipología of construction of the stations was diverse. a) Two continuous screens without intermediate struts. b) Two continuous screens with pile-pillar intermediate, etc.
In zones in which there was no water, being special works, with singular conditions (for example in the great green zone called Casa de Campo, in which it was wanted to affect the trees less possible) or by the problem of disposition of equipment, the use of discontinuous screens of piles was allowed. So, In the Nuevo Batan station (in which Lines 10 and 5 are superposed) construct again with up to four alignments of discontinuous walls of piles, whose outer faces were located about 20 cm of distance. Also this technique in fourprovisinal pit of attack to introduce the tunnels machines and in diverse shafts of ventilation or aid was used . In both shafts of attack of the section of Alcorcón de Metrosur, the discontinuous screens of piles were united with a beam in head, which anchored to the land with anchorages that formed an angle of 30º with the horizontal, excavating itself, after the order of 19 to 21 m without no intermediate stemts. The groumd was "miga"sand and tosquiza sand and there but water.

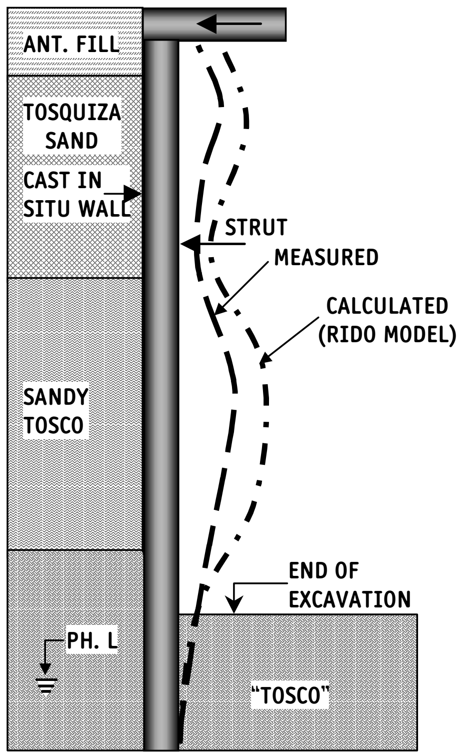
In these cases of discontinuous walls, towards as it was lowered to the excavation a reinforcement with sprayed concrete of 10 cm of thickness, projected on a subject metallic mesh to the piles, after cleaning the area between such.

The control of execution of the props became of the following form: a) it was verified that the cone of Abraams was the established one (16-20 cm) before spilling the concrete inside the excavated panels. b) Were - test samples taken to verify the resistance to 7 and 28 days –c) It was establishing in panels (of minimum length 2,5-3,0 m and maximan 4.50 m) several tubes for verification of the concreting continuty one by means of sonic transparency. The number of tubes in the panel varied between 4 and 6, based on the length of the panel..The number of panels to verify by the transparency method, in each Station was the following one: In first 10% of the constructed panels a 20% were controlled; in second 20% a 10% were controlled; and in the rest a 5% were controlled.

In fig 10 can to see the horizontal displacements (measured and calculated) corresponding to a wall without coaction at head (provisional phase of excavation in a station) and the end of another station that has a slab in head and a row of intermediate struts, since it was excavated more than 18 ms. In fig 11 it appears the case of a discontinuous screen of piles, with an anchorage in head, excavated almost 20 m and that belongs to a shaft of attack of a tunnel´s machine. Also in that figure 11 it appears the case of a station with superior slab in head, but without intermediate strut. .In fig 12 a comparison between the measured maximum horizontal displacements and calculated is introduced Generally, the calculated values are about the 1,3 – 2,0 tines the measured values But in excepcionally cases the measued values higher that the calculated magnitudes (due fo retraction of the upper slabs, excessive excavation without struts, etc.




**a) METROSUR, ALCORCON 2. STATION**




**b) LINE 9, STATION Nº 1**

*Fig. 10.- Typical Measured Displacements*

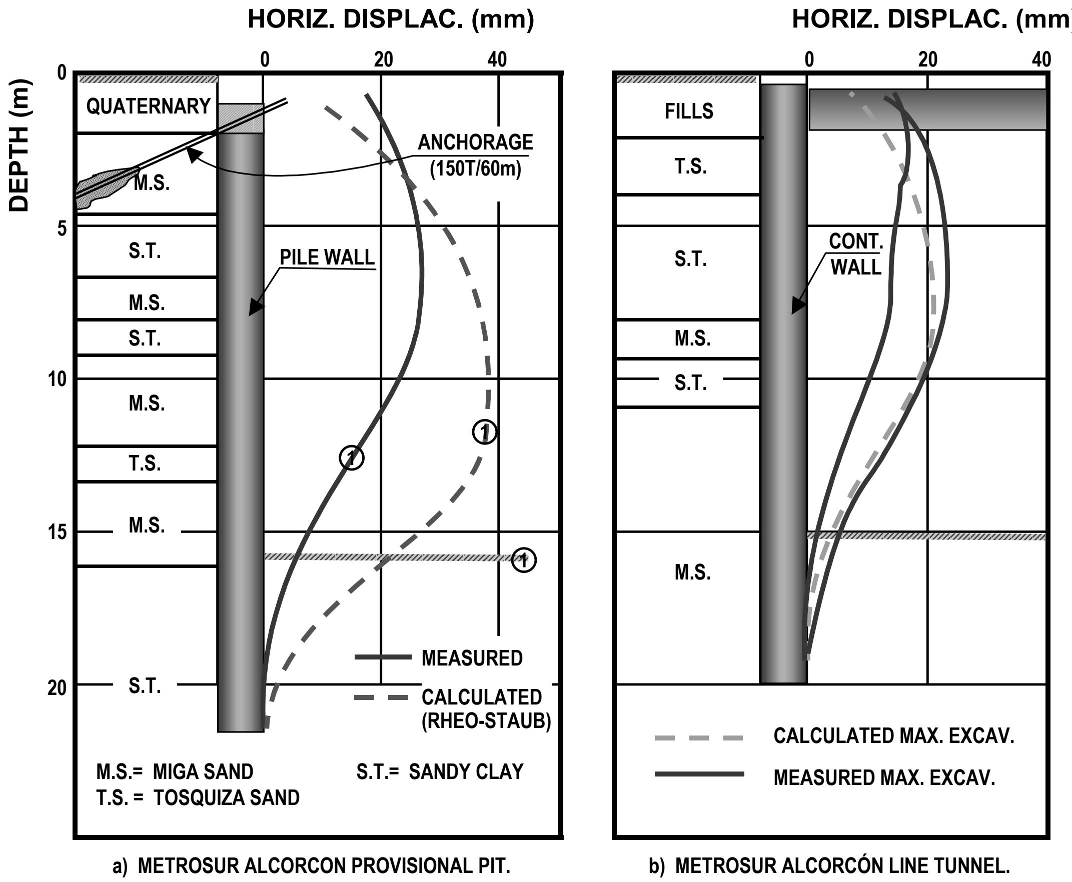



*Fig. 11.- Measured displacements in another walls in Alcorcón (Metrosur).*

The maximum horizontal movements in diverse cases, as much for screens with and without coaction at head appear in figure 13. In the case of walls without coaction at head, the average maximum movement is of the order of one 5 times the average maximum movement of the screens with coaction in head. If the beams embedded are considered below the level of excavation and free or supported in head, with a triangular law of pressures, the relation of total displacements is of 5.2, with which the agreement of the measured and strictly theoretical (equal average values) would be good.

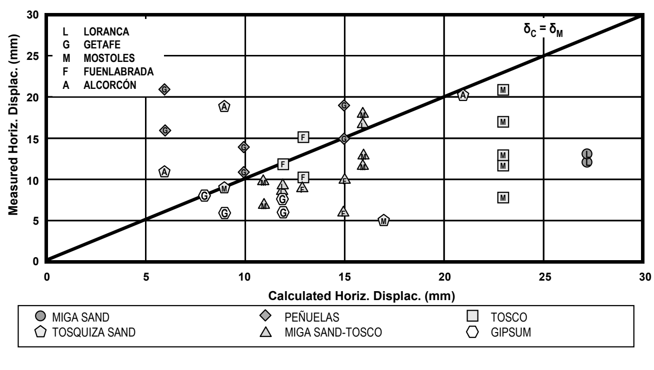



*Fig.12- Comparison between en measured and calculated maximum horinzontal displace ments*

It can adapt, for Pliocene soils in Madrid, the following approximate expressions to determine the maximum horizontal displacement, for a depth excavation H:

a) Without coaction at the head

$$\delta_H/H\ (\%) = 0{,}0030\ H^3$$

b) With coaction at the head

$$\delta_H/H\ (\%) = 0{,}0006\ H^3$$

**CONCLUSIONS**

- A study of the geotechnical properties of Madrid soil has been developped to stablish a serie of the calculus parameters.
- A theoretical model has been selected for walls work design and for movement calculations, introducing soil - structure interaction.
- The instrumentation installed in the walls shows that the measured displacements are in the order of those calculated for the serie situations (differences about 20 - 50 %). Is possible to determinate the orden of magnitude of the maxima horizontal displacements
- As consequence of all the above, it is considered that the selected set of parameters (coming from practical criteria, based on personal experience and contrasted with the

corresponding tests) were entirely appropriate for the design.

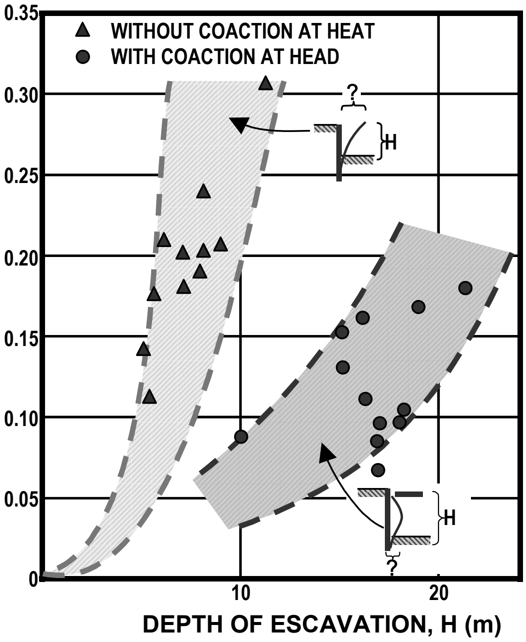



*Fig. 13.- Measured maximum relative deflection in the walls*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