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Experiences in the subsidence problems in Madrid Subway Extension

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ABSTRACT: The Madrid New Subway Extension Plan (1996-99) consisted in the construction of 37,5 km of new underground lines, several procedure kinds has been used: 6 TBM, EPB type, with diameters varying between 6.70 to 8.40 m; cut and cover or Milan's Method; Traditional Madrid System, etc. The soil around the tunnels are constituted by alternated layers of hard pliocenic materials (with different fines content) and miocenic soils. In this paper the differents solutions used to solve the subsidence problems are presented: From the theoretical evaluations and to the practical systems for reducing the damage to the buildings.

1 INTRODUCTION

In the period between June-95 and May-99, the Madrid Regional Authority (Spain) has developed a New Subway Extension Plan. This permitted to increase in 37,5 km the underground network of metropolitan railway, of 120 km of length in 1996. About 33 km have been built in a subterranean way, using six Earth Pressure Tunnel machines (TBM) in 27 of those km (diameter between 6.7 m and 9.38 m). The tunnel works have been done between October-96 and May-99, although the first 35 km were finished during October-98.

The objetives of the EXTENSION PLAN were: a) to improve the interconnections among existing lines; b) to extend the network to the new outlying districts, which have a high population density; c) to bring the METRO service to strategic areas; and d) to extend the network to other municipalities in the RAM.

The extension to other municipalities in the metropolitan area, particularly those lying along the N-III radial highway, where there has been a notable increase in the population, led to an additional Extension of Line 9, with the project known as the "Arganda Train". This project takes the lines as far as the town of that same name, lying to the South-east of Madrid.

To summarise, the PLAN included the construction in four years of 38 km of two track network, all of it underground, with 35 stations similarly underground and two coach sheds. To this must be added the "Arganda Train" with a further 18 km of to track, almost all of it surface, with four stations and one coach shed. With all this, it is expected that the increase in new users of the Metro will imply a 25% rise in the annual flow of passengers.

These works have been accompanied by an important effort in the control and instrumentation field (about 1% of the total budget of the works). This has permitted to know all the ground and near building movements, so they can be rightly protected. But this wouldn't have worked withoud some settlement precaution system that could permit to take decisions about the right protection measurements.

2 THE GEOTECHNICAL GROUND CONDITIONS OF MADRID

Madrid is included in a geologic unity (Central Plateam) that occupies the whole center of the Península Ibérica. This is an Hercinic unit afected by alpinic movements that have originated the Central Cordillera and Toledo Mounts, where the city of Madrid is located. To the North of Madrid there are many granitic and gneistic emergences, which erosion has originated sediments that have parcially filled the Tagus Depression. In the North and Central zones of Madrid these sediments are pliocenic, from the detritic type and they cowe from granites and gneiss. In the South zone, with a facies lateral change, the substratum is constituted by miocenic sediments.

Normally, from the geotechnical point of view, the following typical formations are considered:

- *Anthropic fills* (from human activity) with variable thickness (1-20 m). These concern heterogenous materials, very soft, collapsable, and coming from natural materials mixed up with the remains of rubble, organic matter, etc.

- *Quaternary sediments of alluvial origin*, generally consisting of loose sands and soft silts, associated with the basins of rivers and streams. In the River Manzanares they can reach a depth of 5-10 m.
- *Detrital sediments from the Pliocene*, deriving from the erosion of granite and gneiss from the nearby mountain ranges. These are hard and cemented materials with a fines content varying between 5% and 80% and which are normally referred to as immature arkoses.
- *Base formations from the Miocene*, which start with stiff clay material (around 85-95% fines), very hard (hence their popular name of "peñuelas") fissured, plastic, very often having an expansive nature, and which can include a considerable percentage of gypsum or they can alternate with layers that are clearly gypsum-bearing. Towards the South-east of Madrid the gypsum content usually rises to the point where the other base formation is displayed, gypsum-bearing rocks, in which the variety known as "glauberite" (sodium sulphatc) abounds. This has a marked expansivity and is used in the detergents industry, due to which there are a great many extraction workings in this region. In occasional zones the soil uppert part is constituted by "Softening peñuela", i.e., the weathered hard clay and the antropic fills made by excavated fissurated clay (non compacted).

The pliocenic materials are known from different names, as a function of the fine contents: From the "miga sand" (<25% fines) to the clayeyest "tosco" (60-85% of thin ones, stiff clay). The intermediate materials are the "tosquiza sand" (25-40% of fines) and the sandy clay (40-60% of fines). They are cemented by crystals of feldespat and quartz. In the Table 1, the main geotechnic properties of these materials have been resumed. The antropic fill modulus is about 15-50 times lower than the pliocenic modulus.

3 SUBSIDENCE PROBLEMS. ANALYSIS METHODOLOGY.

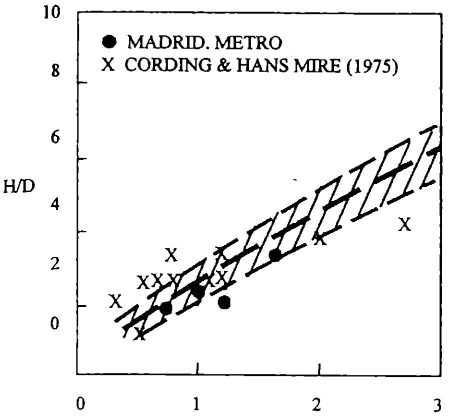
The Methodology used bye the Regional Authority of Madrid to appraise the effect of excavation work on nearby buildings was as follows:

- A geotechnical profile of the alignment of earch work site, with boreholes and penetrometric, pressiometric, laboratory test, etc.
- Definition of standard schematic geotechnical profiles, to the effects of deformation analyses, representative of the most characteristic areas.
- Data compilation on nearby building and installations, plan cartography with location of installations and the tunnel; cross sections with position of tunnel and buildings, definition of number of storeys and basements, data on foundations (when existing), type of structures, general condition of the building, with photographs and anomalies (cracks, open joints, etc.)
- Estimation of subsidence in general and in particular in the area of nearby buildings.
- Damage risk evaluation, taking into account: a) Condition of the building; b) Absolute and differential movements which excavation work might include; c) Thickness of the resistant

Table 1. Summary of the Main Geotechnical Properties

Material	Fine content (%)	Average Bulk dry Density (Kg/m^3)	Water content	Liquid Limit	Plastic Limit	Pressiometric modulus (MPa)	Average cohesion (kPa)	Average International Friction	SPT
Ant. Fills & alluvials	15 - 80	1550	8 - 20	25 - 55	15 - 40	6 - 15	0	28	5 - 15
"Miga & Tosquiza" sand	20 - 40	1860	9 - 14	25 - 40	15 - 20	100 - 200	20	35	30 - R*
Sandy "Tosco"	40 - 60	1850	10 - 28	28 - 43	18 - 25	125 - 250	30	33	40 - R
"Tosco"	60 - 85	1780	10 - 35	33 - 55	20 - 28	150 - 300	50	32	R
"Peñuela"	85 - 95	1650	18 - 35	47 - 100	20 - 40	70 - 100	30	30	50 - R
Soft "Peñuela"	85 - 95	1550	25 - 38	47 - 100	20 - 40	4 - 24	5	28	5 - R
Gypsum	≥ 80	2050	≥ 10	NP - 50	NP - 30	500 - 1000	20	35	R*

(*) R = ≥ 60 blows / 30 cm



— — — SAGASETA & OTEO (1974)

$$\frac{i}{D} = \eta\,(0{,}57\,\frac{H}{D} - 0{,}21)\ //\ \eta = 0{,}75 - 1{,}25$$

$$\delta_{max} = \Psi\,\frac{\gamma D^2}{E^*}\,(0{,}85 - \nu)\ //\ \nu \cong 0{,}2 - 0{,}3$$

$$\Psi = \begin{cases} 1{,}0 \text{ ANTROPIC FILLS} \\ 0{,}5 \text{ MIGA SAND} \\ 0{,}4 \text{ TOSCO} \end{cases}$$

$V_S = 2{,}5 \times \delta_{max} \times i$

ν = POISSON'S RATIO

Figure 1. Sagaseta & Oteo Subsidence Model.

ground over the tunnel crown, d) Thickness of soft soil (alluvial and man made fills) over the resistant ground or over the tunnel crown.

- Having defined the three grades of risk level the following was established: A) The action required to protect the building, if necessary; B) The instrumentation (in ground and buildings) necessary in order to precisely know what movements were really being induced, both in any section before reaching the building and in the building itself.

To stimate the field settlements it was used initially the theorical method by Sagaseta & Oteo, 1974, with several corrections to adjust it to the practice (Oteo & Sagaseta, 1996) and the obtained results in the Line 10 extension (Melis et al, 1997). In Figure 1 this method has been schematized: It is supposed that the settlement law is a Gauss curve (according to the classic Peck-Schmidt suggestion). The inflexion point position, i, is defined in the Figure 1. The maximum settlement, δ_{max}, is given by the expression included in this Figure. The settlement volume, V_s, is deduced by the i and δ_{max} values. δ_{max} is affected by an amprical coefficient, ψ, which values goes from 1 (antropic fills, very slow excavation) to 0.25 (very cemented ground and rapid excavation). The deformation modulus, E*, that is going to be used are the following (in MPa): a) Miga Sand: 50-120; b) Tosquiza Sand: 80-160; c) Sandy Tosco: 110-205; d) Tosco: 160-240; e) Peñuela: 200-300; f) Soft Peñuela: 5-20; g) Antropic fills: 6-15.

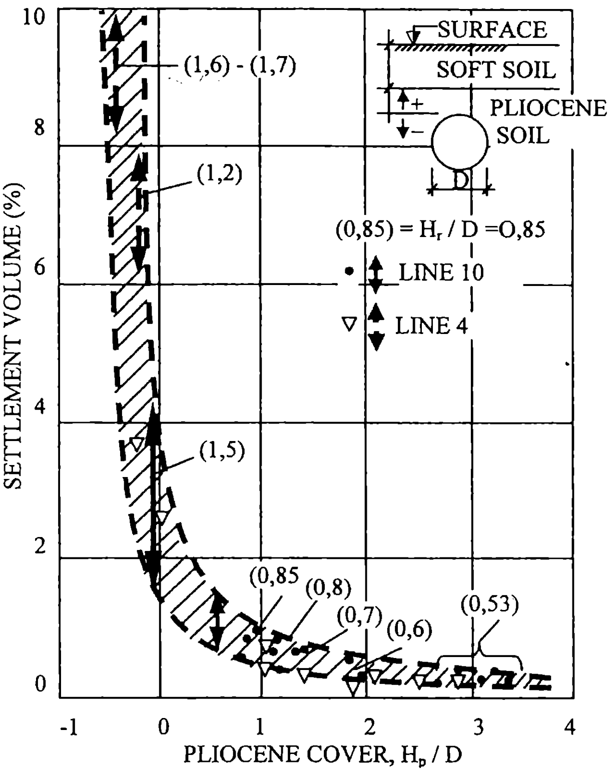


Figure 2. Some settlement volume measured in Madrid Metro Extension.

4 A NEW SEMI-EMPIRICAL MODELLING: MADRID MODEL

Various instruments were installed throughtout the whole work site alignment to measure excavation induced movements: a) Surface references; b) Rod extensometers (from 5 to 20 m in length); c) Inclinometers, etc.

In a first phase, this all enabled orders of surface settlement magnitudes to be established, such as those shown in Figure 6 where the intention was to underline the importance of the thickness of the resistant ground over the tunnel's crown and of the influence of the weak soils (man made fills and alluvial sediments) existing above.

The model schematizated in the Figure 2 was therefored considered valid to simplifying effects. Two stratigraphic levels can be seen there. One clearly weak with a modulus of deformation between 8 and 15 MPa and the other stiff with a modulus in the order of 50-120 MPa (or superior).

Other remarks are to be added to this idea:

- The settlemens evolve rapidly (they generally stabilize between 3-7 days after the tunnel passes).

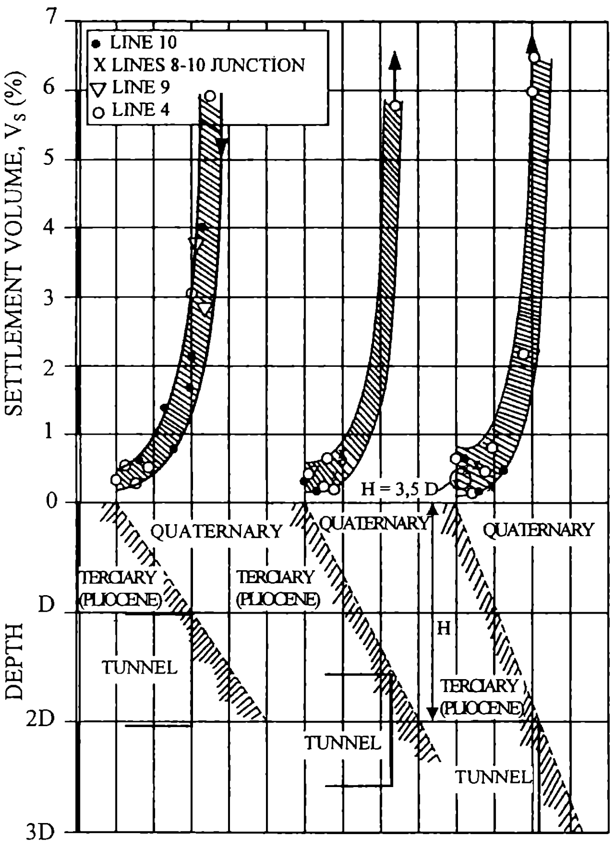


Figure 3. Madrid Model and field measurements.

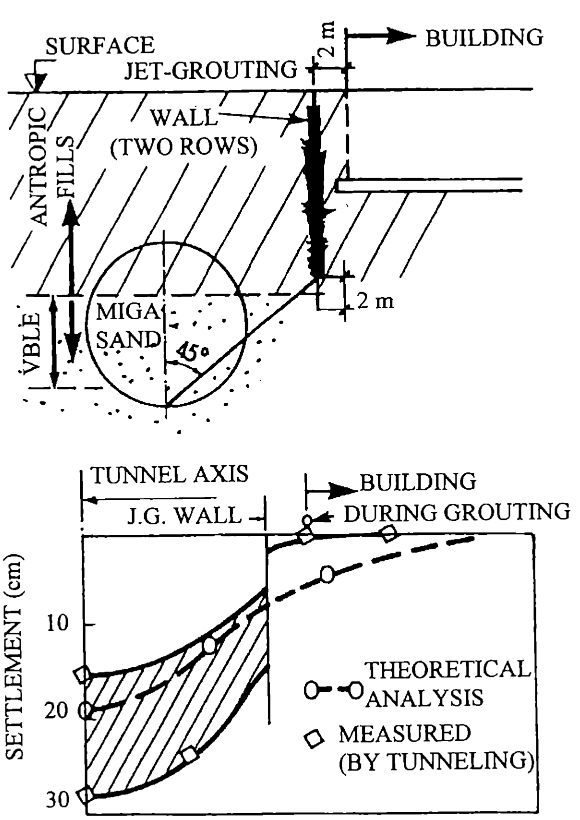


Figure 4. French Institute. Solution and recorded settlement.

- The settlement at one point, the day the tunnel passes below, is in the order of 0-20% of the maximum. Sometimes, slight initial uplifts are recorded when excavating with the EPB system.
- The point of inflection is between 7 and 13 m from the center line depending on the excavation's depth and diameter.
- The surface fill thickness has practically no influence when the overburden in pliocenic ground above the tunnel's crown is 2 diameters or more. If the overburden varies between 0.75 and 2D, the settlement volume varies between 0.15 and 1% of the excavated cross section. For overburdens between 0 and 0.75D, the settlement volume varies between 0.6 and 4%.
- In the case whereby the excavation cross section involves man made fills, problems of tunnel face failure may occur.
- The method described in Figure 2 may be adapted for homogeneous ground with no considerable influence of the fill (HP ≥ 1-1.5D).

The Madrid Model was developed using facts from the three dimensional finit element analysis for studing the shape of the crater of settlement or of Attewel, the Sagaseta and Oteo method (1974) and of empirical corrections based on field measurements of the module of deformation measured in pressiometric and laboratory test. It consists in:

- It is assumed that the typical stratigraphic profile is that in the scheme Figure 2.
- The surface settlements law may be simulated by a Gauss function. The position of the point of inflection, i, and the maximum surface settlement, δ_{max}, althougth the latter may be replaced by the settlement volume, V_s, since: $V_s = 2.5 \cdot \delta_{max} \cdot i$.
- To determine the position of the point of inflection may be used the Figure 2.
- Using actual measurements and theoretical approaches, the use of the empirical law of Figure 3 is recommended (settlement volume as a function of the tunnel's depth and of the relative thickness of hard ground and the possible man made fill above).

Figure 3 shows, also, the application of the Madrid Model recommended, expressed in the form of settlement volume, with various cases measured on Lines 8, 9 and 10 and in the Junction of Lines 8 and 10 of the Madrid Metro.

5 SOLUTIONS USED IN THE GREAT SETTLEMENT CASES

Two main problems can be distinguished to this effect. A: The tunnel runs under a street, but there

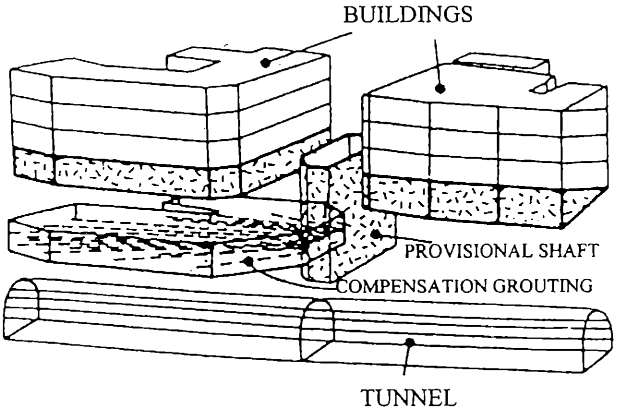


Figure 5. Compensation Grouting lay-out.

are laterally located buildings nearby and, B: The tunnel is under buildings.

An example of the first case corresponds to the Line 4 tunnel, passing close to the Madrid French Institute. The tunnel axis goes to 14-15 m of depth and the antropic fills had a variable thickness (10-18 m). The building were 8-9 m, away from the tunnel axis (Figure 4). Initially it was estimated the value of the possible settlements, obtaining over the 18 cm on the crown and about 6 cm in the building facades, with deformation modulus for the 8 MPa in the antropic soils, estimated from dynamic penetration tests. After this, it was studied the building conditions. This presented some fissures, not too important, due to it is leaned (with a basement) on the fills. For this, it was considered that the facade of the building must not have vertical displacements more than 6 cm or angular distorsions of about 1/150. In these conditions, a protection for the building was designed, on the basis of a screen or wall of jet-grouting (System 1), constituted by 2 rows of columns, separated 70 cm; the distance between column axis of the same line was 90 cm. The average admission in the field was 400 kg of cement by meter of boring. The zone was instrumented to control movements installating references in bluilding pillars and rod extensometers in the ground. In Figure 4 the measured movement and the foreseen ones can be seen. The measured variated between 15 and 30 cm, what is saying that it went over the 19.0 cm value foreseen, due to the ground heterogeny. Although, thanks to the jet-grouting treatment the settlement law was cut and the settlements in the buildings were considerably reduced, measuring in its facade a movement less than 5 mm, instead of a magnitude that could have been between 7-10 cm.

In the second case (tunnel under buildings) if the subsidence analyses show a risk for the nearby structures, compensation grouting treatments are used to disminish the previsible damages.

Vertical shafts are bored near the buildings and tunnel from where fans of horizontal boreholes are drilled (Figure 5), in which pipes with manchettes-tubes (2/m) are fitted, through with successive injection phases may be performed as settling is occurring through the tunnel's excavation operations. The maximum length of the horizontal boreholes was 50 m. A first phase was normally executed (contitioning) until the building started to be raised (≤ 3 mm) and then compensation was made as the tunnel was being excavated. The grouting was prepared in a mixing plant, provided with bentonite and cement storage silos, injection pums, computerized control unit and seals in the mouth of each borehole. Injection pressures varied as a function of the ground, the proximity of buildings, etc. between 7-9 bars for the maximum injection pressure and 80 bars when re-opening a pipe-sleeve.

In the Line 1 Extension -a concrete example- four shafts were made in this work in order to perform the compensation (Figure 6), located according to the length of the boreholes necessary, road trafic, etc. In this example average values for the whole treatment are: a) Area treated: 8,603 m^2; b) Overall borehole length: 5,650 m; c) Conditioning: Grouting volume: 54 l/m^2; d) Compensation: Injection volume: 198 l/m^2 (301 l/m of borehole); and e) Total injection volume: 2,164 m^2.

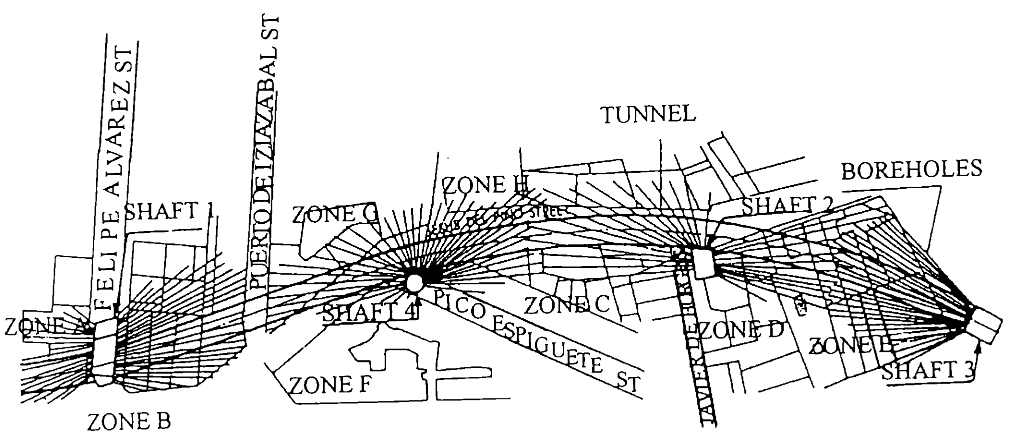


Figure 6. Compensation grouting shafts location

6 CONCLUSIONS

- The methodology used to provind for damage in buildings gave a good result.
- The results obtained with the New Subsidence Madrid Model are very good.
- In the case of buildings located laterally to the tunnel, jet-grouting barriers can give a magnificent result.
- In the case of buildings over the tunnel, compensation groutings allow to reduce to a a minimum the effects of excavations, with reasonable cost and work performance time, although they need major monitoring of movements, volumes injected, etc., while defining the injection process and the area where it has to be performed at all times.

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