CONSTRUCTION OF A URBAN TUNNEL IN LOOSE AND INHOMOGENEOUS TERRAIN UNDER WATER TABLE

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INTRODUCTION

Line 9 of Barcelona's Subway is designed to join the city areas crossed by the two rivers that delimit Barcelona: Besós and Llobregat, connecting the city with the airport placed in the SE. This line is 47.8 km long and it will have 52 stations. It is expected to contribute in an efficient way to improve the transversal transport in Barcelona, by connecting the greater part of the currently existing radial lines.

A relevant characteristic of the Line 9 is the objective of minimizing the volume that the conventional stations occupy transferring a part of it to the line tunnel. In order to reach this objective the line tunnel must house 2 levels for railway traffic, which become platforms in the stations; which requires a diameter of the tunnel excavation of 12 m.

Photo 1 illustrates the layout of the platforms and the vertical shafts that form the stations.


Photo by courtesy of Sánchez-Piulachs Architects.

Photo 1 – Layout of the platforms and the vertical shafts that form the stations of the Line 9

In the vicinity of the Mediterranean Sea, near the airport area, the Line 9 has a branch towards the new area of Barcelona Fair.

This branch starts from the so-called Bifurcation Shaft, which has 32 m diameter and 40 m depth, and the tunnel has a length of 1.129 m, of which 722 have been built between concrete walls and the remaining 407 m with conventional mining methods.

An essential characteristic of this stretch is that it has been built under the water table by excavating in very heterogeneous soils, composed of granular soils and rocks of considerable resistance; which has created numerous problems during the construction of this tunnel which were satisfactorily solved as it can be seen in this report.

SOIL CHARACTERISTICS

The terrain that had to be excavated during the construction of 407 m built with conventional mining methods is characterized by their extreme diversity. In this stretch it was necessary to excavate sandy Quaternary soils, typical for the Llobregat Delta; Pliocene siltstone and claystones and an isolated level of very strong sandstones.

Apart from the great variability of the soils that had to be crossed, it is necessary to point out that 407 m of the tunnel were built under the water table, which created numerous problems to control the water inflow into the excavation, above all in more permeable soils. The detail of the geotechnical profile of this stretch of the tunnel is shown in Figure 1.

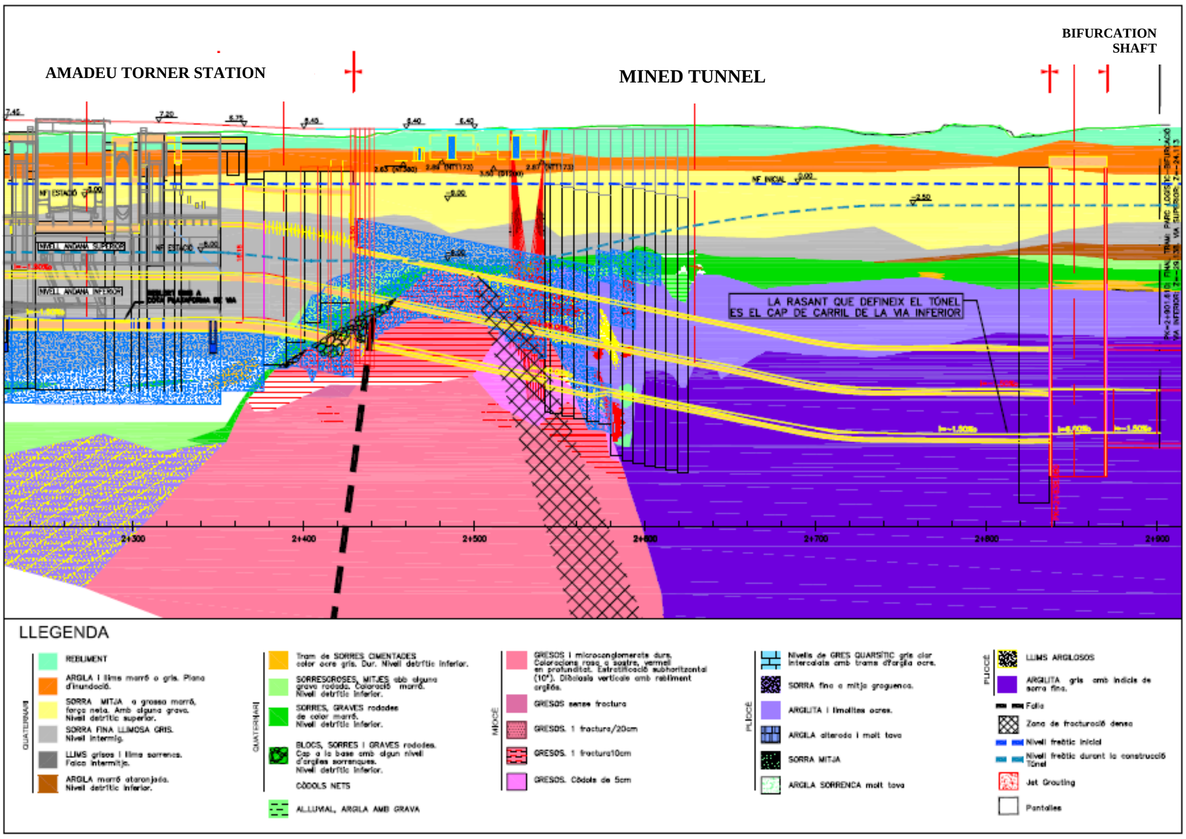


Figure 1 – Geotechnical profile of the stretch built between the Bifurcation Shaft and Amadeu Torner Station

Photos 2 to 4 illustrate three characteristic situations of the behaviour of the soils crossed in the construction of this tunnel.

Thus, Photo 2 illustrates a face in claystones and silts that, being basically impermeable, did not create any problems of stability or water inflow. The situation shown in Photo 2 is rather different as it reflects the effect of the forepole in soil composed of a mix of silts and rock blocks. The situation created when it was necessary to excavate the strong sandstones was also characteristic, as it is illustrated in Photo 4.

Photo 2 - Claystones and silts, impermeable and stable face

Photo 3 - Heterogeneous permeable blocks and silts that produce an unstable face

Photo 4 - Strong and permeable sandstones that produce a stable face

CONCEPTUAL DISEGN OF THE TUNNEL

The greatest difficulty encountered during the construction of this tunnel lay in controlling the water inflow into the excavation. For this reason it was decided to design a tunnel with the least possible width and to plan a constructive process that would guarantee the stability in all the constructive phases.

For that it was decided that the tunnel should have two traffic levels, just as the rest of the Line 9, but with space for only one track in each one of them.

This typology allows to consider the tunnel construction in two basically independent phases; since the concrete slab that permits circulating on the upper level facilitates the closure of the first constructed section, as it is illustrated in Figure 2.

In addition, once the first phase excavation has been closed, a wall of micropiles was foreseen in order to facilitate the excavation of the lower part of the tunnel. With this concept of the tunnel the problems focus on controlling the water inflow through the excavation front in the stretches in which the soil is heterogeneous and, consequently, more permeable.

In order to control the probable water inflow into the front, when more permeable soils were excavated, two actions were anticipated: firstly, lowering of the water table and secondly, reducing the soil permeability through jet-grouting combined with concrete grouting.

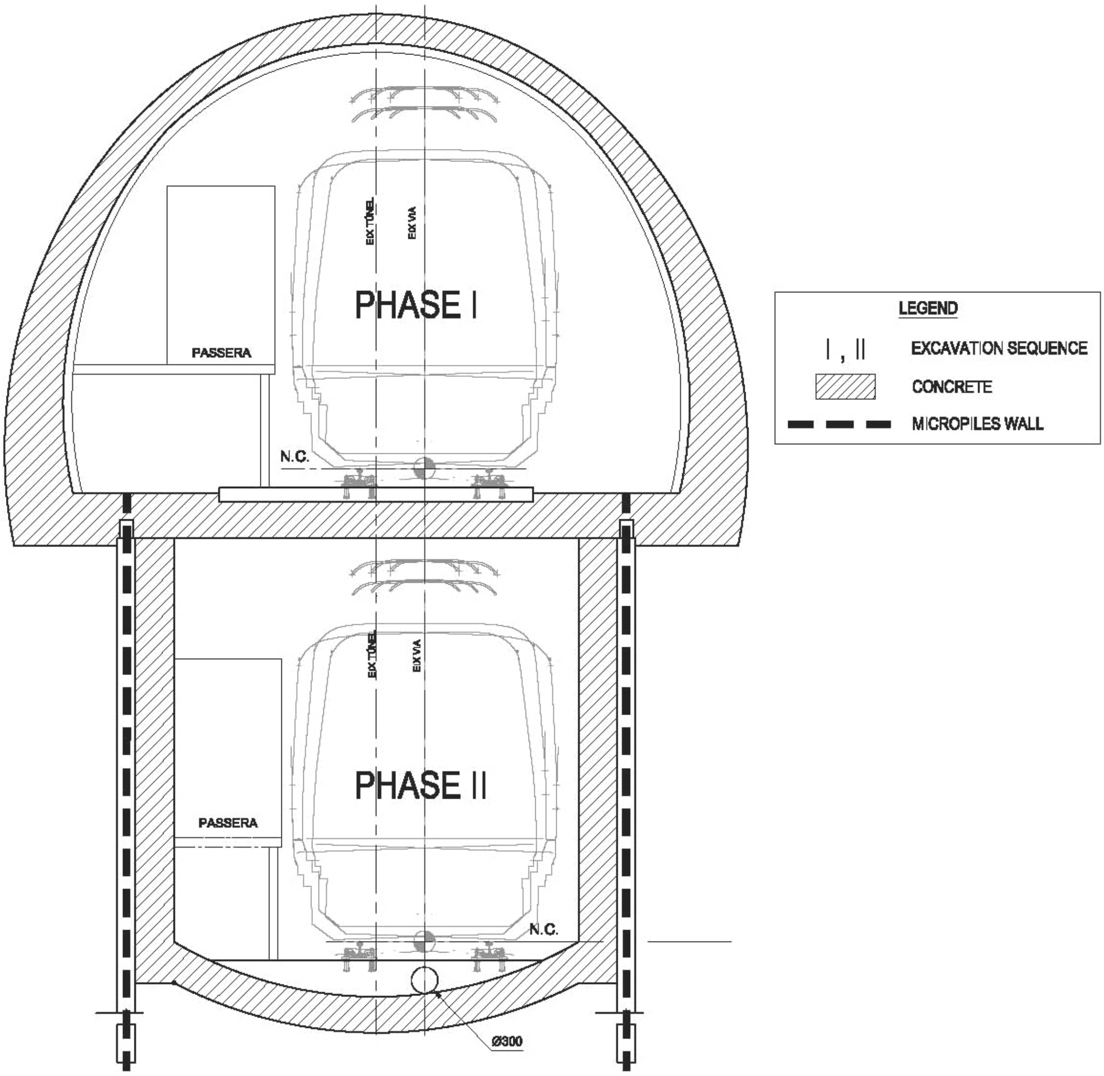


Figure 2- Conceptual design of the tunnel

DIMENSIONING OF THE ADOPTED SOLUTION

The adopted solution was dimensioned using geomechanical models in two and three dimensions. Two-dimensional models were used in the design phase, while three-dimensional ones were utilized during the works in order to adjust the soil properties according to the measurements given by the abundant instrumentation placed at the site.

Figure 3 shows the used two-dimensional model, while Table 1 presents the properties used in the calculi carried out during the design phase.

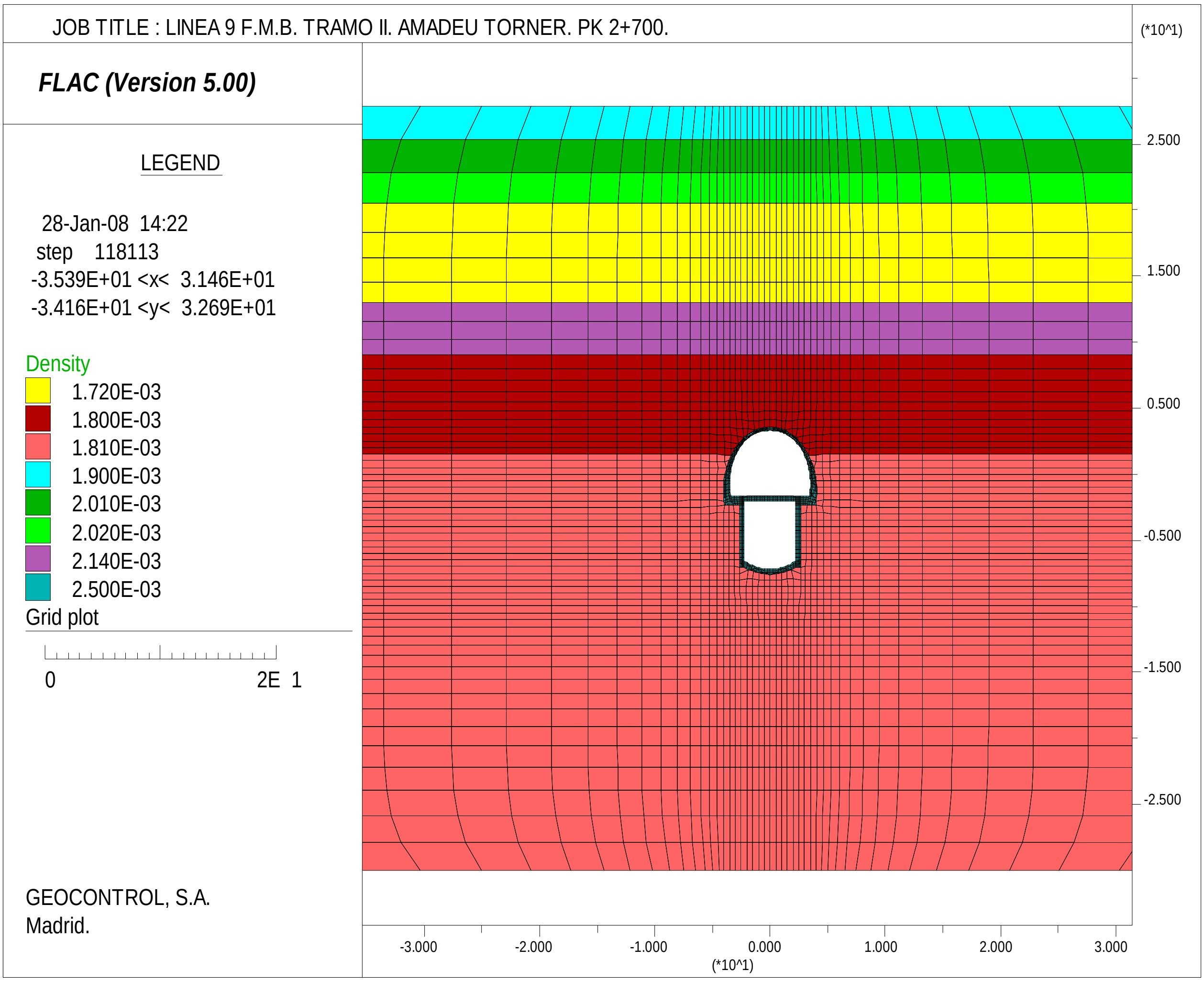


Figure 3 – Two-dimensional model

TERRAINS	d_a (t/m^3)	E (kp/cm^2)	ν	C (kp/cm^2)	ϕ (º)
Q_R	1.90	50	0.35	0.01	28
Ql1	2.01	160	0.35	0.25	29
Ql2	2.02	195	0.32	0.10	34
Ql2b	2.20	500	0.30	0.01	45
Pl1	2.10	375	0.30	0.20	33
Pl2	2.10	500	0.30	0.30	33

Table 1 – Properties of calculus in the design phase

Figure 4 presents the distribution of the displacements calculated with the three-dimensional model once the first phase of the tunnel has been constructed.

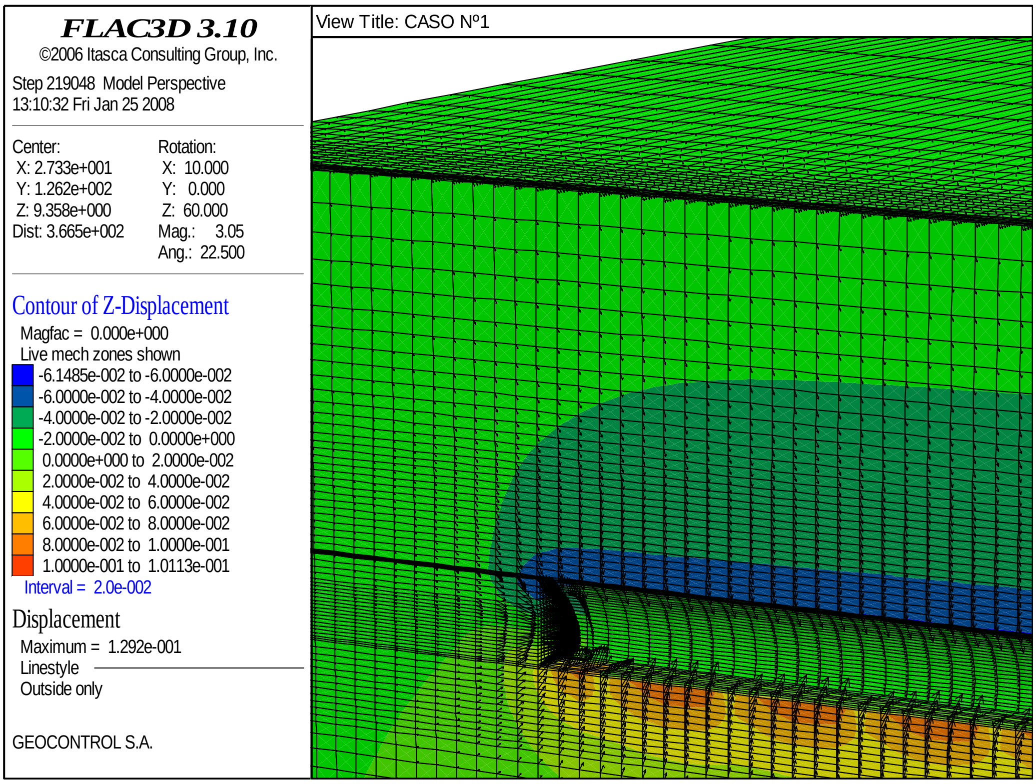


Figure 4 – Distribution of the displacements once the Phase 1 of the excavation has been done

The results of these calculations have been updated in order to adjust the modulus of deformation of the Pliocene claystones (PL1 and PL2) with the purpose of making them match the data provided by the instrumentation.

Figure 5 presents the adjustment of these moduli according to the depth of the soil.

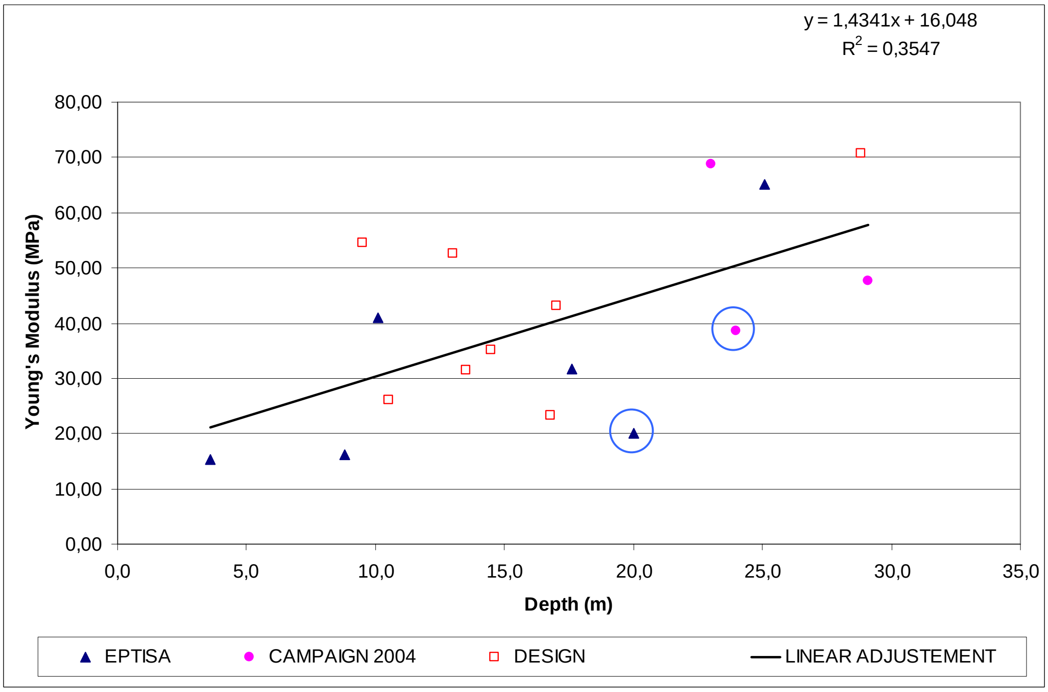


Figure 5 – Adjustment of the modulus of deformation of the soil PL1 with the depth

LONG-TERM CALCULUS OF SUBSIDENCE AND OF POSSIBLE DAMAGE TO BUILDINGS

Once the deformational parameters had been adjusted, in order to make the calculi match the monitoring measurement, we went on to calculate the long-term subsidence and to calculate the possible damage the works might produce on the buildings closest to the tunnel.

Figure 6 shows the distribution of the vertical soil displacements in the final stage of construction and there it can be seen that the maximum subsidence on surface is 45.1 m.

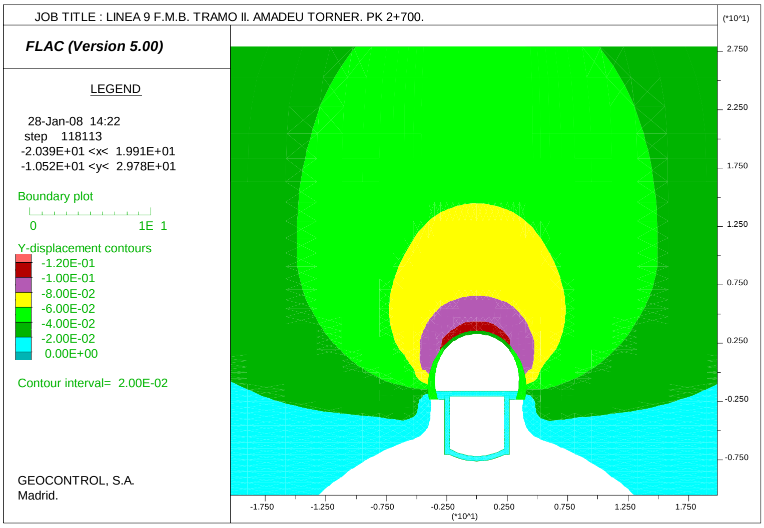


Figure 6 – Long-term distribution of the vertical displacements

Based on the results of the subsidence calculi, which have permitted to define, apart form the maximum value, the transversal subsidence, the potential influence the tunnel construction might have on the nearest buildings was evaluated.

For that reason we used Boscardin and Cording’s criteria (1989), afterwards verified with the Burland’s one (1995), which are based on the calculus of limit strain in tension (ε_{lim}) that permits to classify the damages in the categories indicated in Table 2.

DAMAGE CATEGORY	TYPE OF DAMAGE	ε_{lim} (%)
0	Negligible	< 0.050
1	Very slight	0.050 - 0.075
2	Slight	0.075 - 0.150
3	Moderate	0.150 - 0.30
4 to 5	Severe to very severe	> 0.30

Table 2 – Relation between the damage category and the limit strain in tension (Boscardin and Cording, 1989)

In order to calculate the limit strain in tension the building façade resembles a beam that assumes the vertical settlements in “Greenfield” conditions; without the modelling of the buildings in the calculus.

The maximum horizontal strain in tension is calculated by taking into account the combination of shear and bending ways of deformation to which the horizontal deformation in tension is directly added. The resulting value is compared to the previously established limits in order to determine the level of damages to the building.

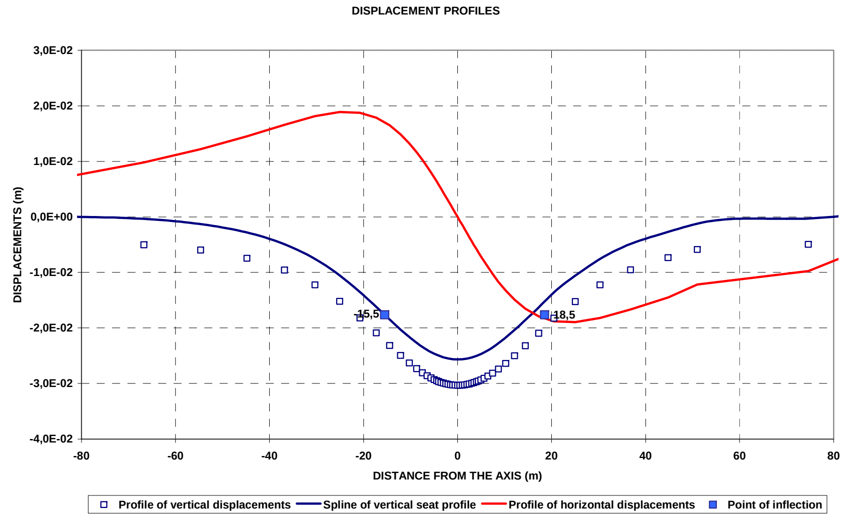
Figure 7 presents the calculus of the evaluation of potential damages to one of the buildings near the tunnel; which are considered negligible.

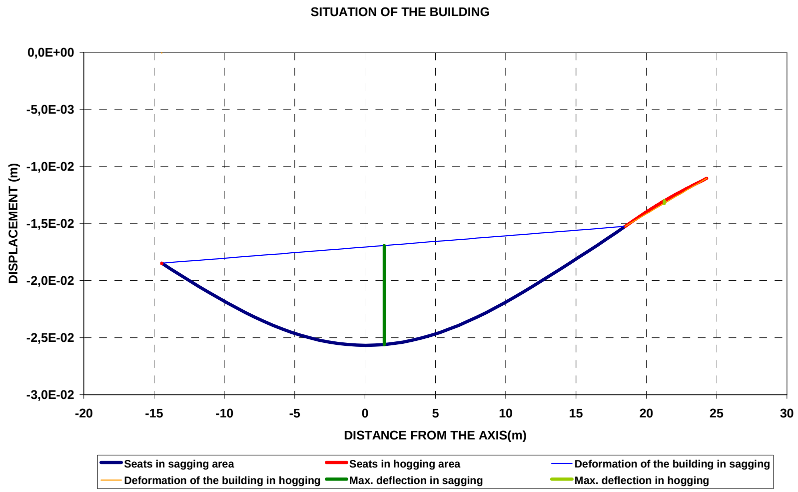
DATA OF THE BUILDING	
Height of the building: H (m):	4,00
Initial distance of the building from the tunnel axis: X1 (m):	-14,45
Final distance of the building from the tunnel axis: X2 (m):	24,30
Distance $X1_{CALCULUS}$ (m):	-14,45
Distance $X2_{CALCULUS}$ (m):	24,30
Distance from the axis of the inflection point: i (m):	18,50
Poisson's coefficient:	0,30
Relation E/G:	2,60

	HOGGING Left area	SAGGING Central area	HOGGING Right area
Maximum relative vertical deflection: Δh (m):		8,7E-03	2,21E-04
Reference length: L (m):	0,00	32,95	5,80
Maximum deflection ratio $\Delta h/L$:		2,6E-04	3,82E-05
Inertia momentum: I (m^4):	21	5	21
Position of neutre fibre: t (m):	4,0	2,0	4,0
Deformación horizontal (terreno): ε_h:		-1,1E-03	-1,1E-04
Deformación unitaria a flexión: ε_b:		1,8E-04	3,8E-05
Deformación unitaria a cortante: ε_d:		1,4E-05	3,4E-05
Deformación unitaria a flexión total: ε_b^*:		1,8E-04	3,8E-05
Deformación unitaria a cortante total: ε_d^*:		1,4E-05	3,4E-05
Deformación máxima: ε_{max} (%):		**0,018**	**0,004**
Damage evaluation:		***negligible***	***negligible***

Values $\boldsymbol{\varepsilon_{max}}$ **(%)** that delimit damage area:

0 - negligibel - 0,05 - very slight - 0,075 - slight - 0,15 - moderate - 0,3 - severe to very severe





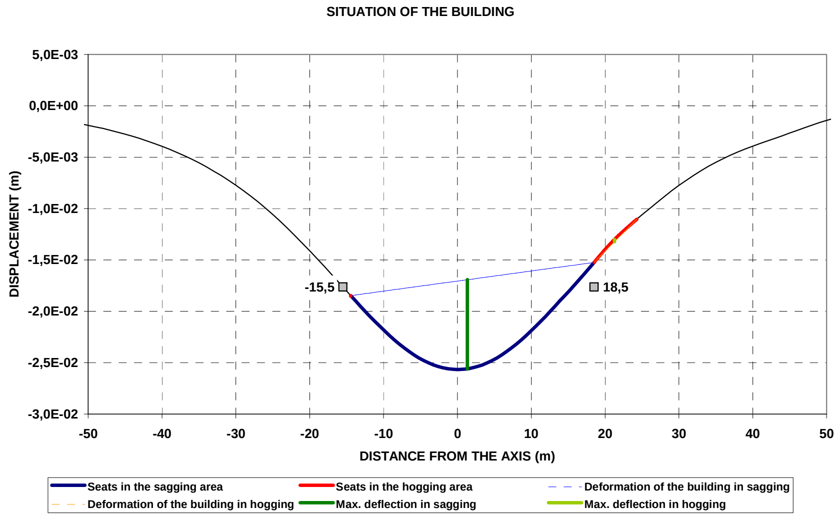


Figure 7 – Calculus of the damages induced to a building

CONSTRUCTION OF THE UPPER PHASE OF THE TUNNEL

The construction of this tunnel has been carried out, in greater part, from the Bifurcation Shaft, as it is indicated in Photo 5.

Photo 5– Beginning of the tunnel construction from the Bifurcation Shaft

The excavation in this phase has been done with mechanical means; in softer soils a backhoe has been used, while in the harder rocks a hydraulic hammer has been utilized.

In more permeable terrains, in order to help stabilizing the excavation face, it was decided to lower the water table. This operation was done by drilling holes for pumping water to the exterior surface and also by drilling horizontal drainage holes at the face of the tunnel, as it is illustrated in Photo 6.

Photo 6 – Drainage from the tunnel face with horizontal holes

In the stretches where the terrain was quite heterogeneous and permeable it was necessary to improve it in front of the excavation front through jet-grouting combined with cement grouting.

Cement grouting was carried out from the outside, through “tube a manchete” technology with pressures up to 0.5 MPa and inflows between 15 and 10 l/min.

A strict control of the volume of grouted cement was carried out in each hole with the purpose of detecting the areas of soil in which more energetic grouting was necessary, which had to be done in various consecutive phases.

Figure 8 shows the distribution of isometric lines of cement intake in one of the grouted stretches. In it the holes in which the intake is evidently greater than in the rest can be clearly observed

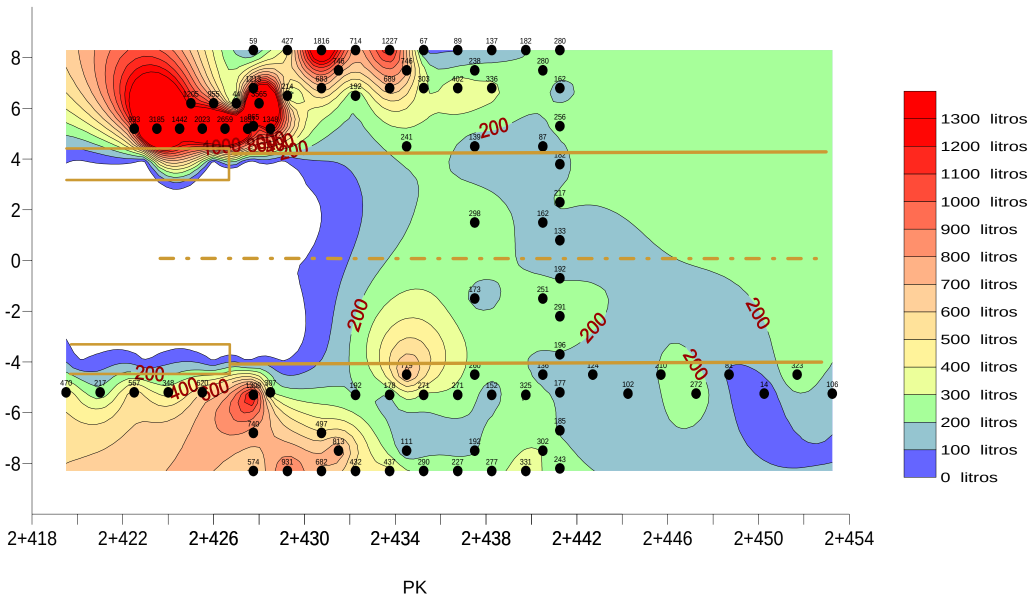


Figure 8 – Example of the isometric lines of grouted cement intake

For the final stage of the construction of the upper part of the tunnel the concrete slab that permits to close the section was built, as it is illustrated in Photo 7, and afterwards the micropiles that should serve to protect the excavation of the lower part of the tunnel were drilled.

Photo 7 – Construction of the intermediate concrete slab that permits to close the upper phase of the tunnel

CONSTRUCTION OF THE LOWER PHASE OF THE TUNNEL

The lower phase of the tunnel is constructed advancing both from the Bifurcation Shaft and Amadeo Torner Station.

The works were developed with no problems owing to good functioning of the structure adopted for the upper phase of the tunnel and to the success of the combined treatments of jet-grouting and reinforcement grouting.

It is necessary to point out especially the good behaviour both micropile walls have had, which have greatly facilitated the excavation of the lower phase in the stretches in rock blocks. Photo 8 shows a detail of the terrain improved by jet-grouting.

Finally, Photo 9 shows the breakthrough between two fronts which have excavated the lower phase of the tunnel. In it the favourable effect provided by the concrete floor built to close the upper part of the tunnel can be appreciated.

Photo 8– Soil improved by jet-grouting over a rock layer

Photo 9 – Detail of the breakthrough of the lower phase

REFERENCES

Boscardin, S.M.D. and Cording, E.J. (1989), "Building Response to Excavation-Induced Settlement", Journal of Geotechnical Engineering, 115, pp. 1-21.

Burland, J.B. (1995), "Assessment of risk of damage to buildings due to tunnelling and excavation. Invited Special Lecture", In: *1st Int. Conf. on Earthquake Geotech. Engineering*, IS Tokyo.

Burland, J.B. Standing, J.R. and Jardine, F.M. (1995), "Assessing the risk of building damage due to tunnelling – lessons from the Jubilee Line Extension", *Geotechnical Engineering. Meeting society's needs*, Vol. 1, pp. 17-44.