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A new tunnel bored in alluvial soft soil under the Malaga Airport

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ABSTRACT: Due to the Extension of Malaga Airport, it became necessary to route the Malaga-Fuengirola railway line underground, creating two subterranean stations and more than 2 Km of tunnel. Although the tunnel end could be resolved using continuous slurry trench walls, the presence of the River Guadalhorce has led to the problem being resolved by executing the tunnel using a tunnel boring machine of ∅ 9.40 m to excavate through the sediments of that river (soft clay, loose sand, etc., with a very high water table). The article describes the works, with the most important landmarks (zones with scarce coverage, pass under the existing railway, emergency shaft, etc.), as well as the treatment of the land that it was necessary to perform. It also describes the auscultation carried out and comparison of its results with the forecasts of movements previously performed, with a code of finite elements.

1 ANTECEDENTS

The Malaga-Fuengirola railway line is a sole electrified track, with double track in some of the stations. There are two fundamental aspects that make action on the line necessary: at the time, a study was carried out of the need to double the line due to increased demand, and to increase the facilities at Malaga Airport. At the same time, the extension of the airport runways made it necessary to route line underground, with it passing under the River Guadalhorce, near that Airport. Those underground routing works consisted of execution of a corridor for double track with a total length of approximately 4,000 metres, from the exit of Los Prados station, at Km. 4 + 355 of the present track, to the exit of the present Airport-Traveller Terminal station (Km. 8 + 347). The feature of that corridor is that it runs underground for practically the whole route.

These underground routing works have been co-financed by AENA and the present Directorate General of Railway Infrastructures of The Ministry of Development.

2 DESCRIPTION OF THE SOLUTION PREPARED

The stretch forming the object of this article includes the following functional elements (Fig. 1):

- Guadalhorce Station (Km. 0 + 735 – Km. 0 + 931).
- Airport Station (Km. 2 + 870 – Km. 3 + 110).
- Initial Pumping Shaft (Km. 0 + 420).
- Emergency Exit, Ventilation Shaft and Pumping Shaft (Km. 1 + 892).

The stretch forming the object of the project had four construction types applied to the underground routing of the railway line, that were:

- a) Section with embankment and/or open cast excavation.
- b) Open cast section between conventional walls.
- c) Open cast section between slurry trench walls with an upper and lower slab (width between walls of 9.40 to 10.34 m).
- d) Tunnel excavated by tunnel boring machine (E.P.B. type, with an exterior diameter of 9.40 m; stretch length: 1,971.55 m).
Figure 1 shows the schematic ground plan of the works with the most outstanding elements.

3 GEOLOGY AND GEOTECHNIQUE

The airport zone and the railway route of the Malaga-Fuengirola line affected by the Project are located in the lower part of the River Guadalhorce Basin, in the municipal district of Malaga. The route is fully located in the fluvial facies and interior estuary of the river Guadalhorce. These materials are flanked by the hills of Malaga, that constitute the main source areas of the materials. The sedimentary basin of the lower Guadalhorce is enclosed by Pliocene materials, their edges and the border faults of the mountain ranges. As these reliefs are relatively near and as a consequence of the isostatic and climatic readjustment movements, the fluvial deposits associated with the river Guadalhorce have quite considerable thicknesses, with the detritic facies of the banks characterized by a gradual transition from coarse to fine materials, at the edge of the basin.

As stated in the Project, the geological site conditions were strongly affected by the presence of the sea; due to the interaction with the sea, continuous material readjustments occurred at the site of the Project. This fact, along with the fluvial dynamic, meant that the co-relation between sedimentary deposits is highly complex and the resulting morphologies have scarce lateral continuity. The alluvial soil of the Guadalhorce is highly heterogeneous detritic sequences of gravels, sands, silts and clay, with lens of organic silts and grey clay associated with the early mouth of the estuary and the abandonment of old canals. That is to say, there are alternating coarse and fine materials, with loose density and soft consistence, respectively.

The planned route runs from its start to Km. 2 + 300 somewhat perpendicular to the present course of the river, then gradually turning to run parallel to it, from Km. 2 + 700 to the end. There is a great variation in horizons on the alluvial plains, the following having been distinguished: a) Anthropic fills (R). b) Quaternary fine sediments (clay and muds, QM). c) Fine to coarse sand with pebbles (Qs). d) Quaternary gravel (QG). e) Quaternary grey and greenish clay (Qc). f) Pliocene clay (PC). g) Partially cemented gravel (PGC). h) Pliocene brown marly clay (PCM), the near stratum. The last three horizons are Pliocene and, thus, more competent. Table 1 summarises the main geotechnical properties of that ground assumed for the analytical analysis deduced from laboratory tests, S.P.T. an pressiometric tests and the engineering personal experience in the zone and Figure 2 shows a schematic geotechnical section through the tunnel development.

4 TUNNEL BETWEEN SCREEN WALLS

The underground stretch, as has already been stated, has a length of 3925 m. The solution adopted alternates between use of the tunnel boring machine between Km. 0 + 930 and Km. 2 + 800, continuous slurry trench walls, both on corbel and in situ top slab, and the corbel walls.

The solution adopted in this case is that of continuous reinforced concrete walls, either corbelled or strutted horizontally by top slabs and invert.

These continuous diaphragms have thicknesses of 1.00 m and 1.20 m, with embedding between 12 and 18 m under the excavation level, due to the low consistency of the ground. Their calculation was performed using the RIDO french numerical program. One must state that, with regard to
Table 1. Summary of geotechnical properties for diaphragm walls analysis.

<table>
<thead>
<tr>
<th>Ground</th>
<th>Units</th>
<th>Dry bulk density (t/m³)</th>
<th>Water content (%)</th>
<th>% Fine content (**)</th>
<th>Cohesion (t/m²)</th>
<th>Friction angle (º)</th>
<th>Horizontal reaction modulus (t/m³)</th>
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<td>27</td>
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<tr>
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<td>27</td>
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<tr>
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<td>30</td>
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<td>33</td>
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<td>Ps</td>
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<tr>
<td>Gravel</td>
<td>QG</td>
<td>1.91</td>
<td>6</td>
<td>11</td>
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<td>36</td>
<td>1800 (1800)</td>
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<td>12</td>
<td>VAR</td>
<td>0.0</td>
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<td>10</td>
<td>40</td>
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</tbody>
</table>

(*) Final parameters
(**) % through nº 200 sieve ASTM

Figure 2. Schematic geotechnical profile of the area.
the definition of the invert, (1.0 to 1.20 m thick), according to the span and depth, with a V shape.

5 GUADALHORCE STATION

The Guadalhorce station and the entrance shaft used for the tunnel boring machine (adjoining the previous one) have a length of 197 m and are located between Km. 0 + 733 and Km. 0 + 930, placing the tunnel boring entrance shaft at the end, between Km. 0 + 835 and Km. 0 + 930. The shaft was formed using continuous slurry trench walls of reinforced concrete, (thicknesses of 1.00 m and 1.20 m). The diaphragm walls were strutted horizontally using reinforced concrete slabs and crossbeams.

The level of intermediate strutting of the tunnel boring machine cutting face shaft is 7.50 m above the floor level. The crossbeams, with a section of $2.2 \times 2.0$ m, were planned in reinforced concrete (5 intermediate side walls), defining four intermediate openings and two ends (Figure 3).

During the planned excavation of the entrance shaft for the tunnel boring machine, the installed instrumentation detected excessive displacements in the walls (nearly 90 mm horizontal displacement) with settlements of about 50 mm in a building near to the works (Fig. 3).

New tests (dynamic penetrometers) allowed a new ground profile to be defined, with a greater level of fillings (3 m more thickness). The project had already been calculated with some short term shear strength parameters (initial parameters of Table 1) that gave less thrust and deformations than those that had been measured at the field (Fig. 3). Recalculation with the another’s geotechnical parameters (Table 1 final parameters defined by local experience deduced from actual works at the Málaga Underground) allowed better reproduction of what had happened (Fig. 3).

Based on the experience obtained from excavation of the start shaft for the tunnel boring machine, in order to apply it to excavation of the line tunnel and with new stretch data provided by the excavations carried out, a new process of the line tunnel construction was proposed in the initial zone. The aim of this was to limit the excessive movements of the walls.

Following the previous study, it was recommended to adopt a new excavation construction process for the continuous walls by using provisional metallic shoring (HEB-300/2.5 m profiles) located at 4.00 m above the maximum excavation, to limit and control the screen deformations.

Within the line tunnel in the initial zone, special attention was paid to Km. 0 + 500, due to the nearness of several existing silos to the tunnel. After carrying out the relevant study, in addition to provisional metallic shoring of the line tunnel,
it was recommended to provide props at the foot of the diaphragm by performing Jet-grouting and mortar set screens under the excavation (Figure 4).

The results obtained were satisfactory and the silo settlement less than 3 mm.

6 TUNNEL WITH T.B.M

6.1 General aspects

As already mentioned, most of the tunnel was excavated using an E.P.B. tunnel boring machine of Ø 9.40 m, with reinforced concrete segments 32 cm thick. The excavation was performed fully in soft clay with sand and loose quaternary gravel, sometimes with scarce top cover. Mainly for that reason apart (from passing under the actual railway to be replaced) diverse ground treatments were designed, in order to assure the stability of the works (and reduce the surface movements, as appropriate).

The following is a brief list and description of the most important treatments performed (not including the “start” shaft of the E.P.B. and other specific cases).

6.2 Initial stretch and Crossing of the Azucarera-Intelhorce road (T-3 and T-4)

Due to the quantity of existing utilities affected in the area where the tunnel boring machine crossed the Azucarera-Intelhorce road (as soon as the tunnel with the E.P.B. commenced), a pile founded concrete slab 1 m thick was laid, on which to fasten the utilities affected and thus prevent the excavation becoming unstable, given the scarce existing top cover (3 m). In order to improve the ground conditions after the tunnel boring machine passed through, injections were executed every 4.5 m to fill the gaps under that slab. The gantry crane was installed on that slab to stock and supply segments (Fig. 5a).

6.3 Stretch between Km. 1 + 202 and Km. 1 + 290 (T-6)

On this stretch, there is a level of quaternary gravel with the groundwater table (GWT) at the tunnel crown level. The fine content of this material is very low, #0.080, lower than 20%. Due to the lack of fine particles in this material at the tunnel crown, it was foreseen that chimneys might possibly form
due to instability of the crown, as happened on one occasion. In order to avoid or limit formation of such chimneys, it was decided to treat the gravel in the tunnel crown zone by columns of jet—grouting laid out in a Canadian tent shape (Fig. 6b). It was also decided to eliminate possible water flows by forming sealed enclosures made of bentonite—cement walls. In order to avoid specific instability and these being dragged, a transversal screen of mortar piles was made at the centre of each one of those enclosures, and after the tunnel boring machine passed through, a draining ditch was dug in each one of those enclosures.

6.4 **Inspection tasks (T-7)**

In order to inspect and review the machinery and repair it when necessary, it was decided to establish inspection shafts, every 300 metres, also taking advantage of the emergency exit at Km. 1 + 870 as an inspection shaft (7 inspection shafts). These inspection shafts (that are 15 m wide and 4 m long), except for that located at Km 1 + 100, are made as a sealed enclosure of weak block concrete inside by mortar piles (3 rows of $\varnothing$ 850 mm). In some cases (Km. 1 + 100) the piles were of the secant type, given the presence of sand and gravel.

6.5 **Emergency exit (T-8)**

Two sealed enclosures were made in this zone using weak concrete slurry trench walls with a thickness of 1.20 m in order to be able to eliminate the thrust of the water to be extracted by pumps from the actual sealed enclosure and facilitate construction of the connections between the tunnel and exit shaft. Moreover, in order to join the tunnel emergency exit, prior to its construction, a treatment was carried out from the surface based on micro-cement injections, limited in turn with a jet grouting screen. Finally, in order to facilitate excavation of the exit shaft and improve the behaviour of the screens, a jet-grouting plug was made under the invert level if the enclosure could not be exhausted (finally, that background treatment was necessary, Figure 6).

6.6 **Passage under the railway (T-9)**

The crossing with the existing railway on the surface takes place toward halfway through the tunnel, where the crown of the tunnel is at a depth of 20 metres. The ground is formed by quaternary sand with punctual alternating gravel, with the water table located about 3–4 metres from the surface. In order to avoid subsidence problems in the train track due to the tunnel boring machine passing under it, the decision was made to from a “Canadian tent” using jet-grouting columns, reinforcing the railway side with a double jet grouting line (similar to T-6 treatment, Fig. 5b).

6.7 **Arrival at the airport station (T-10 and T-11)**

Preloading of the future airport runways was to be carried out in this zone, that runs from Km. 2 + 040 to 2 + 885. The height of the preloading would be double that of the final filling. The tunnel was not built until all the consolidation subsidence had taken place, to avoid excessive deformation to the tunnel once it was built. (Fig. 7, treatment T-10).

Apart from that issue, the natural ground cover above the crown is scarce, sometimes even less than a half diameter. In order to avoid stability problems, all the preloading was left to treatment T-10, until the tunnel had been built, not being withdrawn until afterward, so that, in order to accelerate that consolidation, wick drains were placed from km. 2 + 040 to 2 + 680. The total lining above the tunnel at the moment of it being built, natural ground plus filling with preloading, was at least 1 diameter. In the area where the cover was less, the
preloading was increased until it reached that figure (from Km 2 + 680 to 2 + 885), being performed with the same material and degree of compacting as the rest of the filling, slope 3H/2V and a crown width of 30 metres.

In some zones, the natural ground was reinforced with a layer 4 metres thick of ground stabilised with cement, (Fig. 8, treatment T-11).

7 RESULTS OBTAINED

7.1 Instrumentation

Diverse types of instrumentation has been installed along the works:

a) Into the slurry trench walls: Inclinometers.
b) On tunnel segments: Cells to measure the total pressure on the lining and stresses originated on the tunnel segments. c) In the ground: Instrumentation to measure the settlement and horizontal movements, that might affect near infrastructures (buildings, utilities, etc.).

Inclinometers, total pressure cells, vibrating cord extensometers, levelling markers, rod extensometers, piezometers, electro-levels, etc., were used for that purpose.

The most conflictive points of the stretch, as already remarked in point 6, that were performed by tunnel boring machine, were studied using the PLAXIS two-dimensional finite elements code, in order to estimate: a) Maximum axial thrust on the segments. b) Maximum bending moment on the segments. c) Vertical and horizontal movements in the ground. d) Thrusts on the slab and supporting piles (initial stretch).

7.2 Results obtained in different points from the tunnel

7.2.1 Initial zone and Azucarera-Intelhorce road crossing (T-3 and T-4)

Given the high number of interferences and due to the movements calculated in the project for this zone being very high (settlement at the crown estimated at 128 mm.), a finite element model was prepared in PLAXIS to estimate the settlements, taking the treatment to be performed into account. The calculation model took (drained stress hardening soil model, with the unload modulus 2.5 times the load modulus, with K₀ = 0.6, the profile of Fig. 2 and the shear strength properties from Table 1) into account the two hypotheses that could be raised according to the value assigned to the stress relaxation of the ground coefficient (α) in this soft soil, applying α values of 0.2 and 0.3. The maximum values estimated were about 25% of the design one, about 10–25 mm above the crown; the results measured after excavation of the tunnel (measured immediate settlement) perfectly fitted the models and, more specifically, that obtained using the highest ground relaxation value, that means that the soft ground has an immediate stress relaxation of about 30% (Fig. 9).

7.2.2 Zone between Km. 1 + 202 and Km. 1 + 290 (T-6)

Due to the lack of fine materials and the heterogeneity of the gravel in this area, the results obtained were according to the time, there being immediate settlement according to the PLAXIS

![Figure 7. Treatment under the airport pavement.](image)

![Figure 8. Treatments under runways.](image)

![Figure 9. Comparison between theoretical analysis and the measured settlement induced by tunnel excavation (T-3).](image)
model calculating that by greater release of those tensions or ground accommodation, that settlement would gradually increase, between 30 and 40%, observing that they stabilise after 28–35 days after the excavation, always taking into account that they never reach even 50% of the design settlement (about 10 cm, Fig. 10).

7.2.3 Crossing under the railway (T-9)
The crossing under the existing railway was a delicate point, due to trains continually passing overhead, but the settlement caused in the railway track was about 4–6 mm, being lower than the model estimates. The railway platform was also monitored, there being somewhat greater settlement, of about 20 mm (Fig. 11). The different measured procedure to justify, in our opinion, these discrepancies.

7.2.4 Arrival at airport station (T-10 and T-11)
The last models studied were highly difficult to define, due to the variability of the soil conditions in the cross section that might exist due to there being different thicknesses in each level of the treatments. The results of calculation sections chosen (Figs. 12 and 13) are similar to the results obtained and, in both cases, they were about 15–25 mm.

7.2.5 Summary of the obtained results
In the initial project, evaluation was carried out of the maximum subsidence originated by the tunnel boring machine, by the Sagaseta & Oteo Method (1974 and Oteo and Sageseta, 1982), using average bulk deformation modulus, (Fig. 14): interval of 1 to 3, without taking treatments into account. The cover above the tunnel crown is also shown in this figure. Settlement between 15 and 45 cm was foreseen, without a clear degree of certainty. However, it was possible to reproduce the ground treatment in each case using the PLAXIS Code, obtaining settlement figures from 10 to 40 mm, fairly similar to those really measured. In zones without treatments, settlement reached 20 cm.

As may be appreciated from observation of Figure 14, the degree of approximation was fairly satisfactory.

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