Geodetic network design and strategies followed for drilling a 25 km tunnel for high speed railway in Spain

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Key words: TBM, tunnel breakthrough, underground network, surface network, accuracy, gyrotheodolite azimuth

SUMMARY

During the last years several large high-speed railway tunnels have been built in Spain. To solve these projects, technical and scientific problems have been solved from the geodetic and surveying point of view. These studies have allowed us to set a methodology that optimizes the performance of this kind of works in the world of Civil Engineering.

We have applied our studies to the Tunnels of Pajares that are the second longest ones in Spain with a total longitude of about 25 km. The studies summarize the design of the geodetic networks to support the guidance of the TBMs used as well as the election of the observations to be done, the instrumental to be used and the observation and computation procedures to be followed.

A special emphasis has been taken into account for the treatment of the uncertainty of the coordinates, displacements and breakthrough obtained during the drilling tasks. The article shows the results obtained and the conclusions that can be followed in order to successfully complete a similar project.

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1. INTRODUCTION

During the last years, 25 km long high-speed railway tunnels (Tunnels of Guadarrama and Tunnels of Pajares) have been built in Spain, being currently the 4th and 7th longest tunnels in the world. Technical and scientific problems that we have had to solve in these projects, in geodetic and surveying fields, have allowed us to set a methodology that optimizes the performance of this kind of works in the world of Civil Engineering.

Pajares tunnels are part of the so called Pajares bypass and they belong to the new high-speed railway line leading to Asturias from the Castilian plateau and through the Cantabrian Mountain Rift.

Pajares base tunnels, of about 25 km length, consist of two parallel tubes followed by two 40 meters long parallel viaducts. The distance between the tubes of the tunnels is about 50 meters with cross-passages every 400 meters, see figure 1.

The boring of these tunnels has been made with five Tunnel Boring Machine (TBMs). Two of them started from the South end (Pola de Gordón) boring with north direction; another one started from Buiza, located in an intermediate zone of the project, boring a 5.5 km gallery. The last two ones connecting the North end with the South end from Telledo. Four of the five TBMs used were single shielded and one double shielded.

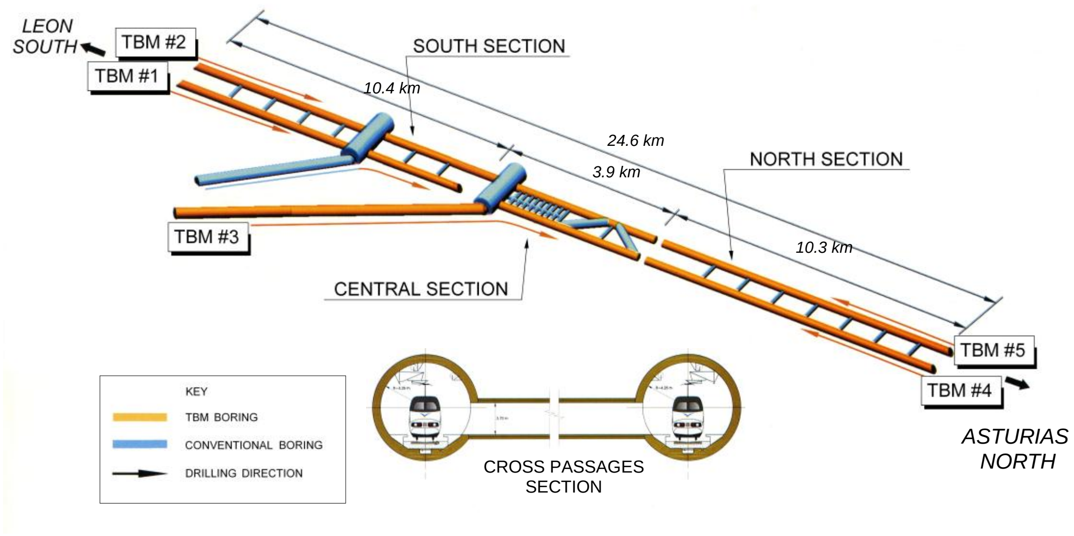

Figure 1.-PajaresTtunnels project overview (modified from adif.es)

The main aim of our work has been the elaboration of a design, methodology, calculation and compensation of surface geodetic control network and underground geodetic networks, serving for guiding TBMs and the correct drill inside the tunnel. In addition, these networks serve also for the rest of the geodetic works that are necessarily performed inside the tunnels (convergences, rail layout, etc). The tolerance demanded in the set of technical specifications for the breakthrough errors in the perpendicular plane and cross were 0.2 metres. When compiling this article, some of the drills in the foreheads have already been done. The definitive calculation of the rest of the drills is scheduled for the next months.

2. PREVIOUS STUDIES

From a geodetic point of view some questions must be solved when executing those tunnels:

- Selection of the geodetic reference system.
- Design, observation, calculation and compensation of surface networks.
- Design, observation, calculation and compensation of underground networks.

The choice of the geodetic reference system (GRS) in which the tunnel has to be built is called "zero order design" (Grafarend, 1974), but in general, the above mentioned system is determined by the GRS in which the construction project is based, usually the official one of the country. But it can happen, such us the case of the Eurotunnel, that due to particular characteristics, it is necessary to define a new GRS, here known as CTG86 (Radcliffe, 1989). In Gotthard tunnels two main solutions were analyzed, LV95 and LV03 (Haag et al., 1997), (Schneider et al., 1997).

At the present time in Spain, the GRS used is ED50 (from 2015 the system will change to ETRS89). In Spain, the coordinate system currently used is UTM. But it can happen, like in the case of GRS, which a particular coordinate system must be defined; mentioned example for the Eurotunnel, the so called TransManche 87 (RTM87) system was developed.

In general, tunnels longer than 5 km are usually approached from multiple portals. In the case of the Tunnels of Guadarrama (Arranz, 2006) two portals where used. Tunnels of Gotthard (Braker, 1997) where built with five portals. In every portal a geodetic network must be built to support the underground network. Surface geodetic networks must be computed and adjusted together. Observations are being performed by GNSS procedures, which are less laborious, more accurate and profitable than classical technologies (Schödlbauer, 1997). For surface networks design the following steps must be taken (Grafarend, 1974): design of first, second and third order.

For the optimization of tunnels boring the effort focused on the design and observation of underground networks, which normally consist of *zigzag* traverses inside the tunnels (Chrzanoswki, 1981) or by means of the utilization of a new type of shoots known as "spigots" (Ryf et al., 2000).

The greatest source of errors inside the tunnels is due to the lateral refraction. As a consequence of the experience in the construction of the Channel Tunnel (Johnston, 1998), (Koritkke, 1990), the results confirmed that a traverse along the axis is the smartest way to reduce the above mentioned effect. Also new technologies developed to minimize the effect of lateral refraction have been detailed (Ingesand, 2008), (Bockem et al., 2000).

Following conclusions have been taken using the tests performed by different observation methods in the access gallery to the central well of Gotthard tunnel: always avoid sights closer than 1.5 meters from the tunnel walls and the use of a gyrotheodolite. The gyrotheodolite avoid the lateral refraction errors and checks the traverse angular transmission errors.

The studies of Lewen (2006), Brunner and Grillmayer (2002) hardly describe the gyrotheodolite and its applications in tunnel control networks. But the question is: how many axes of gyrotheodolite and how many observations to minimize errors must be done? In Charznowski (1981), Martusewicz (1993), Jaroz and Baran (1999) the final conclusion is to observe approximately a gyro-azimuth every kilometre, doing cross observations to minimize lateral refraction effect.

For the design of the traverses, as we have commented previously, the most suitable are those that go along the tunnel axis. But the problem is that the tunnel axis is usually occupied by transportation infrastructures and services inside the tunnel. For this reason, this design is only useful to make control measures during technical stops in the works. In other case, a zigzag traverse must cross from gable-wall to gable-wall in order to avoid lateral refraction. This design permits machinery movements along the tunnel with minimal affection to traverses (Velasco, 2007).

3. METHODOLOGY AND RESULTS

The Tunnels of Pajares have been bored from three tunnel portals. Therefore, three geodetic surface networks were designed, one per portal, each of them formed by four survey points linked to each other and observed by means of GNSS technologies (figure 2). Initially the GNSS observation procedure was designed following certain algorithms (Snay, 1986), (Unguendoli, 1990) but due to the difficulty of access to a series of survey points, the observation was finally designed in the following the way:

Phase a. - Observation from four points of the National Geodetic Network to two survey monuments of each surface network.
Phase b. - Observation from four markers of the National Geodetic Network to the all survey points of each surface network.
Phase c. – Observation of six survey points (two of each surface network).
Phase d. – Observation of the remaining six survey points (two of each surface network).
Phase e. - Observation from five points of the National Geodetic Network to each of the main survey monuments of each surface network.

In each phase, the lengths of baselines remain homogeneous in order to facilitate the correct selection of weight when the adjustment was computed. This phased observation was done to optimize the reliability and redundancy of the network with observation time and logistic operations. This procedure shows an overdesigned scheme which was successfully used on other tunnel networks like Guadarrama, Abdalajis, La Cabrera or San Pedro (Velasco, 2007).

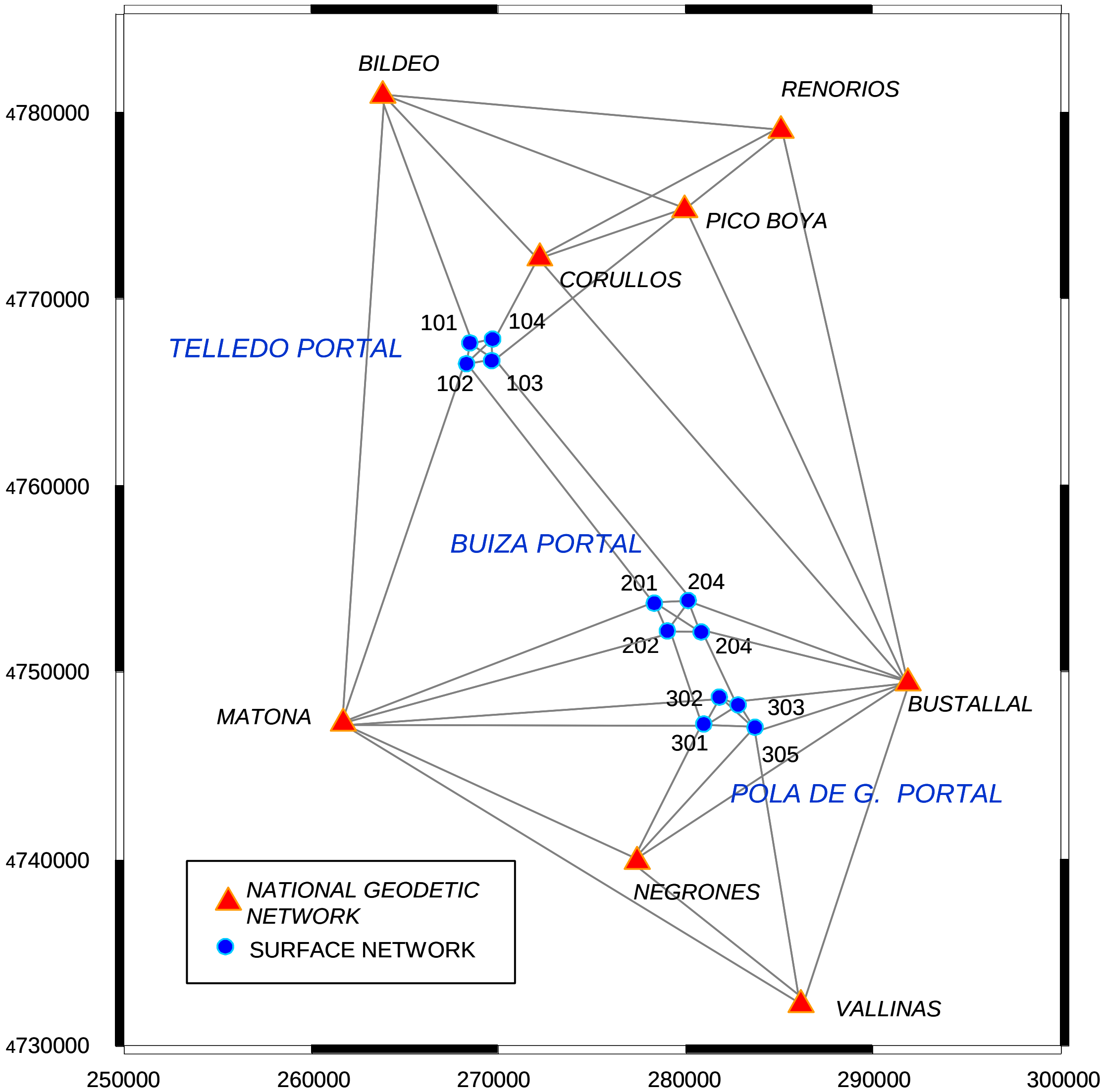

Figure 2.-Schematic view of the geodetic networks

Six ASHTECH UZ-12 double frequency receivers, provided with geodetic antennas, were used following the static method procedure (Hofmman-Wellenhoff, 2001). Each session took between 3 and 5 hours. The computations were performed using Leica SKI-PRO with IGS precise ephemeris. Finally, the surface network adjustment was done using SKI-PRO and GEOLAB package.

Using computed baselines, once the closing errors where analyzed and the outliers where rejected, network adjustment was performed, by means of GEOLAB package. As a result, the error ellipses to 95 % of confidence level are below 10 mm, as shown on Table 1.

Control Points	Latitude σ (m)	Longitude σ (m)	Altura Elips.(m) σ (m)
101	N 43 1 30.49577 0.003	W 5 50 24.49944 0.003	690.206 0.003
102	N 43 1 9.82545 0.003	W 5 50 28.18001 0.003	829.595 0.003
103	N 43 1 17.17141 0.003	W 5 49 49.61426 0.003	848.000 0.003
104	N 43 1 33.68953 0.003	W 5 49 41.60397 0.003	905.063 0.003
201	N 42 54 8.79002 0.003	W 5 42 54.99967 0.003	1471.146 0.003
202	N 42 53 22.93267 0.003	W 5 42 16.39381 0.003	1457.319 0.003
203	N 42 53 23.37593 0.002	W 5 40 57.29589 0.002	1226.028 0.002
204	N 42 54 16.80775 0.003	W 5 41 30.45889 0.003	1460.740 0.003
301	N 42 50 42.47085 0.003	W 5 40 47.43989 0.003	1224.153 0.003
302	N 42 51 27.93574 0.003	W 5 40 10.72729 0.003	1148.030 0.003
303	N 42 51 22.48001 0.003	W 5 39 39.91327 0.003	1227.335 0.003
305	N 42 50 40.50697 0.003	W 5 38 44.98602 0.003	1132.160 0.003
Bildeo	N 43 8 27.68414	W 5 54 14.39435	1298.382
Bustallal	N 42 52 2.77576	W 5 32 51.31698	1432.779
Corullos	N 43 3 56.10098 0.002	W 5 47 48.78124 0.002	1373.905 0.002
Matona	N 42 50 24.74635	W 5 54 54.84868	1571.433
Negrones	N 42 46 38.74467 0.003	W 5 43 9.25217 0.003	1373.471 0.003
Pico Boya	N 43 5 29.10004 0.002	W 5 42 8.80730 0.002	1785.287 0.002
Renorios	N 43 7 53.89877	W 5 38 33.38436	1390.095
Vallinas	N 42 42 38.91640	W 5 36 38.55234	1140.781

Table 1.- Coordinates and error ellipses

Two years later (2006) a new GNSS observation campaign of the surface network was done in order to verify the local movement of some of the survey points and to integrate new survey points required in the network. Table 2 compares latitude and longitude coordinates obtained in the first and second campaign.

Control Points	Latitude Differences (m)	Latitude Differences (m)
102	-0.007	0.000
103	0.007	0.006
104	-0.004	0.006
201	0.007	-0.001
202	0.004	-0.004
203	0.007	0.001
204	0.007	-0.010

301	0.003	-0.004
302	0.000	0.000
303	0.000	0.011
305	0.003	0.005
2001	0.002	-0.001
3001	0.003	-0.003
3002	-0.006	-0.009
3004	0.004	0.001
3005	-0.003	0.006
3006	0.000	0.003

Table 2.- Differences between the first and second campaign

Differences obtained between the first and second campaign are in the range of 10 mm. We can say they are according to the accuracy of both instrumentation and methodology used.

Different computations tests were done involving different time span observation periods, elevation mask, kind of ephemeris and strategy of baselines computation. After analyzing the results the following conclusions were obtained: the optimal time span observation is about an hour. The use of precise or transmitted ephemeris does not influence the computation of baselines The use of elevation masks of 15º or 10 º does not affect the final accuracy of the network.

To transform coordinates to the ED-50 geodetic reference system, where the original tunnel project was referred, a stepwise regression method was selected. This procedure starts with a previous 3-dimension Helmert transformation. Then a horizontal Helmert transformation is performed to obtain North and East coordinates. Finally, a vertical adjustment has been done. This allows us to consider separately points in horizontal transformation and points in vertical transformation, according to ED50 genesis. The residuals of the transformation were in the range of 50 mm, with an important scale-factor of -22 ppm.

As a consequence of losing accuracy on coordinates due to the transformation between geodetic reference systems, a study was performed in order to analyze whether the loss of accuracy might or not affect the accuracy required for the work. One way to do study this effect would be to apply least square collocation techniques (Moritz, 1978), (Lachapelle, 1979) which has been also applied in Pajares tunnels. Another way of doing it is through a comparative study between the UTM (ETRS89) azimuth and UTM (ED50) azimuth for a set of directions inside the networks. These differences allow us to quantify the influence error due to transformation processing.

The following table 3 shows the differences between the azimuths on both systems ETRS89 and ED50:

Baseline	Azimuth ETRS89 (g)	Azimuth ED-50 (g)	Difference ETRS89-ED50 (cc)
101-102	208.2708	208.2717	-8.6
101-103	130.5521	130.5528	-7.1
101-104	93.5570	93.5577	-6.5
201-202	164.7110	164.7115	-4.6

201-203	130.7535	130.7540	-5.2
201-204	91.8235	91.8240	-5.9
301-302	34.1250	34.1253	-2.8
301-303	56.8341	56.8343	-2.5
301-305	101.3744	101.3748	-4.3
302-303	115.0315	115.0321	-5.9
101-201	159.0928	159.0934	-5.6
101-301	163.0538	163.0544	-5.2
201-301	172.8165	172.8169	-4

Table 3.- Azimuth differences between coordinates in both systems ETRS89 and ED50

The greatest difference is seen in the 101-102 direction located in the North portal (Telledo) because it is the shortest distance between all the control points. If we assign zero to the lowest computed value, then the relative difference is about six seconds at the most unfavourable case. This implies no significant loss of accuracy to achieve the future tunnels breakthrough.

The previously mentioned scale factor of -22 ppm was taken into account when the observations of the surface networks of tunnels were computed. This scale factor was applied to distances measured by conventional surveying methods (RTS) when computing the underground networks adjustment.

Once again a new problem appeared when carrying out the guided underground networks and their connection to surface networks:

- Inside the tunnel there is a coaxial laminar gas flow at a speed of approximately 2 ms^{-1} which presumably stabilizes the horizontal and vertical thermal gradients.

- The steady flow from the interior of the tunnel clashes with the outer atmosphere, not stabilized because, regardless of other climatic aspects, undergoes a diurnal cycle. The turbulence thus generated has a very negative influence on the transmission of observed directions from outside to inside.

The ideal solution would be to take surface and interior thermographies and schedule observations when conditions inside and outside match. On a practical level we used periods of time when inner and outer temperatures were similar to each other. In those moments azimuth transmissions were performed in order to analyze possible differences.

From the conclusions reached in Guadarrama Tunnels (Arranz, 2006), performing network simulations with different distances scenarios and analyzing the obtained results a *zigzag* kind of traverses of 250 meters length was finally selected.

Horizontal observations on the tunnels have three aspects: the measure of distances, angles and gyro azimuths. Observations of distances and angles have been made with LEICA TCA2003 RTSs. Nominal accuracy is 1" for angles and 1mm +1 ppm for distances. A GYROMAT 2000 was used to measure azimuths (Leica Geosystems)

Distance measurements procedures were not so difficult because the instrumental was properly verified and appropriate corrections by meteorological parameters were taken into account. In each of these measures, correction of meteorological parameters was applied, with the express exclusion of any other ones. For instance, scale projection and scale factor when changing height or refraction coefficient. All of them were incorporated on the network computation process.

Angle measurements were made using the method of six series of direction observation sets. Once the series were recorded, average angles and distances were "in situ" computed. Then standard deviation for horizontal angles, vertical angles and distances were finally computed. When the standard deviations were greater than 5cc, the series was rejected. If the standard deviation between sets was greater than 5cc, two new series were observed an added to the procedure, discarding the highest and lowest ones.

The geometric design of the underground networks along the tunnels has all the characteristics that geodetic references and manuals advise to avoid (Shepherd, 2003). With the help of the underground network, the TBM is controlled and further network verification of any kind is never performed again, simply because there are no control points until the drill is finished or the TBM meets another TBM. As the network groves and moves forward behind the machine, the accuracy obtained in the computation of the coordinates is also exponentially worsening. The following figure 3 shows this effect on the Buiza Portal underground network. The network also has the disadvantage of short distances from the portal point to some of the control points due to the complex topography of the area.

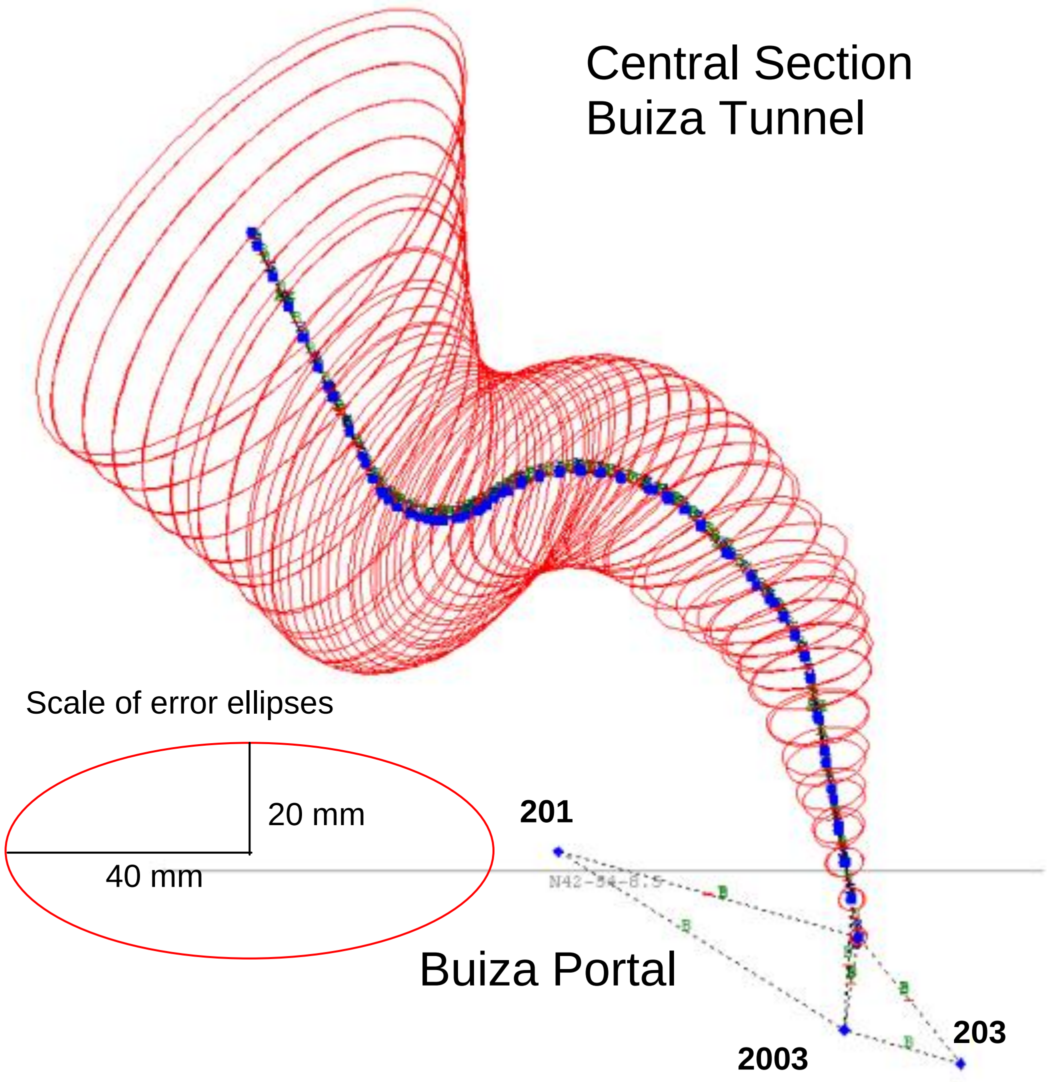

Figure 3.-Groving of error ellipses when increasing the network

The computation and adjustment of the coordinates of each of the stations belonging to these underground networks is as important as the analysis of errors found on them and their reliability. The standard deviations of the obtained coordinates indicate at any moment the degree of uncertainty in the movement and guidance of each TBM. The computation and adjustment of all these observations were performed by least squares, simultaneously with coordinate calculations. Standard deviations and horizontal error ellipses were obtained at a 95% confidence level.

The analysis of errors and therefore the reliability of the results were obtained following these steps:

1.- Selection of appropriate instrumentation based on the estimation of the observing errors: total station (angle and distances) and gyrotheodolite (azimuths). In addition, each of these instruments followed a calibration protocol throughout the tunnel construction process.

2.- 'A-priori' analysis and estimation of the errors due to the methodology of observation used in the tunnels.

3.- 'In situ' quality control of the series of observations obtained. Repetition of the series when required.

4.- Verifying the normal distribution of the errors, once the network has been adjusted by least squares method. In this phase, the maximum Tau criterion has been applied and then the observables are properly weighed. This phase is particularly important in the analysis of two key situations:

a.- Biases were detected during the computation process and isolated in order to avoid any influence on the correct guidance of TBM. Once biases are corrected and reduced a final adjustment was performed.

b.- The computed error ellipses gave us the reliability of the results. It was so important for us to know where we were located as to know the uncertainty.

5.- When the progress of the works allowed, coordinate differences of common points were obtained from observations done from other drilled tunnels. Those common points connected directly a pair of tunnels or through cross passages.

Some questions must be considered when adding gyrotheodolite observations to these networks. Without those azimuth observations the good results of accuracy and reliability might not have been reached. Angle and distance networks are based initially on the control points at each portal. As observations in the tunnel advance, errors grove, and accuracy and reliability of network guidance stations decrease.

Observations of gyrotheodolite are expected to be from 5 to 8 times worse than those obtained from RTSs. The great advantage of this instrument is that errors are not transmitted, in other words, azimuth observations are independent. The following figure 4 shows error ellipses with and without gyrotheodolite for the same underground network.

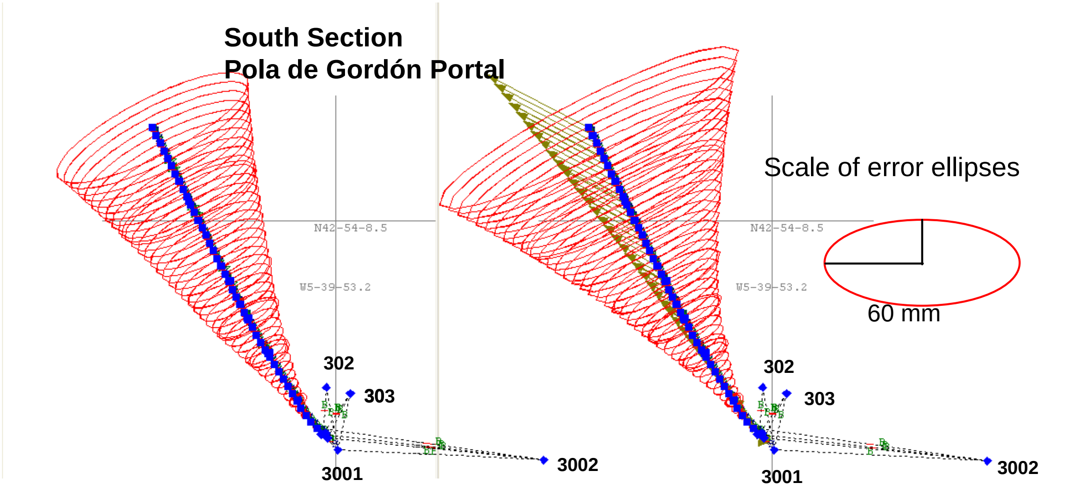

Figure 4.-Comparison of error ellipses with and without gyrotheodolite observations adjustment

The first drill finished by connecting two of the tunnels is placed at an approximate distance of 9,9 km from Pola de Gordon portal (south). The biggest uncertainty at that moment was estimated in 80 mm. Table 4 shows the coordinate difference in common points from both tunnels.

SOUTH LINK EASTERN TUNNEL AND WESTERN TUNNEL				
COORDINATES DISPLACEMENT				
Points	NORTH (m)	EAST (m)	σ North (m)	σ East (m)
26265E	0.015	-0.020	0.021	0.036
26284O	0.014	-0.019	0.021	0.036
DISPLACEMENT ALONG TUNNEL AXIS				
Points	FORWARD (m)	TRANSVERSAL (m)		
26265E	-0.011	0.023		
26284O	-0.011	0.021		

Table 4.- Coordinate differences and formal errors on common points from eastern and western

The difference or shifting from one tunnel to the other is in the range of 15 mm in Northern component (y), with a standard deviation of 21 mm, and 20 mm in East component (x) with 36 mm standard deviation. These results, which can be classified as excellent, were expected and within the range of uncertainty, take into account that these points were reached almost at 10 km along underground network. In the same table, error components in forward and transverse direction to the tunnels axis have been computed. Displacement in TBM advance direction is 11 mm, while in the transverse displacement does not exceed 23 mm in the worst case.

We also present the Southwest tunnel link to the tunnel of Buiza portal, with observations through the gallery no. 24, which connects both tunnels at an approximate distance of 10.2 km from the Southwest end of the tunnel and at an approximately a distance of 5.5 km. from the central portal. The results of the link between the two tunnels are reflected in Table 5.

LINK SOUTH WEST TUNNEL AND TUNNEL OF BUIZA

COORDINATES DISPLACEMENT				
Points	**NORTH (m)**	**EAST (m)**	**σ North (m)**	**σ East (m)**
FG024D	-0.052	-0.006	0.020	0.027
FG024I	-0.048	-0.005	0.020	0.027

DISPLACEMENT ALONG TUNNEL AXIS		
Points	**FORWARD (m)**	**TRASVERSAL (m)**
FG024D	0.044	0.029
FG024I	0.040	0.026

Table 5.- Coordinate differences and formal errors on common points western and Buiza tunnels

The displacement from a tunnel with respect to the other is in the range of 52 mm in Northern component (y), with a standard deviation of 20 mm, and 6 mm in East component (x), with 30 mm standard deviation. Those results were also expected within the margin of uncertainty. These control points were reached along of nearly 11 km of underground network in one of the tunnels. The other one was half long and described curve radii close to 500 m as can be seen on figure 5. Here the gyrotheodolite observations on the curve area were performed on each traverse axis.

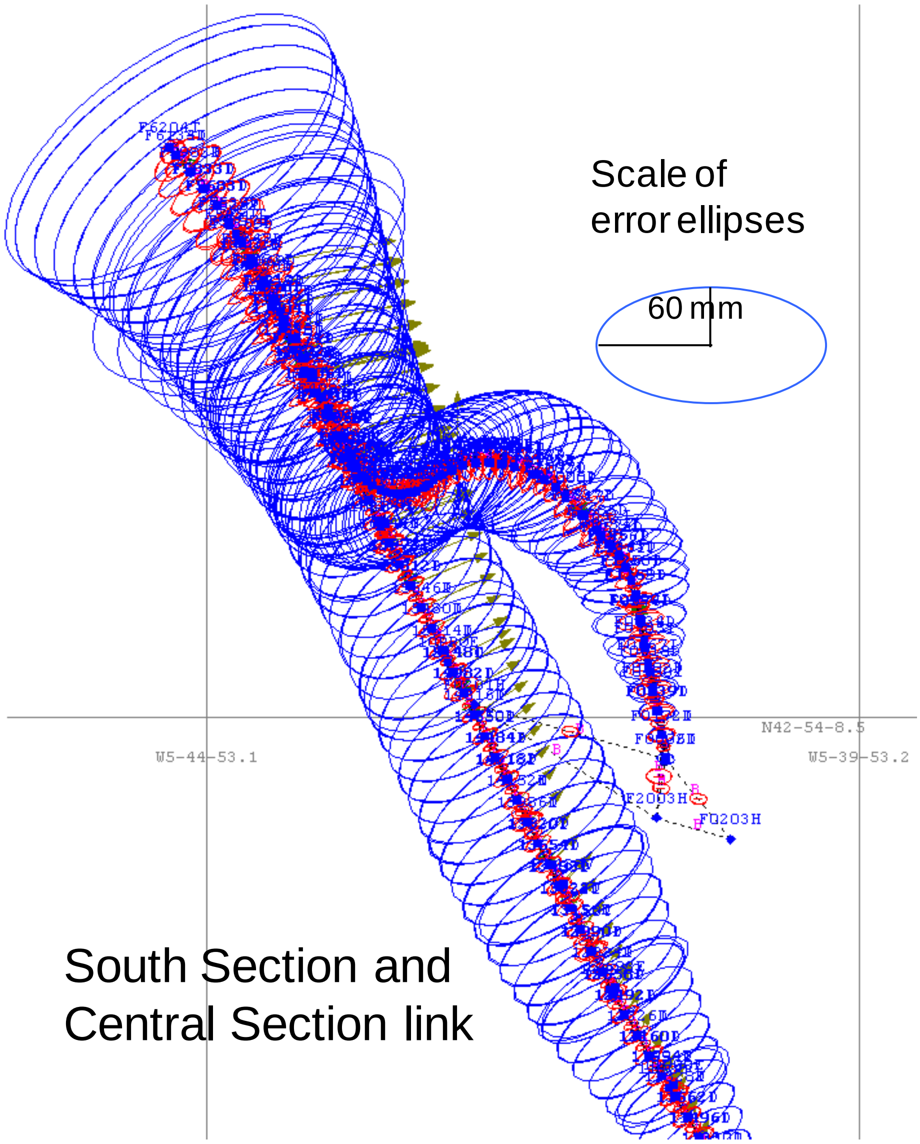

Figure 5.-Eastern and Buiza tunnels link

Also error components in forward and transverse direction to the tunnels axis have been computed and shown on the table. Displacement in TBM on advance direction was 45mm, while transverse displacement did not exceed 30 mm in the worst case.

Subsequently, the second link with observations along cross passage #27 was performed together with a third connection along cross passage #33. As result of cross passages #27 and #33 links, common points of cross passage #33 were located at an approximate distance of 13.5 km from East-south tunnel and with a maximum uncertainty of 80 mm. From the other intermediate portal, the distance was approximately 8.5 km.

The difference or shifting from both tunnels are in the range of 60 mm in Northern component (y), with a standard deviation of 27 mm, and 50 mm in this component (x), with standard deviation 36 mm. Results are within the margin of uncertainty in relation to their respective portal distances.

Also the same table shows the components of error in forward and transverse directions. The maximum displacement in TBM advance direction is about 20 mm, while in the transverse direction reaches no more than 50 mm in the worst case.

4. CONCLUSIONS

It can be said that differences found in each of the points of each link done have produced an excellent result. After analyzing deviations and error ellipses which indicate the error these points have been determined with, it can be said that measured values of displacement are as expected, tolerable and reliable.

The values of the discrepancies found in each tunnel, separately adjusted and checked along their common cross passage, would be at a maximum value at about 50 mm. Those values are also consistent with the values that provide the uncertainties computed for these same points that remain within that range or even higher.

It is important to highlight that errors found in each of the adjustments done are of similar magnitude to the deviations experienced by comparing these coordinate adjustments two by two. Theoretically speaking, there would be no possibility of saying that any of these adjustments are better than the others, being equal displacements and uncertainties, as indeed expected.

Given the results presented in this communication, the best suitable methodology for this type of work may have the following characteristics:

1 – Surface network observations must be done by GNSS techniques. Static method in each survey point must have multiple observations of at least 1 hour which guarantee repeatability and reliability.

2 – In order to evaluate the loss of accuracy due to the change of geodetic reference system, azimuths in the sides of surface geodetic network between two systems should be compared. Computed differences give a quantification of the error due to the transformation of GRS, assuming the hypothesis that the transformation errors are due to the geodetic reference system of the project, unless geodetic reference system is a global one.

3 - As the axis of the tunnel has to be free, the underground networks must be designed as *zigzag* traverses, in order to minimize lateral refraction error. Optimal traverses shall have 250 m length sides. At least six sets of observations have to be performed.

4 - Gyrotheodolite observations are needed to reduce the loss of accuracy on the transmission of azimuths in traverses of this length. From 4 km on, observations should be performed every kilometre, observing two crossed axes, in order to minimize lateral refraction error. On critical areas, such as curves, the observations must be performed on each traverse axis.

5 - Traverses along tunnel axis with sides of 375 m. are most suitable to control the underground network and the observation is restricted to times when technical stops inevitably happen in this type of work.

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