THE CONSTRUCTION OF THE SAN CRISTOBAL TUNNEL (SANTIAGO DE CHILE)

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ABSTRACT

The San Cristóbal Tunnel is located at Santiago de Chile city and will connect the pheripherical urban motorway of Américo Vespucio, that surrounds Santiago, with the downtown, between El Salto Avenue (Comuna Huechuraba) and El Cerro Avenue and Lo Saldes Bridge in Providencia municipally, connecting later with Kennedy Avenue that goes to the Andes Range.
It has a twin tunnel scheme of 1808 m length each tube, with a functional width of 9 m, considering two road lanes in each tunnel, except in the South Portal of the eastern tunnel that has three lanes.
Also eight cross passages have been excavated to connect both tubes, for safety purpose, every 200 m.
The tunnels have been excavated basically in andesites of good and fair geomechanical quality, with RMR usually higher than 50 points, except the first 100 m section located at the south portal that have been excavated in coluvial soils, and also at some faults encountered with a total length of 150 m approximately in each tube.
The tunnels have been constructed following the NAT method, without top heading and benching, and therefore full face excavated with a total section of 80 m^2, and tunnel pulls ranging from 2 to 4 m.
The south section of both tubes, excavated in coluvial soils, was also full face excavated, although in the East Tunnel (Tunnel C2) the total section was 158 m^2, considering three lane road tunnel. This section was really a challenge and a special care was taken, including:
- Forepoles of 12 m length each one, overlapping 3 m one each other, for a total length of 100 m.
- Mechanical excavation in the 158 m^2, using either hydraulic hammer or conventional digging.
- Invert, constructed weekly following the tunnel face advance.

The excavation of the tunnel started in April 2006 from both South portals, after a soil nailing treatment, and at June 2006 from the North Portal. The end of the excavation took place in March 2007, with an average advance rate of 303 m/month considering the four tunnel faces.
In the South sector, in soils, the average advance rate was 28.2 m/month in both tunnels, while in the conventional rock mass sections the higher advance rate was 389.7 m/month.

1. INTRODUCTION

San Cristóbal relief is a natural barrier between the NW and the SE of Santiago de Chile city and constitutes an exceptional touristic viewing point of Santiago.
Nowadays the unique traffic connection between these two parts of the city is the Peripherical Avenue “Américo Vespucio” that crosses this relief at the sector called “La Pirámide”, with the consequent traffic congestion.
The project consist in the excavation of twin road tunnels that will increase the connectivity between the North and the South of Santiago, connecting “Américo Vespucio Norte” from Avenue “El Salto” in “Huechuraba” with the municipality of “Providencia” at two places Avenue “El Cerro” and “Lo Saldes” bridge.
The population that will directly be benefit from this new infrastructure is about 1,8 M people and the total cost of it, is 110 MUS$.(www.tunelsancristobal.cl)
The excavation of these tunnels was finished last May 2007 and it will be opened by the end of June 2008.

2. TUNNEL DESIGN

a) Geometrical and functional data

The Tunnel of San Cristóbal has a twin tunnel scheme of 1808 m length each tube, with a functional width of 9 m, considering two road lanes in each tunnel, except in the South Portal of the eastern tunnel that has three lanes.
Also eight cross passages have been excavated to connect both tubes, for safety purposes, every 200 m.
Figure 1 shows the functional section of the tunnel while in **Figure 2** the general layout of the tunnel has been included.

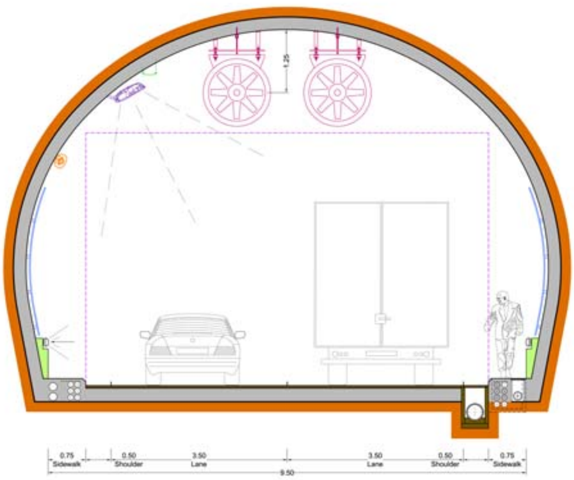

Figure 1. – Tunnel Section

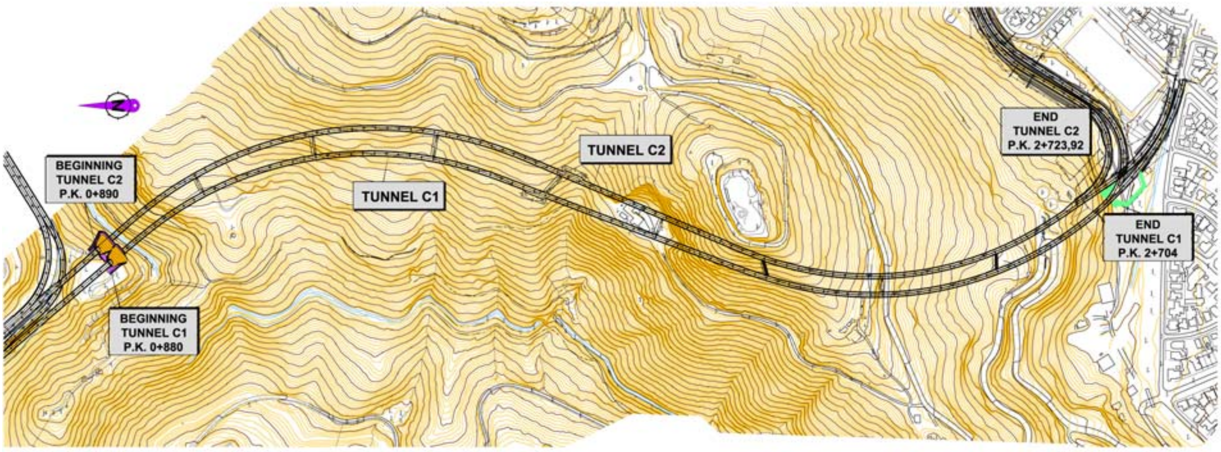

Figure 2. – General Tunnel Layout

b) Geological data

The tunnels are located in the Pre-Andine reliefs. One of these reliefs is known as **"*Cordón del Cerro San Cristóbal*"** in which several types of types of stratified volcanic and continental rocks of "*Abanico*" formation (Upper Cretaceous to Oligocene) can be distinguished (Aguirre, 1960). Also some stock intrusive rocks from the Lower Miocene are present (Intrusive Unit I and II). Finally several non-consolidated quaternary deposits fill up the central basin (alluvial, glacial, volcanic ash, lacustrine sediments, etc.).
The following three **geological units** have been considered:
- **Andesitic lava and volcanoclastic levels (Tv and Tvt)**

The lavas are basically andesites and porphyric andesites (Tv), greenish and grey while the pyroclastic rocks are constituted by tuffs (Tvt). This stratified formation is gently folded (NS to N20º) dipping between 15º and 25º to the East. In this unit the following rocks have been distinguished: Andesites (A1), Porphyric andesites (A2)and Tuffs and volcanic breccias (A3).

- Intrusive rocks (Tp)
They correspond to stocks, dykes and volcanic pipes that are intrusive to the “Abanico formation”. In this unit two different rock type have been considered: Porphyric andesites (P1) and Hydrothermal altered porphyric andesites (P2).
- Non-consolidated deposits
They are basically elluvial soils (Qv), slope debris (Qc, colluviums), alluvial sediments (Qf) and antropic debris (Qx).
From all of them the slope debris must be remarked as since the beginning of the basic engineering jobs, the presence of important slope debris was detected in the South portal. In this sector an old landslide affecting the eastern tunnel exists. This landslide has a thickness of 30 m and involves not only the colluvium but also the weathered head rock. Basically they consist in debris (gravels and boulders in a silty-clayly matrix supported) moved downslope largely by gravity creeping specially with rainfall.
Figure 3 shows the longitudinal geology of the tunnel where it can be observed that most of the tunnel has been excavated in different types of andesites except the south portal in which there are some tuffs and the landslide described above. Finally it can be observed the existence of 10 main faults distributed along the tunnel.
Three main phases have been described (Thiele, 1980):
- Tecto-genetic phase (Oligocene), affecting the Abanico formation that its part of the Pre-Andine Unit.
- Second Compressive Phase (Upper Miocene-Lower Pliocene), with smooth folds and intrusive rocks defining the Andine Unit.
- Third Phase (Pliocene up to today), were the Central half graben is formed as well as the main existing morphological features.
The Abanico formation is composed by massive stratigraphical sequences dipping between 15 and 25º to the East corresponding to the first tectonic phase. Some small structures have been detected. Finally the rock mass is fractured by several joints sets and faulting.
Figure 4 includes a rosette diagram showing the distribution of the fault strikes distribution.

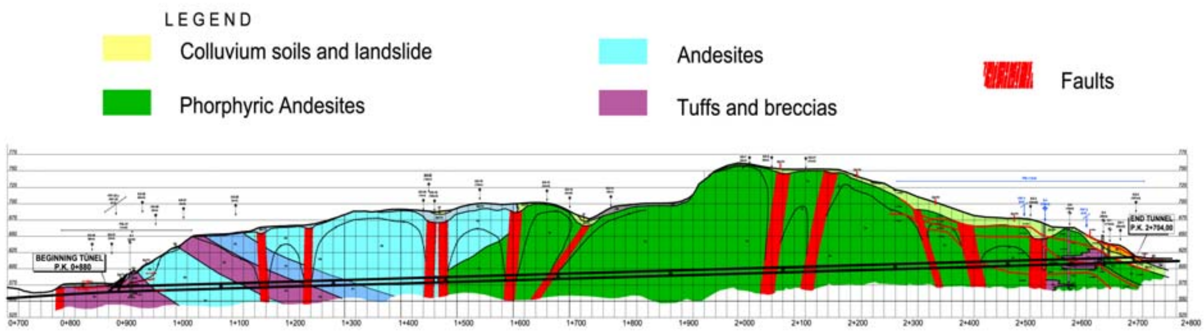

Figure 3. – Geological Cross Section of the Tunnel

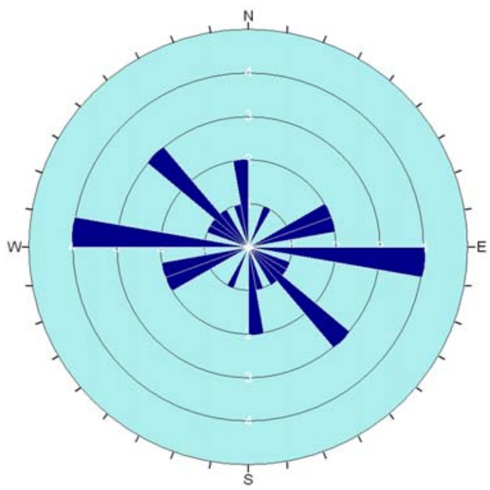

Figure 4. - Fault stripes rosette diagram.

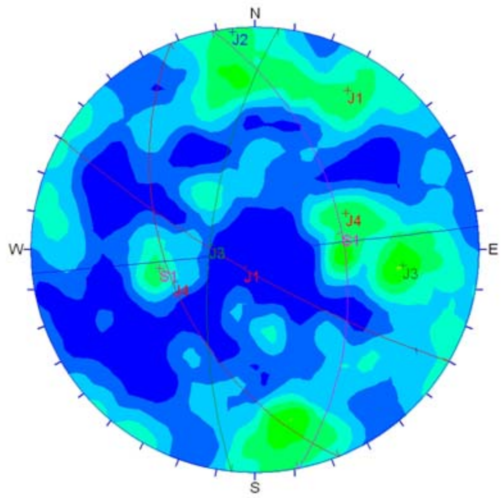

Figure. 5. - Contour Plot showing the five joint sets detected

From this figure five main faulting directions are obtained.
Finally **Figure 5** shows a plot containing the results of structural data measured. In this contour plot the pole concentration can be observed as well as the location of the five main poles corresponding to the existing joint sets.
For all the joint sets statistical analyses have been done to determine the scatter of discontinuities in terms of persistence, roughness, aperture, and spacing.
According to the Chilean seismic standard the tunnel is located in the Second Region. Considering the type of rock mass and the type of construction the seismic acceleration to be considered is 0.3g.
The natural stress field adopted corresponds to a Ko stress coefficient distribution of 1.8 in the E-W direction and 0.8 in the North –South direction.
Finally the water presence will not be important and will be reduced to some drops and small inflows.

c) Geotechnical data

Rock mass has been intensively studied to make a realistic prediction about the stress-strain behaviour of it.
For this purpose the following site investigation was done: 7 boreholes, 2652 m of seismic refraction profiles, 1960 of Electrical Resistivity Tomography profiles, and 2600 of Electromagnetical boreholes. Also an intensive program of lab test was done.
From all the above information and considering the geology of the tunnel, the geomechanical characterization of the rock mass has been done. **Table 1** includes the properties assigned to the different existing lithologies.

Lithology	RMR	Overburden	c (MPa)	Φ (º)	E (MPa)	ν	ρ (t/m^3)
Porphyric Andesites	61	145	1,26	53	11.302	0,21	2,69
Porphyric Andesites	54	170	1,07,	50	7.554	0,21	2,69
Porphyric Andesites	35	60	0,32	45	4.121	0,30	2,5
Tuffsand Breccias	40	42	0,21	38	2.484	0,30	2,5
Fault	25	90	0,12	26	1.124	0,35	2,4
Fault	25	170	0,24	23	1.303	0,35	2,5
Colluvium	-	30	0,04	35	100	0,35	1,80

Table 1. – Rock mass geological properties.

d) Construction method

The selected constructive method has been NATM, while the support has been designed using DEA methodology (Celada, 1997).
The excavation that has been done using both, blasting and mechanical, was decided to be done full faced although in one of the faces the excavation section reaches 158 m^2.
With the mentioned rock mass properties several 2D and 3D FLAC calculations have been done. As result of this calculations 7 different rock supports types have been defined. **Table 2** shows the main data concerning each type of support class The application of these supports as it is shown on Table 2 depends on the rock mass geomechanical quality expressed by its RMR, the overburden and an special support was designed for the tunnel excavated in the South Portal in soil colluvium deposits.
In all cases the rock bolts used have been fully grouted with cement 3 m length and the shotcrete used has been 30 MPa of compressive strength reinforced with fibres. The steel ribs used were THN type of 29 kg/m of weight.
As an example in **Figures 7** and **8** it is shown the rock support classes IV and VII which are respectively the heaviest with rock bolts and with steel ribs.
As special reinforcement in some support classes an invert was considered as well as micropile forepoles.
Finally as it will be described later in some case it was necessary to reinforce some tunnel section with long bolts 9 m long.
Finally a shotcrete lining with a minimum thickness of 5 cm was applied all over the tunnel.

Support class	Application	Shotcrete sealing (cm)	Bolts L=3m	Shortcrete Sh 30 with fibers (cm)	Steel Ribs	Lining (Sh 30)	Others
ST - I	RMRc > 70	3	2,5 (L) X 2,5 (T)	-	-	7	
ST - II	60 < RMRc < 70	3	2,0 (L) X 2,0 (T)	3	-	7	
ST - III	45 < RMRc < 60	3	1,5 (L) X 2,0 (T)	7	-	7	
ST - IV	35 < RMRc < 45	3	1,5 (L) X 1,5 (T)	15		7	
ST - V	25 < RMRc < 35	3	-	15	THN 29 a 1,5 m	5	
ST - VI	RMRc < 25 until 100 m overburden	3	-	15+10	THN 29 a 1,0 m	5	Invert
ST - VIIa	RMRc < 25	3	-	15+15	THN 29 a 0,75 m	5	Invert
ST - VIIb	colluvium	3	-	15+15	THN 29 a 0,5 m	5	Invert Forepole

Table 2. – Rock Support Classes

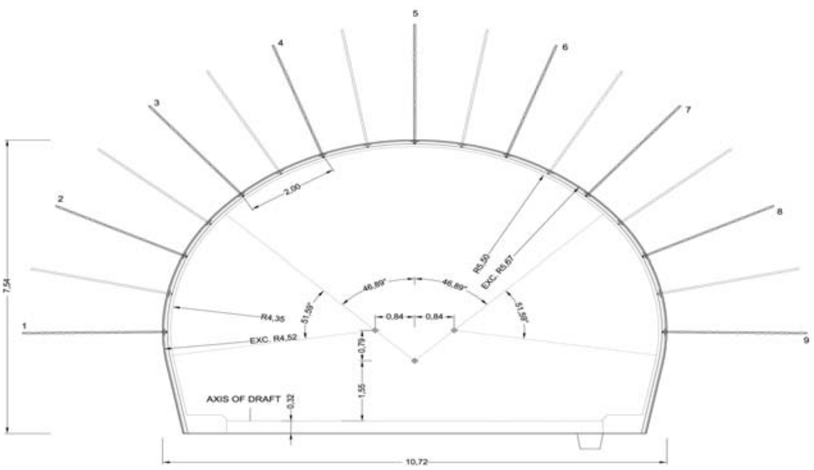

Figure 6. - Excavation Class IV

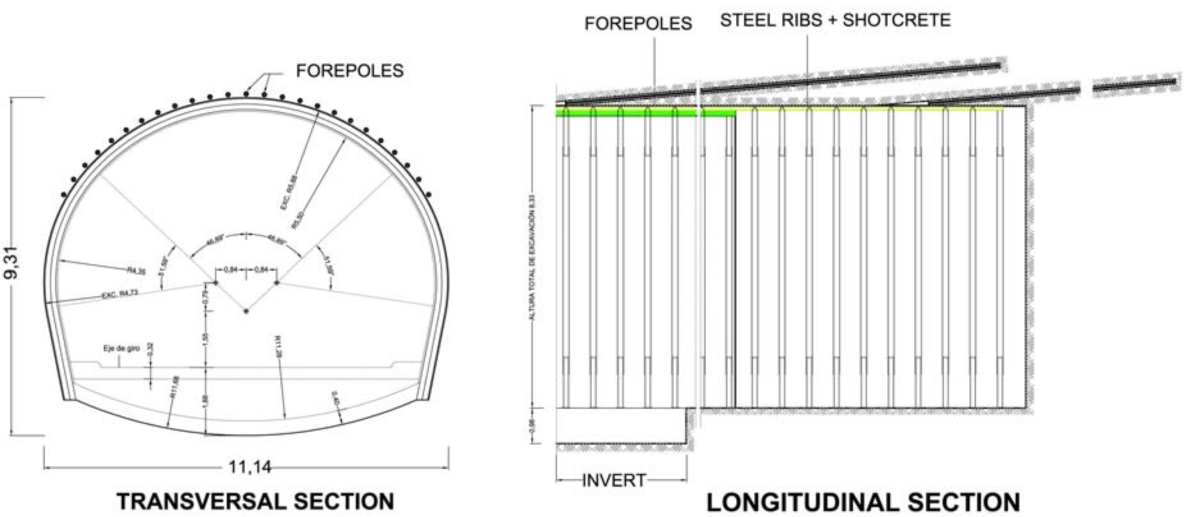

Figure 7. - Excavation Class VII

3. TUNNEL CONSTRUCTION

Following the main data concerning the construction of both tunnels is described.
The excavation of the tunnels started in April 2006 and finished on April 2007, so 12 months were necessary to complete the excavation of 3629.61 m of tunnel. This gives an average advance of 303.3 m/month. In 10 of these 12 months the tunnel has been excavated using 4 faces, two coming from the South and two from the North. It must be remarked that really only two complete equipments were available, one for the southern faces and another for the two northern faces.
The western tunnel was called C1 tunnel and was excavated from the North (face C1N) and from the South (face C1S). The eastern tunnel was called C2 and accordingly two faces were used (faces C2N and C2S).
Table 3 shows the rates of advance reached by the 4 excavation faces. From this table it can be clearly derived that the advance from the North portal was higher than the one obtained from the South portal. It must be considered that the face called C2S that is the one with a poor rate of advance is the one that has excavated the colluvium and the landslide located at this portal and also 65 m of the 702.03 m excavated in this face had an exceptional section of ranging from 140 to 158 m2 corresponding to the transversal section with 3 lanes.
To be precise the rate of advance obtained has range between 63.8 m/month for the face C2S to 111.8 m/month fro the face C2N.

	Ap-06	May-06	Jun-06	Jul-06	Aug-06	Sep-06	Oct-06	Nov-06	Dec-06	Jan-07	Feb-07	Mar-07	Apr-07	total
C1S	19,19	38,45	35,92	63,69	49,47	83,83	28,3	135,2	143,8	159	154,4	58,8		970,05
C2S		18,55	21,03	33,67	39,58	42,7	61,5	53,6	50,6	86,8	113,8	90,7	89,5	702,3
C1N		6,97	42,93	113	142,8	128,8	146,4	9,6	22,7	50,5	96,9	79		839,60
C2N			45,93	73,8	106,3	102,9	143,7	191,3	81,2	75,7	84,2	124,7	88,2	1.117,93
total	19,19	63,97	145,81	284,16	338,15	358,23	379,90	389,70	298,30	372,00	449,30	353,20	177,70	3.629,61

Table 3. – Rate of advance obtained during the excavation of the tunnels

a) Portals
Both portals were extensively studied to optimize its design in order to reduce the environmental impact. It must be taken into account that in both cases the portals have special importance. In the North portal as it can be clearly seen from all the Huechuraba and El Salto city sector. And in the case of the South Portal as it is located in a high standard sector of the city (Pedro de Valdivia Norte). Therefore in both cases the high of the portal slopes have been minimize. **Photos 1** and **2** show the excavation of both portals.
In the 4 cases a micropile forepole was done prior to the start of the excavation. In the North portal a reinforcement of the frontal slope with rockbolts and shotcrete was done while in the South portal the reinforcement was done with a soil-nailing.

Photo 1. – North Portal

Photo 2. – South Portal

b) Construction development
As it has been described the excavation of both tunnels took place from April 2006 to April 2007.
The final length excavated in tunnel C1 (going into Santiago) is 1809.65 m while the final length for tunnel C2 is 1819.96 m. Both totalize the sum of 3629.61 m of tunnel.

During the excavation all the faces have been characterize and mapped to obtain the RMR value of the face and the main data of the structural discontinuities. Systematically, all the data concerning the RMR and discontinuities have been obtained.
The value of RMR as well as the evolution (magnitude and velocity of convergence at the tunnel) has been the two main criteria to select the rock type support to be applied for each face.
The distribution of the lithologies as well as the position and thickness of the faults coincides basically with the predicted longitudinal geological cross section that has been included at Figure 3. The main difference nevertheless it was one of the places in which the site investigation has focused, has been the South portal where the thickness of the removed mass is bigger than expected. Consequently the stretch of tunnel affected by soil deposits has been of 90 m instead of 65 m as it was predicted in the design engineering.
Table 4 and **5** include the comparison between the prediction of the rock support type distribution and the support that have been really applied in the excavation of both tunnels. As it can be observed two main conclusions can be derived from these tables. The first one is that the sum of supports V, VI and VII (with steel ribs) for tunnel C1 and C2 was originally predicted as 14.65 and 17.3 % respectively while the real length applied correspond to 18.65 and 19.97 %, that gives a difference of 4 and 2.7 %. These differences can be considered small and therefore the prediction done in the design engineering, acceptable.
The second conclusion is that roughly the support types I and II have change their distribution, so the values of RMR and consequently the geotechnical quality of rock mass have been better than foreseen.
Considering both conclusions it can be said that the real costs have not been so different as the predicted.

EXC. CLASS	DESIGNED		CONSTRUCTED	
	m	%	m	%
ST-I	282,1	15,63%	856,6	47,12%
ST-II	761,33	42,19%	343,6	18,90%
ST-III	360,18	19,96%	232,05	12,76%
ST-IV	136,39	7,56%	46,82	2,58%
ST-V	43,75	2,42%	226,49	12,46%
ST-VI	137,53	7,62%	56,74	3,12%
ST-VII	83,11	4,61%	55,77	3,07%

EXC. CLASS	DESIGNED		CONSTRUCTED	
	m	%	m	%
ST-I	245	13,38%	776,4	42,41%
ST-II	814,38	44,48%	386,1	21,09%
ST-III	321,5	17,56%	219,57	11,99%
ST-IV	133,28	7,28%	83,13	4,54%
ST-V	50	2,73%	239,57	13,09%
ST-VI	177	9,67%	82,41	4,50%
ST-VII	89,79	4,90%	43,57	2,38%

Tables 4 and 5. – Comparison between the excavation classes applied and the prediction

c) Monitoring

The rock mass has been monitored to measure its real behaviour. The monitoring has consisted basically in convergence measurements. The total number of convergence station used inside the tunnels have been 74 for the tunnel C1 and 71 for the tunnel C2.
The convergences have been systematically measured using topographical total station and optical targets. But in those sections in which a better accuracy was necessary basically at faults, the method was changed measuring the convergence with extensometer tape.
The convergence control has been a very useful tool to decide in the critical sections the reinforcement to be applied as well as to have an objective criteria to start the construction of the shotcrete lining over the tunnel support.
In order to obtain further information about the real stress-strain behaviour of the rock mass and the support also some extensimeter as well as total stress cells at the shotcrete were installed at some transversal sections.
A special mention must be done to the South Portal due to the presence of the already described land slide and the existence of two big water reservoir deposits (Aguas Andinas). In this location an extensive instrumentation was decided, including 2 inclinometers, 2 extensometers and four topographical profiles for monitoring the surface and subsurface movements above the tunnels. Further it is discussed the results of these instrumentation.

4. MAIN PROBLEMS ENCOUNTERED

During the 12 months constructing San Cristóbal Tunnels, basically two main difficulties were encountered. The first one since the beginning associated to the existence of a landslide in the South Portal and the second one, later, associated to the presence of an important fault with the maximum overburden and therefore with a relative high stress conditions. Following, briefly both circumstances are described.

a) Portal South (C2S)
As it has been described the South portal was since the beginning one of the most problematic places for the construction of these tunnels. Due to the city urban characteristic of the site it was not possible to optimize the location of these two south portals. It was only to avoid tunnel C1 to be affected by this landslide.
It must also be considered that the first 60 m of the tunnel C2 have three traffic lanes instead of two and the excavation width is 18.1 m while the excavation section is 158 m^2.
The South sector of tunnel C2 between PK 2+723 to 2+700 was very critical as it was excavated in removed colluvium and elluvial soils. The tunnel was excavated under a heavy umbrella of micropiles (excavation class VII) and the following support:
- 3 cm of shotcrete reinforced with fibres sealing
- 15 cm of reinforced shotcrete
- Steel ribs THN-29 type, ½ m spaced
- Second layer of 15 cm thickness of reinforced shotcrete
- Structural invert.

Photo 3 shows the aspect of one of the faces while excavating this tunnel.

Photo.3. – Detail of Tunnel C25. (South Portal) in colluvium soils.

Although this heavy support the behaviour of the tunnel observed from the convergence, was that nevertheless convergences were not so high; a horizontal divergence was measured as well as a clear sinking of the tunnel.
Figure 8 shows this behaviour. In it can be observed a unstable vault descent of 56 mm, while in the tunnels walls these descents are 27 and 17 mm respectively. It was decided to spread 10 cm of additional shotcrete. Also it was decided to install horizontal self-boring bolts 6 m long spaced 1 m in each wall of the tunnel to avoid the descent of the tunnel structure.
The result of this reinforcement is shown at **Figure 9**, where it can be observed how the vertical and horizontal convergences were stopped with velocities lesser than 0.1 mm/day. Finally the section was stabilized.
An intensive control was taken due to the mentioned existence of two water reservoir above the tunnel. **Figure 10** shows a scheme in which the position of the two water reservoirs in relation with the tunnels can be observed. Also in this figure all the instrumentation (extensometers, inclinometers and topographical measurements) is shown.
During the excavation of the tunnels some movements at surface were measured. A clear correlation between the tunnel movements and the one observed at surface was detected. Nevertheless all the described reinforcement measurements produced an stabilization of the land surface movements as it can be observed at **Figure 11**, that correspond one of the control milestones located between the portal and the water reservoir deposits.

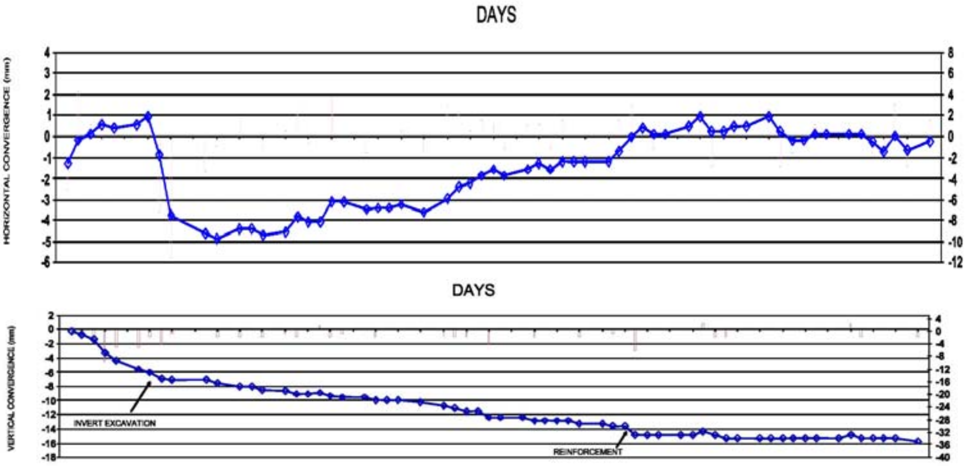

Figure 8. – Convergence Station measurement C2 PK 2+720

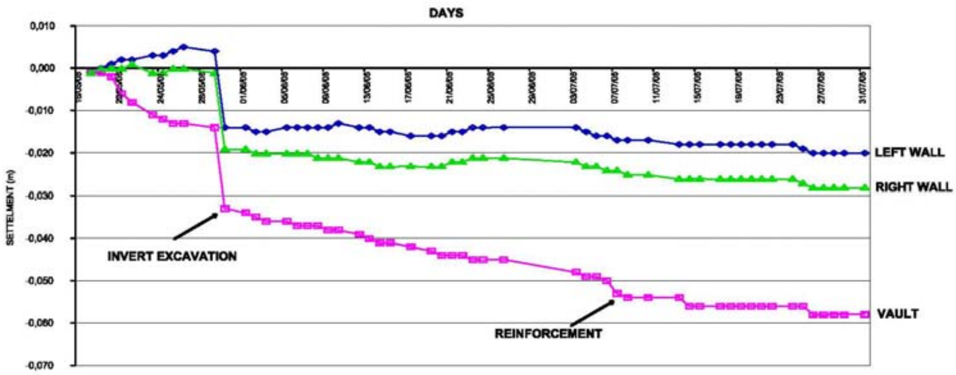

Figure 9. - Descent of the tunnel vault in the convergence station measurement C2 PK 2+720

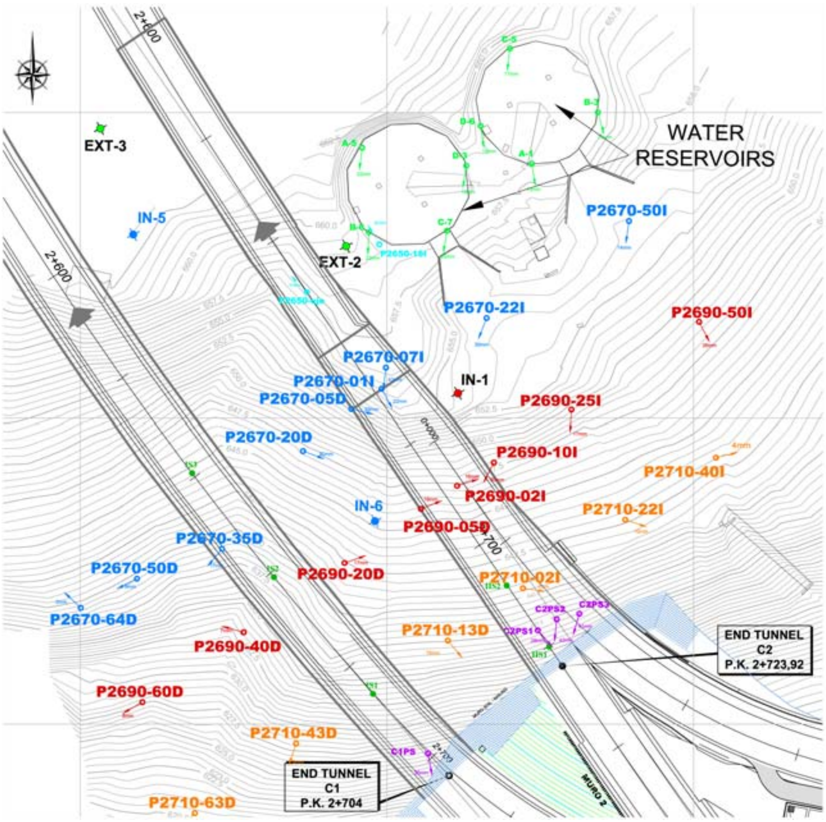

Figure 10. – South Portal of San Cristóbal Tunnel

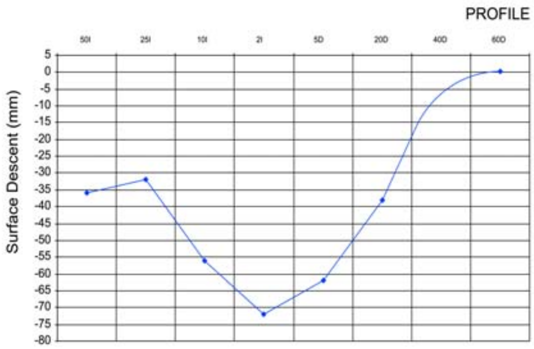

Figure 11. – South Portal surface settlements **Photo 4. – Face at PK 1+48, showing the fault at the right hand side**

b) Fault C1

The western tube (Tunnel C1N) crossed an important fault with a total thickness of 115m (PK 1+ 460 to 1+575) where the overburden is approximately 100m. The fault was mainly located in the right hand side of the tunnel (considering an advance from the North portal) as it can be observed in the **Photo 4.**

The excavation of this 115 m took place between December 2006 and February 2007 with the following Excavation Classes.

From	1450,00	to	1466,00 : ST-V
From	1466,00	to	1477,00 : ST-VII
From	1477,00	to	1479,00 : ST-VI
From	1479,00	to	1482,00 : ST-IV
From	1482,00	to	1541,00 : ST-V
From	1541,00	to	1543,30 : ST-IV
From	1543,30	to	1571,30 : ST-V
From	1571,30	to	1575,30 : ST-IV

In all the cases the advanced was done under a reinforcement rock bolt umbrella except in the section VII in which a micro pile forepole was applied. All this section was fully monitored with 14 convergence sections and other complementary instrumentation inside the tunnel.

Figure 12 shows the response of the ground, with convergences that reached 5 cm. Some of the convergence sections showed an unstable behaviour with relative high velocities and some fissures and minor cracks were developed at the shotcrete.

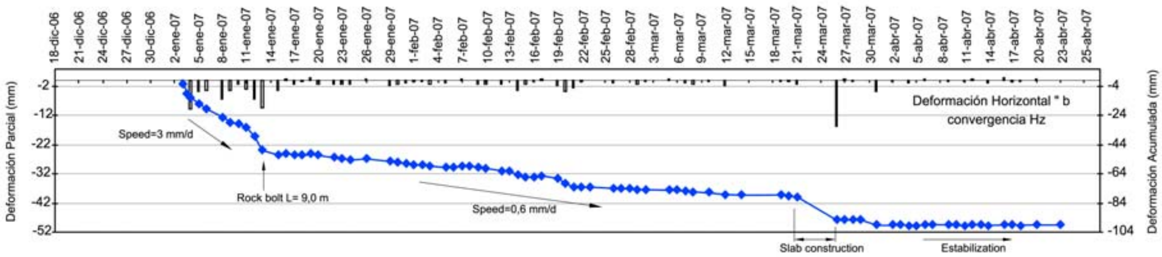

Figure 12. – Convergence section at PK 1+485.

For this reason it was decided to reinforce some of the unstable sections with additional support. The reinforcement carried out was:

- From 1+470 to 1+477 (first week of January 2007), rock bolts of 6 length and 10 cm of shotcrete.
- From 1+484 to 1+490 (first week of January 2007), the same previous described reinforcement.
- From 1+481 to 1+491 (second week of January 2007), with 7 rock bolts rows each one with 3 bolts 9 m long.
- The rest of the section described was reinforced using self-boring bolts of 9 m of length.

All these additional reinforcement support slowed dramatically the convergence velocities from 4 mm/day to 0.6 mm/day. But still, as it has been shown at the figure, a clear tendency was observed.

Due to this tendency it was decided to carry out a three-dimensional back analysis of the fault section, reproducing the deformations measured as well as the construction sequence and the supports applied. **Figure 13** shows a detail of the•-3D model used that was solved using FLAC-3D code (Cundall, 2006).

As a result of this back analysis it was decided to close structurally the support by means of a steel reinforced slab of 25 cm of thickness. The construction of this slab was initiated by the end of March 2007 very carefully side by side using stretches of 5 m long in order to protect the existing tunnel support. Although this precaution, as it can be clearly observed at the previous figure, some important movements took place but afterwards a sharp decrease of the convergence velocities occurs, with the final stabilization of all the convergence sections.

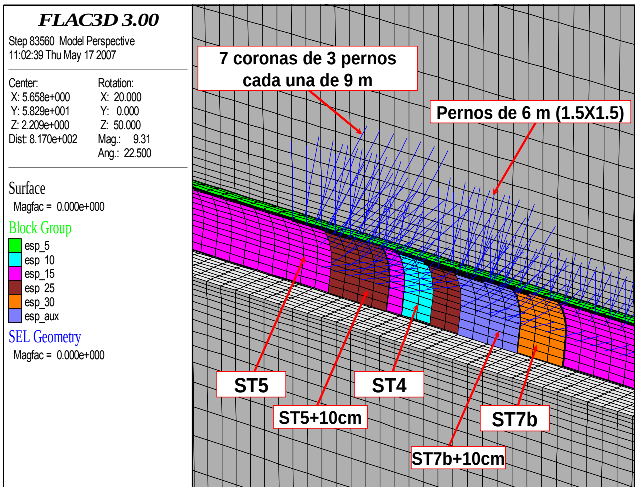

Figure.13. – Detail of the 3D numerical model used for back analysis.

5. CONCLUSIONS

From the design and construction of the Tunnel of San Cristóbal the following conclusions can be derived:
-The full face excavation even in soils, allow the employ of high efficiency equipments that provide excellent advance rates and optimum safety condition.
- A good combination of geotechnical characterization, numerical 3D modelling and convergence measurements allows solving relevant problems during the tunnel construction.
- The comparison between predictions and the real behaviour of the rock mass shows that the design done had been accurate enough without relevant costs deviations.
- Even considering the high complications and geotechnical problems happened, the rate of advance obtained during their construction was satisfactory.

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