Metro tunnels in Buenos Aires. Development of construction procedures 1998-2009.

Túneles para subterráneos en Buenos Aires. Desarrollo de procedimientos constructivos 1998-2009.

Alejo O. Sfriso
University of Buenos Aires and SRK Consulting (Argentina) SA
Rivadavia 926 of. 901 (C1002AAU) Buenos Aires, Argentina.
(+54-11) 4342-5447/3160. (+54-11) 5291-5118/5119
asfriso@srk.com.ar

ABSTRACT: Buenos Aires, Argentina, is expanding it's metro network. Some 17 km of new tunnels have been excavated betwwen 1998 and 2009, and some 20 km are scheduled for construction in the near future. Many major improvements have been implemmented during these years in the fields of design and construction procedures of NATM tunnels. Some of the achievements and lessons learned are described in this paper, including: characterization of Buenos Aires soils for the numerical modeling of NATM tunneling, description of the design and construction procedures in use and some comments on the observed ground behavior during construction. The paper is an update of (Sfriso 2008).

RESUMEN: Buenos Aires, Argentina, está ampliando su red de subterráneos. Se han construido unos 17 km de túneles nuevos entre 1998 y 2009, y hay planificados otros 20 km para el futuro inmediato. En estos años se implementaron muchos avances importantes en los campos del diseño y de los procedimientos constructivos de túneles NATM. En este artículo se describen algunos de estos logros, incluyendo la caracterización de suelos para la modelización numérica de túneles NATM, la descripción de los procedimientos constructivos que se han utilizado y algunos comentarios acerca del comportamiento del terreno durante la construcción. Este artículo es una actualización de (Sfriso 2008).

1 INTRODUCTION

The City of Buenos Aires is extending it's metro network as shown in Figure 1. Ongoing projects are: Line A, extended 5 km, Line B, extended 5 km; Line E, extended 2 km; and new Line H, 5 km long. Some 20 km of new Lines F, G, I are scheduled for construction in the near future (SBASE 2006).

Landmarks of new construction procedures are: i) introduction of shotcrete, Line B, 1998 (Fig. 2); ii) so called "belgian" tunneling method, Line H, 2000 (Fig. 3); iii) full face excavation, Line B, 2004 (Fig. 4).

Geotechnical and structural analysis techniques evolved concurrently, from earth-load theory to state of the art computer simulation of construction procedures and calibration of constitutive models via back analysis of monitoring data (Núñez 1996, Sfriso 1996, 1999, 2006, 2007, 2008).

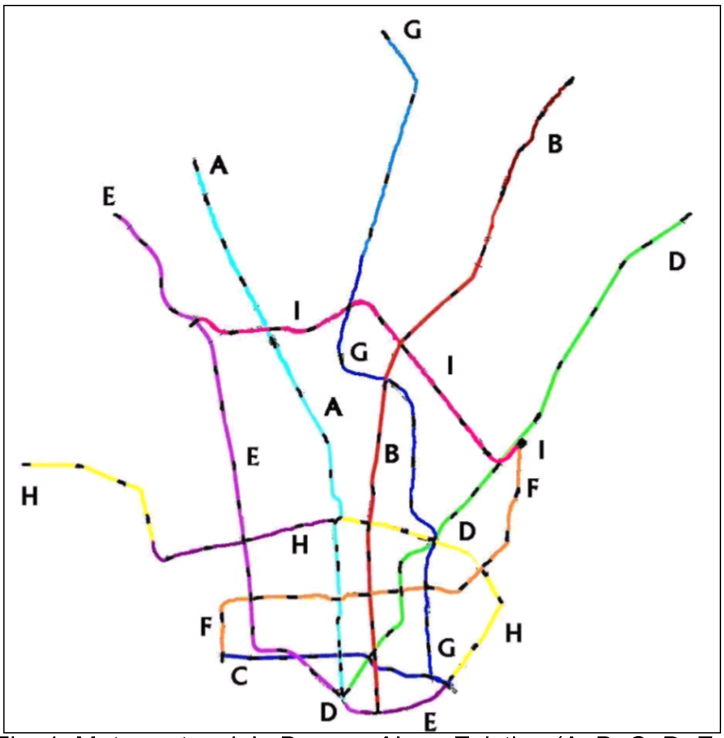

Fig. 1. Metro network in Buenos Aires. Existing (A, B, C, D, E, H) and new projects (F, G, I).

Concurrently, some research work has been carried out on the phisical and mechanical characterization of Buenos Aires soils (Quaglia & Sfriso 2008, Quintela & Sfriso 2008, Sagüés & Sfriso 2008, Sfriso et al 2008). Some results of this research program is briefed below.

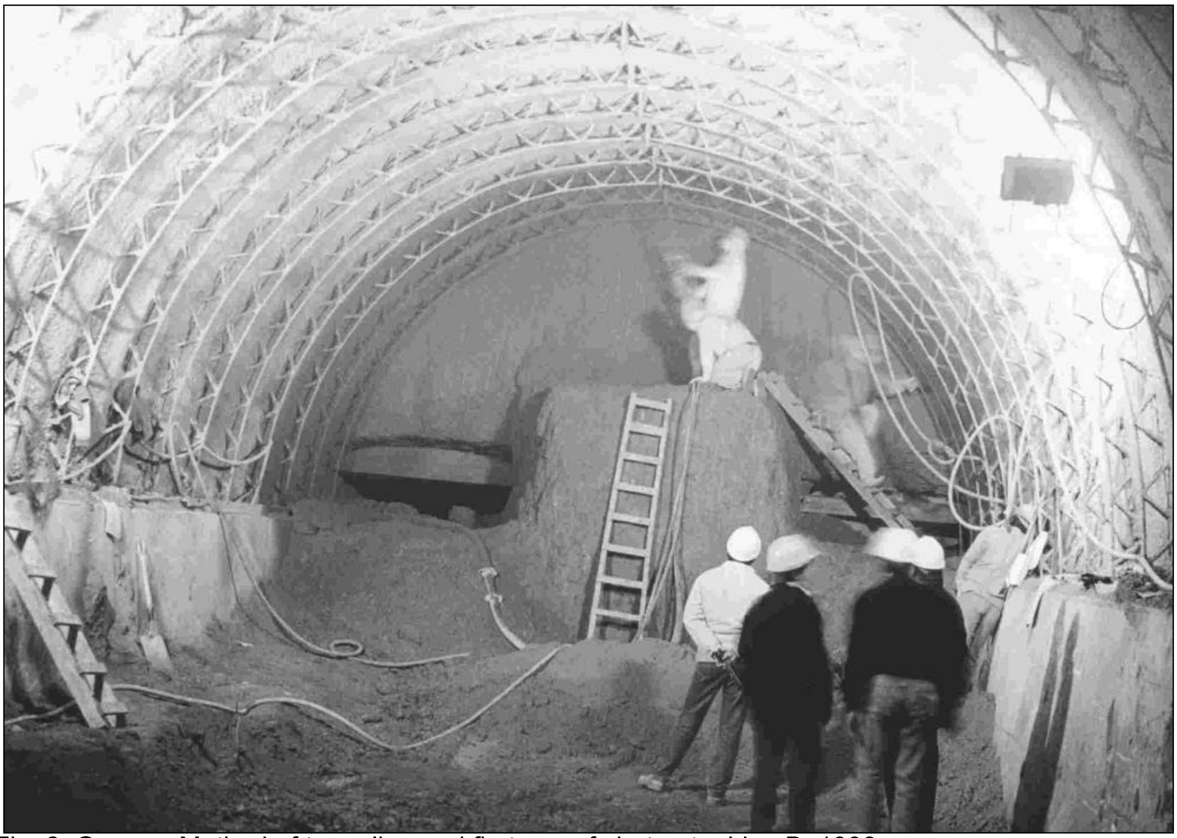

Fig. 2. German Method of tunneling and first use of shotcrete, Line B, 1998.

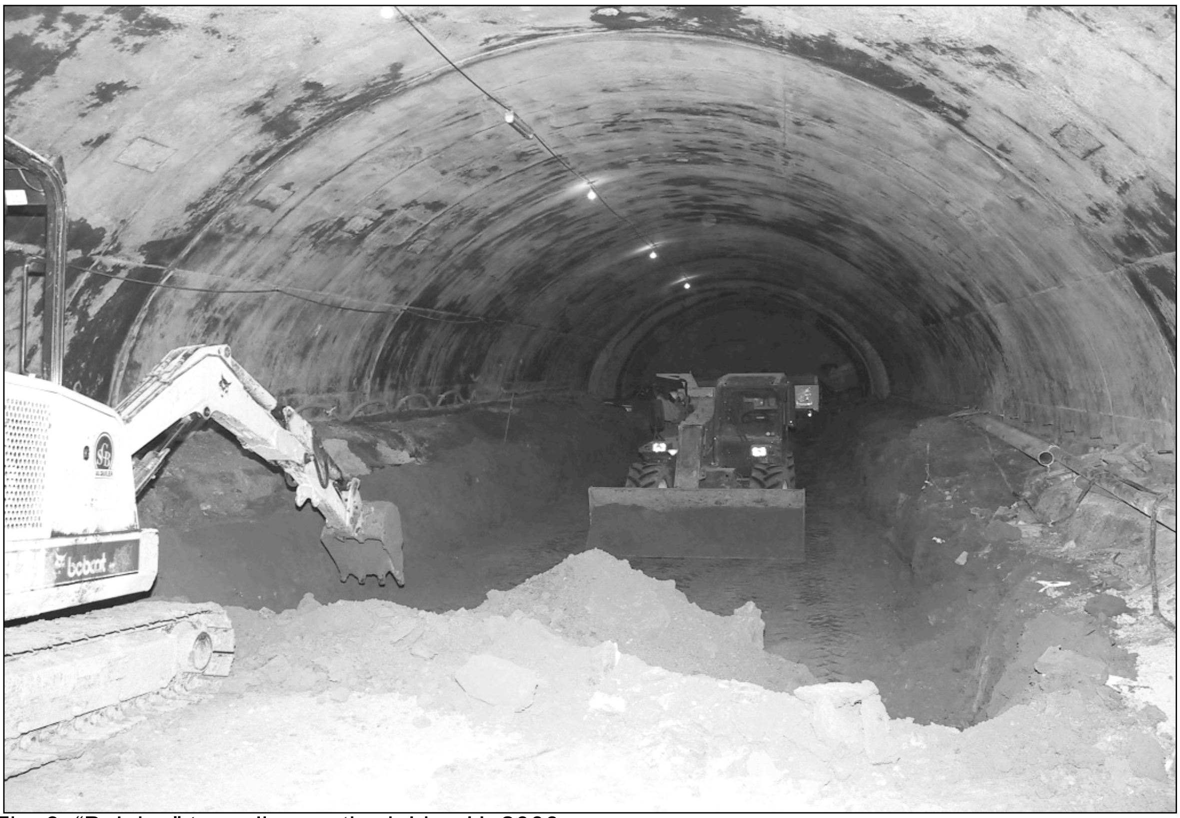

Fig. 3. “Belgian” tunneling method, Line H, 2000.

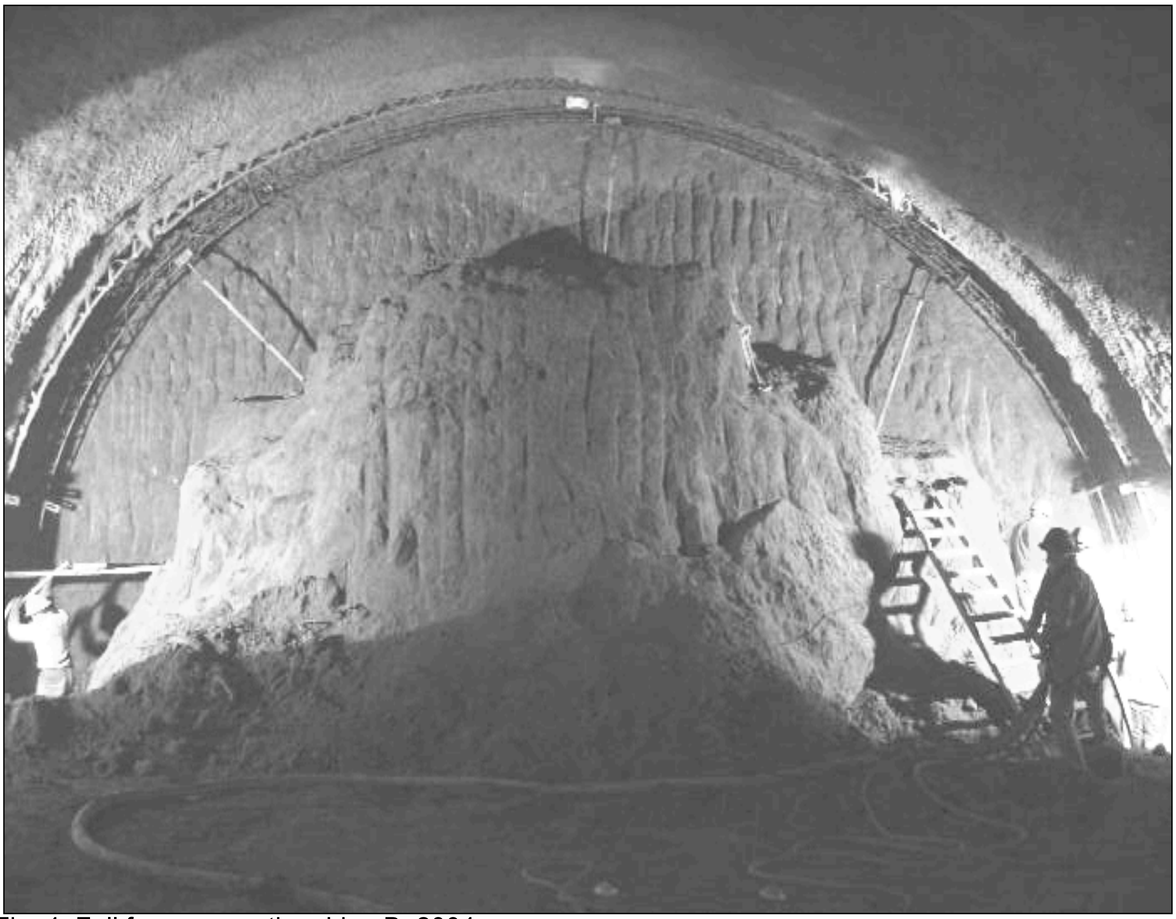

Fig. 4. Full face excavation, Line B, 2004.

2 CHARACTERIZATION OF BUENOS AIRES SOILS FOR TUNNELING

2.1 *Description*

Buenos Aires City soils have been described in other contributions (Bolognesi 1975, Fidalgo 1975, Núñez 1986a, 1986b, 2007). Briefly, the Pampeano formation underlying Buenos Aires is a modified Loess, overconsolidated by dessication and cemented with calcium carbonate in nodule and matrix impregnation forms.
Except for the heaved upper three to six meters, penetration resistance is systematically $N_{SPT} > 20$ with some heavily cemented zones that exhibit very weak rock behavior with $N_{SPT} > 50$ (Nuñez 1986b). Soil mass, where cemented, is systematically fissured, yielding high secondary permeability. Thin non cohesive lenses are ocassionally found interbedded with cemented material. While these lenses are extremely rare in the upper part of the formation, chances to hit them are increasing rapidly, as new tunnels need to be driven deeper due to higher restrictions in underground space.

2.2 *In situ testing*

The most used site investigation technique in Buenos Aires is SPT penetration using a 2 ½" sampler along with standard lab testing and CTUC testing on recovered samples. Some plate load testing (PLT) and Menard pressuremeter testing (PMT) have been recently included as part of the field investigation specifications for metro projects (Sfriso 2006, 2007, 2008). Figure 5 shows one PLT test performed in Line B in 2006 (Sfriso 2006).

Fig. 5. Plate load test, Line B, 2006.

2.3 *Shear strength and stiffness*

Drained triaxial compression tests of undisturbed samples were recently performed at the University of Buenos Aires. Samples were recovered by direct pushing during the excavation of Corrientes Station in Line H, and tested at the very low confining pressure of 20 kPa. Table 1 summarizes some relevant test results (Quaglia & Sfriso 2008).

Table 1. CTC test results, undisturbed samples of Pampeano soil, σ_c=20 kPa.

Test	ω %	c kPa	ϕ_{max} °	E_{50} MPa	E_{ur} MPa
T9	36.0	38.0	45.8	6.7	25.2
T10	40.9	43.3	37.4	12.8	24.6
T11	40.0	30.8	38.7	11.6	22.1
T12	38.8	37.7	36.5	12.4	25.3
T13	36.3	57.2	35.0	13.8	31.0
T14	36.5	22.4	45.1	14.5	32.2
T15	32.9	47.3	36.6	21.5	34.7
T16	40.5	24.4	35.0	9.3	22.7
T17	34.9	1.8	48.4	13.1	28.4
T18	35.7	21.7	42.6	12.3	28.8
T19	37.0	18.8	43.3	5.3	20.8

In table 1, ω is moisture content, c is effective cohesion, ϕ_{max} is peak friction angle, E_{50} is the secant Young modulus at 50% shear mobilization and E_{ur} is the unload-reload young modulus. For the latter, the following calibration for Jambu's expression was proposed (Quaglia & Sfriso 2008).

$$E_{ur} = 120\left(\frac{\sigma_3}{100\text{kPa}}\right)^{0.52} \text{MPa} \tag{1}$$

Parameters presented in Table 1 confirm previous results (e.g. Núñez 1986, 1986b), showing the excellent mechanical properties of Pampeano soils.

2.4 *In situ stresses*

It is accepted (Bolognesi 1991, Núñez 1986a, 1986 b, Sfriso 1999, 2006) that upper Pampeano soils are overconsolidated by dessication to an equivalent pressure 0.8-1.2 MPa. Table 2 lists the assumed in-depth variation of K_0 used for the design of underground structures (Sfriso 2006). These figures have not been actually measured directly but estimated after back-analysis of monitoring data.

Table 2. Assumed in-depth variation of K_0

Depth m		K_0 -
0	to 8/12	0.55 – 0.70
8/12	to 20/24	0.65 – 1.00
20/24	to 30/32	0.55 – 0.80

2.5 *Modulus of subgrade reaction*

Reliable information of static in-situ stiffness is obtained with plate load tests performed in vertical shafts or pilot tunnels (Sfriso 2006). A primary loading modulus of subgrade reaction K and an unload-reload modulus K_{ur} are obtained in PLTs. K_{ur} can be used to estimate a pseudo-elastic Young's modulus

$$E_{PLT} \simeq \tfrac{2}{3} \cdot K_{ur} \cdot B \tag{2}$$

where B is the diameter of the plate. The typical in-depth variation of K, K_{ur} and E_{PLT} for Buenos Aires soils is listed in Table 3 (Sfriso et al 2008).

Table 3. In depth variation of PLT modulus of subgrade reaction and derived Young's modulus

Depth m	K MN/m^3	K_{ur} MN/m^3	E_{PLT} MPa
0 to 8/12	200 - 300	500 - 800	100 - 160
8/12 to 12/14	400 - 600	800 - 1200	160 - 240
12/14 to 20/24	600 - 800	1200 - 1800	240 - 360
20/24 to 30/32	250 - 500	600 - 1400	120 - 280

Values of Young's modulus, as determined in PLT tests and shown in Table 3, are consistent with back-analyses of measured behavior of excavations and foundations, and seem to be reliable stiffness parameters for simple preliminar estimations of ground response to underground works.

3 UNDERGROUND CONSTRUCTION IN THE PAMPEANO FORMATION

The Pampeano formation is very favourable for underground construction due to its high stiffness, reliable compressive strength, rapid drainage and good frictional behavior when drained.

Two particular characteristics of the formation must be accounted for in the design of underground projects: i) the Pampeano formation is fissured and has lenses of quasi-granular behavior, forcing the installation of a primary support close to the face in order to avoid crown overexcavation; and ii) materials drain at a speed comparable to that of the construction.

Due to these factors, the max allowable drift without support is about 2.5 meters. Up to this maximum, the unsupported drift has very little influence on the resulting settlements, as soil behavior remains quasi-elastic (Sfriso 2006, Núñez 2007).

3.1 *Parameters for numerical modelling*

Hyperbolic model (Duncan 1970, Vermeer 1998) has been extensively used for the numerical analysis of underground construction in Buenos Aires soils (Sfriso 1999, 2006, 2007). After eight years of continuous usage and calibration, a set of input parameters for the Plaxis implemmentation of the hyperbolic model (Vermeer 1998) has been found to best represent the observed behavior of tunnels, caverns and open pit excavations. This set is listed in Table 4. Stress-strain relationship of the HSM model is reproduced in Equations 3a to 3d.

Table 4. Material parameters used for numerical simulations

	Fill min-max	0-8/12 min-max	8/12-20/24 min-max	>20/24 min-max
c_u (KPa)	20-50	50-100	110-220	40-120
ϕ_u (°)	8-15	10-20	5-20	0-5
c' (KPa)	0-5	10-25	25-50	15-30
ϕ' (°)	28-30	28-31	30-34	28-31
ψ(°)	0	0-3	0-6	0-3
E_{50}^r (MPa)	10-20	60-100	75-150	60-100
E_{ur}^r (MPa)	25-50	150-250	180-300	140-220
m (-)	0	0	0	0
ν (-)	0.20	0.20-0.30	0.20-0.30	0.25-0.35
R_f (-)	0.85-0.90	0.80-0.90	0.80-0.90	0.80-0.90

$$\sigma_1 - \sigma_3 = \begin{cases} \text{load: } 2E_{50}\left(1 - R_f \dfrac{\sigma_1 - \sigma_3}{\sigma_3 N_\phi + 2c\sqrt{N_\phi}}\right)\varepsilon_1 \\ \text{unload: } E_{ur}\varepsilon_1 \end{cases} \tag{3a}$$

$$N_\phi = \tan^2\left[\frac{\pi}{4} + \frac{\phi}{2}\right] \tag{3b}$$

$$E_{ur} = E_{ur}^r \left(\frac{\sigma_3 + c\cot[\phi]}{c\cot[\phi] + p_{atm}}\right)^m \tag{3c}$$

$$E_{50} = E_{50}^r \left(\frac{\sigma_3 + c\cot[\phi]}{c\cot[\phi] + p_{atm}}\right)^m \tag{3d}$$

In Equations 1a to 1d and Table 3, σ_1 and σ_3 are the major and minor principal stresses, ε_1 is the major principal strain, c is either undrained cohesion c_u or drained cohesion c', ϕ is either undrained friction angle ϕ_u or drained friction angle ϕ', ψ is dilatancy angle, E_{50}^r and E_{ur}^r are reference loading/unloading Young's modulus, m is stiffness exponent, ν is Poisson's ratio and R_f is the failure ratio.

The apparent inconsistency between $m=0.52$ eqn. (1) and $m=0$ in Table 4 can be explained with the aid of eqns. (3c) and (3d), which show that in the HSM model stiffness is affected by cohesion. This feature of the model, which obscures its calibration process and usage, is deactivated by setting $m=0$.

4 CONSTRUCTION PROCEDURES

4.1 *Tunnels*

Construction procedures evolved from german method (Fig. 2) to "belgian" method (Fig. 3) and have probably reached an optimal stage with full face excavation (Fig. 4). These three methods, as adopted in Buenos Aires practice, are briefly outlined as follows:

- German method of tunneling: Long, straight side walls with continuous footings are first casted in pilot tunnels. A circular crown is then excavated in slices 1.0 /

1.5 m long and supported with shotcrete & lightweight lattice girders. A cast-in-place secondary lining is afterwards added to the crown. Finally, the invert is made with cast-in-place concrete. Sometimes, girders are set to bridge over the primary and secondary linings, although it has been found that this procedure might produce enhanced cracking in the secondary lining (See Fig. 2).

- “Belgian” method of tunneling: This method is a (minor) modification of the Madrid method of tunneling. The upper half of the tunnel is excavated and supported using standard NATM techniques including shotcrete and lattice girders and a secondary plain concrete lining is casted short afterwards. After the cast-in-place concrete is cured, the bench is excavated and side walls are excavated and casted in a “batache” (i.e. tooth-like) configuration.Finally, a cast-in place invert is built.
- Full face excavation: This is a standard NATM full face tunneling method with open invert. The tunnel is excavated full-face, supported by shotcrete & lattice girders, and resting on temporary continuous footing. A secondary unreinforced cast-in-place lining is placed afterwards. In some cases, the secondary lining is formed by an unreinforced, thick shotcrete layer.

Figure 6 shows the cross section of a typical two lane, full face tunnel, as used in Line B and afterwards in lines H, A, E. A 15 cm unreinforced shotcrete layer and 1.0 m spaced lightweight lattice girders account for the primary support of the tunnel, later supplemented with 30 – 40 cm of cast-in-place unreinforced concrete.

The Metro authority requires that tunnels remain dry during operation, thus rendering cast-in place secondary lining as the cheapest option, when compared to membrane barriers and secondary shotcrete lining. Some full face, robot placed all-shotcrete sections with impermeabilization barriers have been successfully tested in lines H and B and is now used in lines A and B. Line E, starting excavation by the time this article is delivered, is planned to employ a PVC membrane between a primary shotcrete lining and a cast-in-place secondary lining.

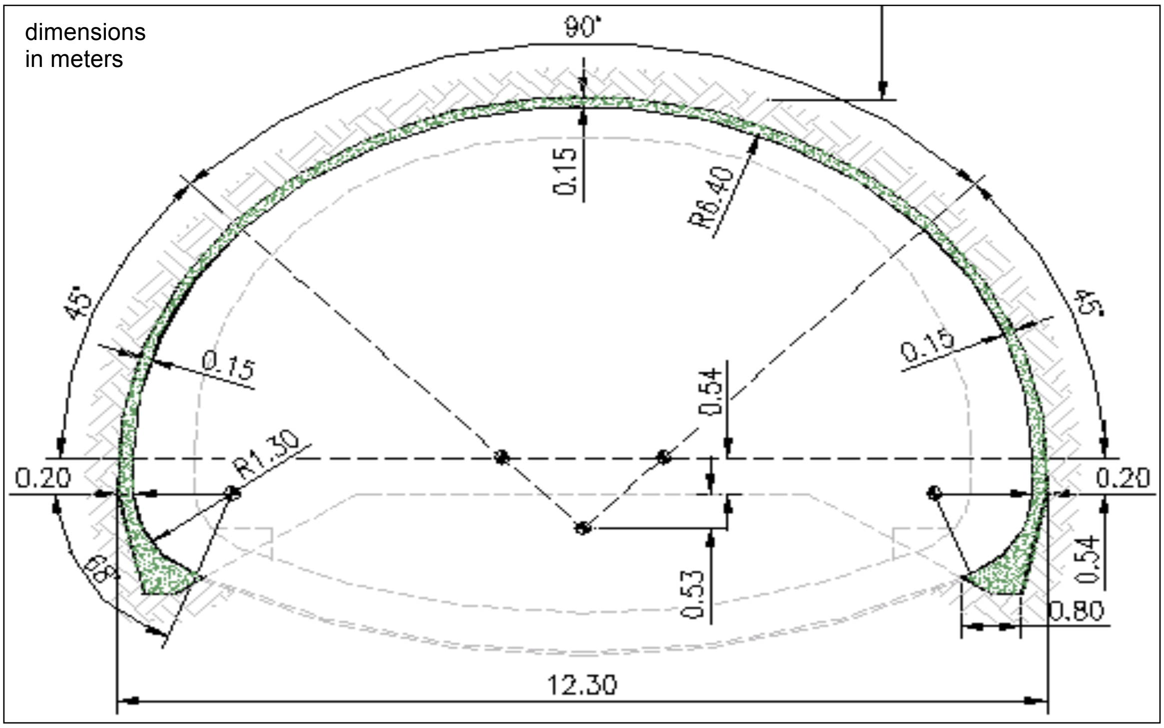

Fig. 6. Cross section of a typical two lane, full face tunnel, lines B and H.

No closure of the structural ring is needed for stability, and therefore advance rates of 2.5 m – 3.5 m per 12 hr shift are consistently achieved. After the tunnel is excavated, a cast in place invert is placed in 5 m – 6 m segments, allowing for the placement of the secondary lining in single poured 5 m segments. Figure 7 shows a tunnel after placement of the invert, while Figure 8 shows the formwork being driven into the tunnel.

Fig. 7. Tunnel after placement of the invert, Line B.

Fig. 8. Formwork used to cast the secondary lining, Line B.

4.2 *Stations and caverns*

Underground caverns have been built using many techniques including: i) cut&cover slab-on-piles; ii) underground excavated main cavern & open pit excavated upper hall; and, iii) underground excavated main cavern & upper hall.

The flagship of underground construction is Corrientes Station (Fig. 9). It is an underground cavern 14.1 m high, 18.9 m wide and 135 m long (Fig. 10). On top of the main cavern, a 6 m high access hall was excavated after completion of the secondary lining of the main cavern.

Fig. 9. Corrientes Station, Line H. Full face excavation.

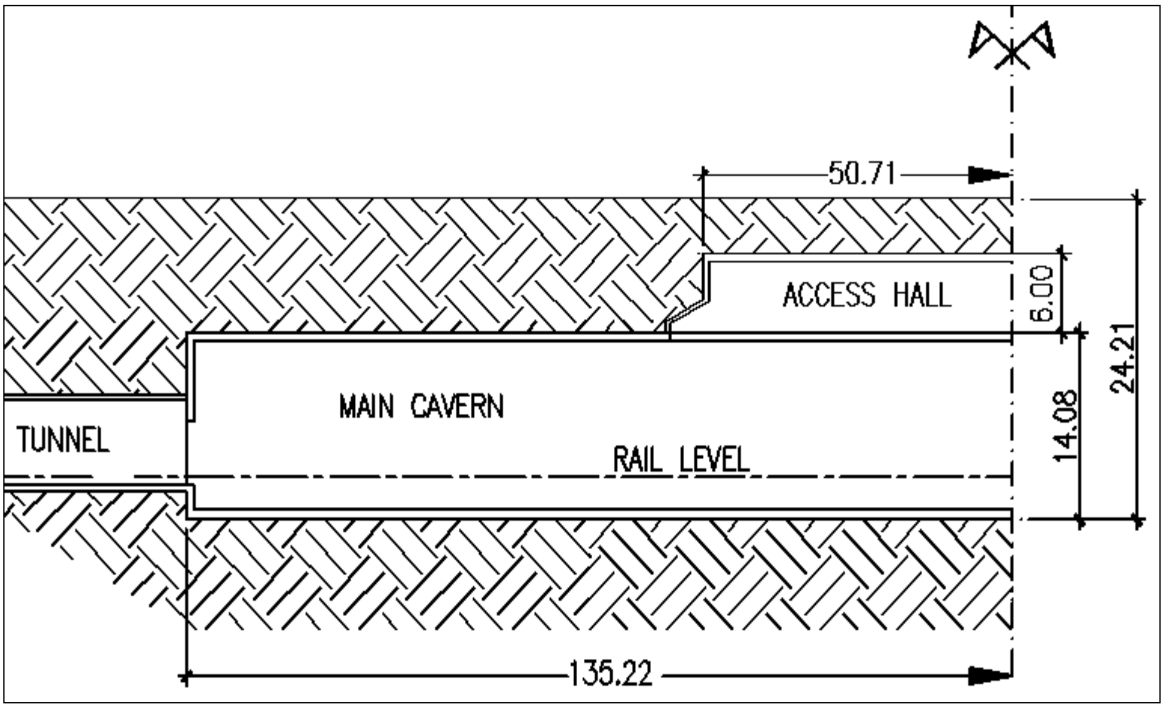

Fig. 10. Longitudinal sketch of Corrientes Station, Line H. Dimensions in meters.

The primary lining of Corrientes Station is formed by 20 – 40 cm mesh reinforced shotcrete placed in two layers, and 1.0 m spaced lightweight lattice girders. Full-face excavation is accomplished via a series of four benches, each one 5 m long. Two excavators are permanently set at the two top and two bottom benches, respectively. The bottom bench excavator alternatively lies on soil or on top of the cast-in-place invert, included into the primary support lining to reduce costs and time schedule. This construction procedure allowed for a reduction of the duration of the excavation of 50% to six months, a major achievement for a NATM station excavated in soil. Fig. 9 briefs this achievement: the crown, side walls, invert, secondary lining and interior decks are all being placed at the same time.

5 DESIGN PROCEDURES

5.1 *Primary lining*

The preliminary design of the primary lining is largely based on experience. By the time the tunnel shown in Fig. 2 was being analyzed, a simplified design method was developed to estimate forces acting in the crown of the primary support of circular sections (Núñez 1996). The expressions are

$$N=\frac{1}{2}\left(K_0+\frac{2}{3}\frac{1-K_0}{1+a}\right)\frac{2D-A}{3D}p_vD \tag{4a}$$

$$M=\frac{1-K_0}{16}\frac{a}{1+a}\frac{2D-A}{3D}p_vD^2 \tag{4b}$$

$$a=16\frac{E_r\left(1-\nu^2\right)}{E\left(1-\nu_r^2\right)}\left(\frac{e}{D}\right)^3 \tag{4c}$$

where *N* is the normal force at crown, *M* is the flexure moment at crown, p_v is the vertical pressure on the crown, *D* is the tunnel diameter, *A* is the unsupported drift, *E, ν* are the elastic parameters of the soil mass and E_r, ν_r are the elastic parameters of the support system. Structural forces obtained with equations 4a, 4b and 4c compare within 10% - 15% with those computed using the more involved procedure by Einstein & Schwarz (Einstein 1979).

5.2 *Simulation of construction procedures*

Construction procedures are simulated using 3D elastoplastic models that allow for the estimation of surface settlements, the computation of face stability and the determination of structural forces acting on the primary lining. For simple tunnels where numerical methods can be compared with analytical computations, it is found that structural forces computed with 3D FEM are some 20% lower than those obtained with Eqns. 4a, 4b, 4c. These equations, when applied to the tunnel shown in Fig. 6, resulted in M=0.55 kNm/m and N=416 kN/m. 3D numerical models yielded M=0.47 kNm/m and N=415 kN/m. (Sfriso 2006, Núñez 2007).

5.3 *Secondary lining*

Metro authority requires that the secondary lining be designed using earth-load procedures and beam on springs analyses. Both primary lining and the effect of construction procedures are disregarded in the design of the secondary lining. A change in this requirement is currently being considered for some service tunnels.

6 GROUND BEHAVIOR

Ground behavior has been largely elastic for all construction procedures and underground structures built so far. Disturbance to surrounding structures and facilities has always been minimal, and surface settlements in the range 2 mm – 8 mm for tunnels and 4 mm – 15 mm for underground caverns have been observed for all construction procedures and soil covers. While this is a desirable behavior from the point of view of construction and safety, it also means that uncertainty of the predictions remain high, because it is unknown how safe the construction procedures really are.

A numerical excercise has been performed to compare the construction procedures for safety and impact to surroundings. A tunnel section 10 m wide, 8 m high with a soil cover of 5 m was used, and the low side parameters listed in Table 4 were adopted. The results are listed in Table 5 (Sfriso 2006). It can be noticed that the german method proved to be the least safe construction method, due to the low safety of the unsupported access tunnels excavated to build the side walls (Fig. 11).

Table 5. Numerical comparison between construction procedures for tunnels.

	German	Belgian	Full face
Max. surface settlement, undrained parameters (mm)	4.9	4.3	4.6
Max. surface settlement, drained parameters (mm)	7.4	5.3	6.7
Max. angular distorsion, undrained parameters (10^{-3})	0.26	0.22	0.18
Max. angular distorsion, drained parameters (10^{-3})	0.30	0.26	0.26
Factor of safety, undrained parameters (-)	2.6	>7	4.7

Fig. 11. Unsupported pilot tunnel for side walls, german method of tunnelling.

At Corrientes Station, the observed surface settlement 5 mm – 8 mm is much lower than the predicted value of 20 mm. After interpretation of the monitoring data, it has been concluded that the unload Young's modulus of Pampeano soils is lower than originally estimated, and that the deposit elastic rebound is partly responsible for the small settlements observed. Being the first large closed ring structure ever built in Buenos Aires, Corrientes Station is the first opportunity to properly calibrate the unloading Young's modulus and the effect of soil rebound.

7 REMAINING CHALLENGES

In a previous contribution (Sfriso 2008), it was commented that (by 2007) some remaining challenges need to be addressed. These are: i) the implemmentation of a reliable procedure to measure K_0; ii) the abandonment of cast-in-place concrete and "dry" tunnels; iii) the use of robot-placed, fiber reinforced shotcrete; iv) the implemmentation of more advanced topographic guiding systems; v) optimizations in the usage of lattice girders; and vi) better control of ground water during construction.

Little has been advanced in these fields in these last two years, except for the use of robots which has been enforced by the metro authority. Water control is, by far and large, the major challenge to be addressed, as deeper tunnels must deal with higher water inflow and real chances to hit water bearing sandy seams. Line E, starting excavation by the time this paper is delivered, shall probably be the next opportunity to advance in this field.

8 CONCLUSIONS

17 km of metro tunnels have been excavated in Buenos Aires in the period 1998-2009. Construction procedures now include shotcrete and full face excavation both in tunnels and caverns, while design procedures include state of the art numerical simulation of construction processes. Best fit parameters for the constitutive models used were introduced and some observed features of soil behavior have been described.

Corrientes Station is the latest improvement to construction procedures used in Buenos Aires metro tunnelling so far. In short, an underground cavern 135 m long was completely excavated in six months with surface settlements lower than 10 mm and minimal disturbance to surroundings.

Water control during construction is the major remaining challenge for NATM tunnels in Buenos Aires, and the one requiring most inmediate attention.

9 ACKNOWLEDGEMENTS

The companies involved in these projects, Techint, Dycasa, Roggio and SBASE, shared their impressions, opinions and experience with the writer, allowing him to learn many things from them. Eduardo Núñez taught the writer the basics, the involved theories and the tricks of tunnelling, carefully supervised the writer's early works, and remains as the day-by-day writer's source for expert advice.

REFERENCES

Bolognesi, A. 1975. Compresibilidad de los suelos de la Formación Pampeano. V PCSMFE, Argentina, V: 255-302.
Bolognesi, A. and Vardé, O. 1991. Subterráneos en Buenos Aires. IX PCSMFE, Chile, III:1329-1350.
Duncan, J. and Chang, C. 1970. Nonlinear analysis of stress and strain in soils. JSMFD, ASCE, 96, SM5, 1629-1653.
Einstein, H. and Schwartz, C. 1979. Simplified analysis for tunnel supports. JGED, ASCE, 105, 4, 499-518.
Fidalgo, F., De Francesco, F. and Pascual, R. 1975. Geología superficial de la llanura Bonaerense". VI Arg. Geol. Conf, 110 - 147.
Núñez, E. 1986a. Panel Report: Geotechnical conditions in Buenos Aires City. V Conf. IAEG, 2623-2630.
Núñez, E. and Mucucci, C. 1986b. Cemented preconsolidated soils as very weak rocks. V Conf. IAEG, 403-410.
Núñez, E. 1996. Túneles de sección circular en la formación pampeano. bull. SAMS, 29, 1-15.
Núñez, E. 2007. Casagrande lecture: Uncertainties and Approximations in Geotechnics. XIII PCSMGE, Venezuela. In press.
Quaglia G. and Sfriso, A. 2008. Cohesión efectiva y rigidez inicial del Pampeano inalterado. XIX CAMSIG, La Plata, CD-ROM.
Quintela M. and Sfriso A. 2008. Compresión edométrica de muestras inalteradas del Pampeano. XIX CAMSIG, La Plata, CD-ROM.
Sagüés P. and Sfriso A. 2008. Rigidez a baja deformación en el Pampeano compactado. XIX CAMSIG, La Plata, CD-ROM.
SBASE 2006. Institutional web page. www.sbase.com.ar.
Sfriso, A. 1996. Túneles de sección circular en la formación Pampeano - comentario. bull. SAMS, No. 29, 16-19.
Sfriso, A. 1999. Tunnels in Buenos Aires: Application of numerical methods to the structural design of linings. XI PCSMGE, Brasil, 637-642.
Sfriso, A. 2006. Algunos procedimientos constructivos para la ejecución de túneles urbanos. XIII CAMSIG, Argentina, 1-17.
Sfriso, A. 2007. Procedimiento constructivo de la Estación Corrientes del Subterráneo de Buenos Aires, Argentina. VI Cong Chil Geot, Valparaíso, d6:1-13.
Sfriso, A. 2008. Metro tunnels in Buenos Aires: Design and construction procedures 1998 – 2007. 6th Intl Symp Geot Aspects Underg Constr Soft Ground, Shanghai, 335-341.
Sfriso, A., Sagüés, P., Quaglia, G., Quintela, M., Ledesma, O. 2008. Small-strain stiffness of the Pampeano Formation. IS-Atlanta 2008, IV Intl Symp Def Char Geomat I:237-244.
Vermeer, P. 1998. Plaxis Users Manual. Balkema, Rotterdam. Ne, 577 p.