Metro tunnels in Buenos Aires: Design and construction procedures 1998 - 2007

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ABSTRACT: The City of Buenos Aires, Argentina, is expanding it's metro network. Some 13 km of new tunnels have been excavated since 1998 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. 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.

1 INTRODUCTION

The City of Buenos Aires is extending it's metro network as shown in Figure 1. On-going projects are: Line A, extended 4 km, Line B, extended 4 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).

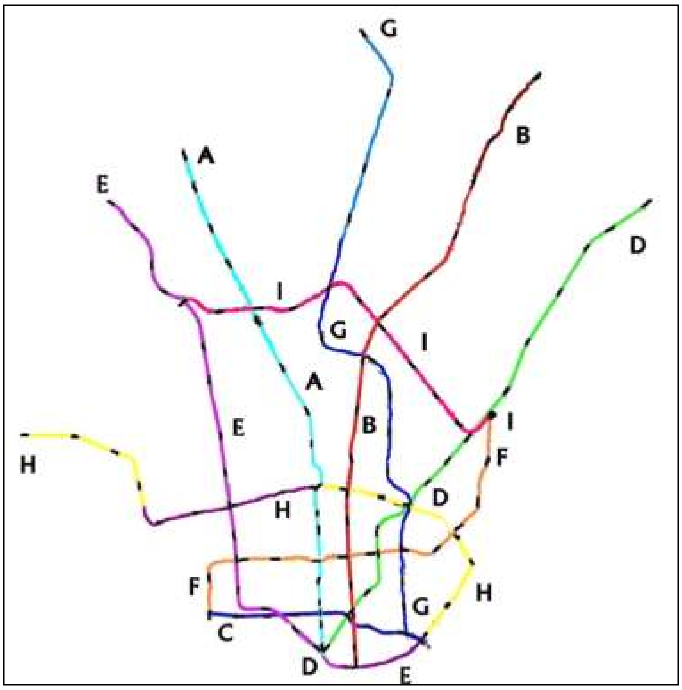

Fig. 1. Metro network in Buenos Aires. Existing (A, B, C, D, E, H) and new projects (F, G, I).

Landmarks of new construction procedures are: i) introduction of shotcrete, Line B, 1998 (Fig. 2); ii) 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).

Fig. 2. First use of shotcrete, Line B.

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

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

2 CHARACTERIZATION OF BUENOS AIRES SOILS FOR TUNNELING

Buenos Aires City soils have been described in other contributions (Bolognesi 1975, Fidalgo 1975, Núñez 1986a, 1986b). 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).

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 and Menard pressuremeter testing have been recently included as part of the field investigation specifications for metro projects (Sfriso 2006).

2.1 *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).

2.2 *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 1 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 1. 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

2.3 *Modulus of subgrade reaction*

The most reliable information of in-situ stiffness is retrieved via plate loading tests performed in vertical shafts or pilot tunnels. Typical in-depth variation of the modulus of subgrade reaction, as determined by PLT, is listed in Table 2.

Table 2. PLT Modulus of subgrade reaction

Depth m		Primary loading MN/m^3	Un-Reloading MN/m^3
0	to 8/12	200 - 300	500 - 800
8/12	to 12/14	400 - 600	800 - 1200
12/14	to 20/24	600 - 800	1200 - 1800
20/24	to 30/32	250 - 500	600 - 1400

2.4 *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). 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 3. Stress-strain relationship of the HSM model is reproduced in Equations 1a to 1d.

Table 3. Material parameters used for numerical simulations

	Fill		0-8/12		8/12-20/24		>20/24	
	min	max	min	max	min	max	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	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	0	0	0	0
ν (-)	0.20	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

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

$$N_\phi = \tan^2\left[\frac{\pi}{4} + \frac{\phi}{2}\right] \tag{1b}$$

$$E_{ur} = E_{ur}^r \left(\frac{\sigma_3 + c\cot[\phi]}{c\cot[\phi] + p_{atm}}\right)^m \tag{1c}$$

$$E_{50} = E_{50}^r \left(\frac{\sigma_3 + c\cot[\phi]}{c\cot[\phi] + p_{atm}}\right)^m \tag{1d}$$

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.

3 CONSTRUCTION PROCEDURES

3.1 *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). Figure 5 shows the cross section of a typical two lane, full face tunnel, as used in Lines B and H. 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, all-shotcrete sections with impermeabilization barriers have been successfully built in lines H and B.

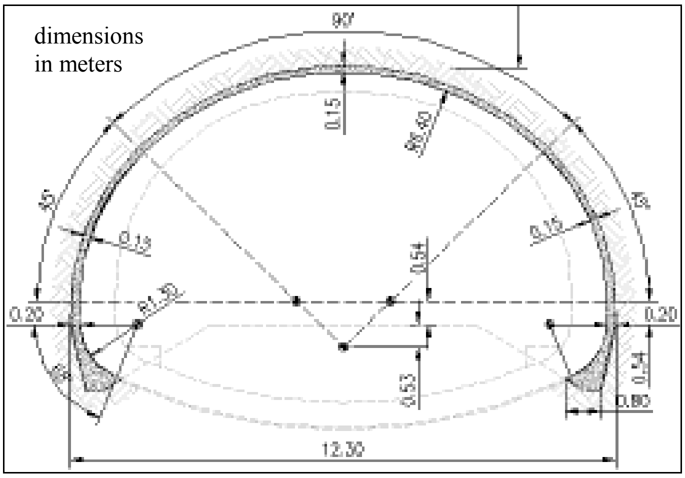

Fig. 5. Cross section of a typical two lane, full face tunnel, lines B and H.

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 6 shows a tunnel after placement of the invert, while Figure 7 shows the formwork being driven into the tunnel.


Fig. 6. Tunnel after placement of the invert, Line B.


Fig. 7. Formwork used to cast the secondary lining, Line B.

3.2 *Underground stations*

Underground caverns for metro stations 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.

As per 2007, three stations are in excavation stage. The first two are Echeverría Station (Fig. 8) and Villa Urquiza Station, Line B, where the german tunnelling method was used. The third station is Corrientes Station, Line H, where full face excavation is being used (Fig. 9).

Corrientes Station is the latest and more challenging improvement to construction procedures used in Buenos Aires metro tunnelling so far. 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 shall be excavated full-face after completion of the secondary lining of the main cavern.

Fig. 8. Echeverría Station, Line B. German tunnelling method.

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

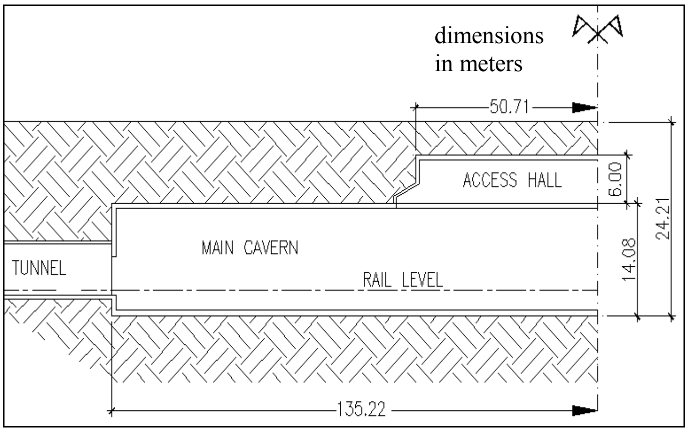

Fig. 10. Longitudinal sketch of Corrientes Station, Line H.

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. The construction procedure is shown in Figure 11. 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. While excavation of Echeverría Station took some 14 months, it shall took some 6 months to complete Corrientes Station, with advance rates 1.0 m per day.

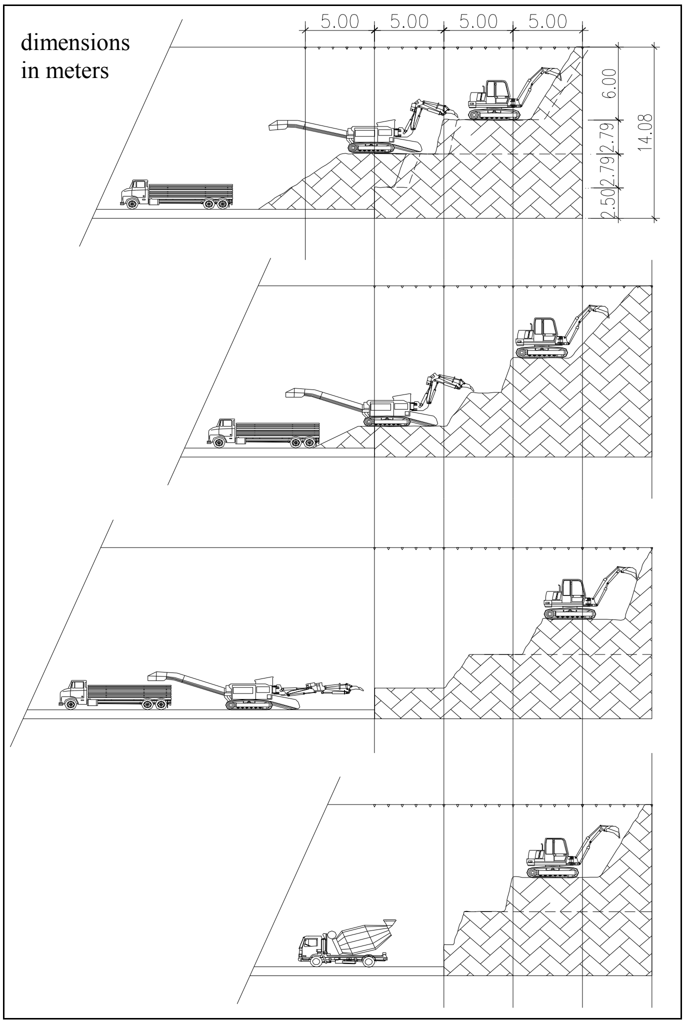

Fig. 11. Construction procedure for Corrientes Station, Line H.

4 DESIGN PROCEDURES

4.1 *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

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

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

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

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 2a, 2b and 2c compare within 10% - 15% with those computed using the more involved procedure by Einstein & Schwarz (Einstein 1979).

4.2 *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. Structural forces computed with 3D FEM are some 20% lower than those obtained with Eqns. 2a, 2b, 2c. These equations, when applied to the tunnel shown in Fig. 5, 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).

4.3 *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.

5 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 3 were adopted. The results are listed in Table 4 (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. 12).

Table 4. 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. 12. Unsupported pilot tunnel for side walls, german method of tunnelling.

Echeverría Station and Corrientes Station have proven enlightening experiences for the purpose of checking predictions. For safety considerations, it was decided that Echeverría Station be excavated using the german method, and a max. surface settlement of 15 mm was predicted. After a series of small access tunnels were excavated to improve the construction schedule, and before the main cavern excavation was started, a surprisingly high 10 mm settlement was observed at surface.

When the numerical model was re-run with the access tunnels included but without any change in material parameters, the observed surface settlement could be reproduced. It turned out that the unconfinement produced by the too many intersecting small tunnels was responsible for the undesired behavior and yielded a temporarily unsafe condition. It was dediced that the abandoned access tunnels be supported by struts (Fig. 13).

Fig. 13. Strut supported access tunnels, Echeverría Station, Line B.

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 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.

6 REMAINING CHALLENGES

The advancement in design and construction procedures is an endless activity. Despite the efficient methods actually in use, some remaining challenges need to be addressed in the near future. 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.

7 CONCLUSIONS

13 km of metro tunnels have been excavated in Buenos Aires in the period 1998-2007. Construction procedures in 2007 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.

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

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.

Corrientes Station is the latest improvement to construction procedures used in Buenos Aires metro tunnelling so far. An underground cavern 135 m long shall be completely excavated in six months with surface settlements less than 10 mm and minimal disturbance to surroundings.

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