# Subway station retaining walls: case-histories in soft and hard soils

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**ABSTRACT**. In the last twenty years, sixteen pile-supported metro stations have been built in Buenos Aires in a wide range of geotechnical conditions. After an initial design of the construction procedures including partial open-trench (in two cases), all subsequent fourteen stations were excavated employing cut&cover techniques. In all cases, vertical bored piles were employed both to support the lateral ground pressure and the loads acting on the roof slab. This paper revisits the geotechnical conditions in Buenos Aires City, describes the procedures employed for the design and numerical analysis of pile-supported excavations and presents the behavior of five recent cases located in widely different geotechnical profiles. The paper ends with a summary of lessons learned which are relevant both for design and construction.

## 1 INTRODUCTION

Buenos Aires metro network opened in 1913, being the first one in the southern hemisphere. Figure 1 shows the excavation procedure for Line A in 1911 (SBASE 2017). Expansion was fast until the 40's, when it halted for until it was resumed in the late 90's. The network is formed by six lines, 55km of tunnels and 86 stations, and complemented by two surface lines (Figure 1).

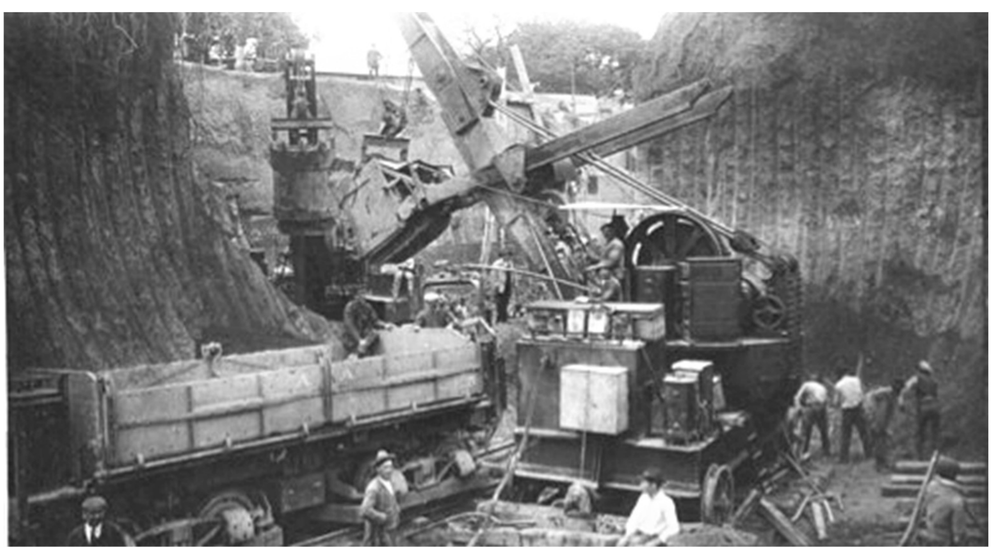


Figure 1. Excavation of Line A in 1911 (SBASE 2017).

Projects completed as of april 2017 since 1997 are the extensions of A Line (5 km), B Line (5 km), and E Line (2 km); and the construction of the new 5 km long H Line. Line H is now being further extended on both ends: C2 northbound extension consists in four new stations and 2km long double-track NATM tunnel, while the southbound A0/A1 extension comprises one station, two large underground maintenance facilities and 1.5km long tunnel (includes 350 long railyard tunnel located south of future Sáenz Station). Due to very poor geotechnical conditions, construction of the latter will require the use of diaphragm walls and secant piles.

## 2 GEOTECHNICAL CONDITIONS IN BUENOS AIRES CITY

### 2.1 Geotechnical setting

The description below briefes relevant published papers addressing the characteristics and behavior of the Pampeano Formation (Bolognesi and Moretto 1957; 1961, Trevisán and Mauriño 1963, Moretto 1972, Bolognesi 1975, Fidalgo et al 1975, Núñez 1986, Núñez and Micucci 1986, Bolognesi and Vardé 1991, Sfriso 2006, Codevilla & Sfriso 2010; 2011). Cites are omitted from the text below for ease of reading.

Buenos Aires City can be divided in two different areas: upper plains, and lowlands located on the margins of the Riachuelo river and River Plate. The Pampeano formation underlying the center of the city is a modified Loess, overconsolidated by dessication and cemented with calcium carbonate in nodule and matrix impregnation forms. The Postpampeano formation that lies on the river margins is a holocene fluvial deposit of normally consolidated or slightly overconsolidated soft silts and clays and loose silty sands. Man-made fills have covered most of the margin of the River Plate (Figure 2).

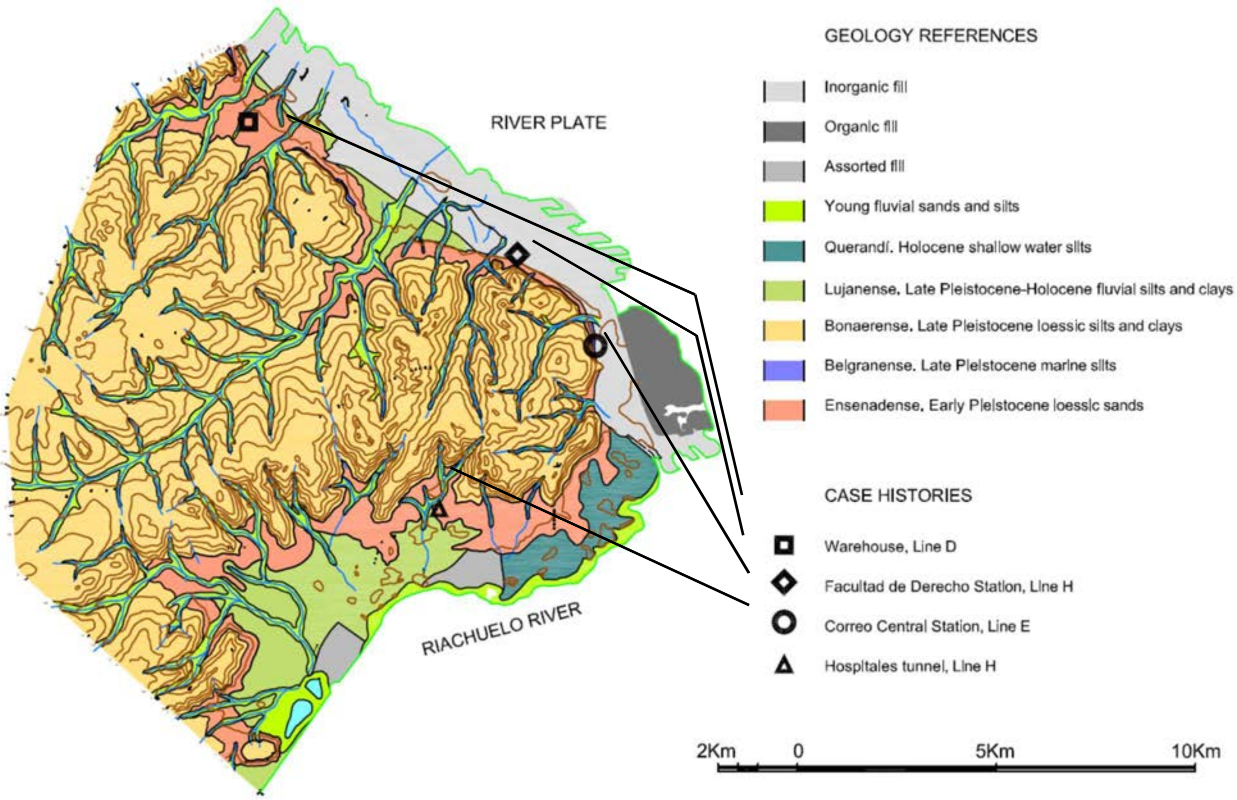



Figure 2. Buenos Aires Geological map; location of projects described in this paper.

The stratigraphic profile can be summarized as follows (Figure 2 and Figure 3): i) man-made fills (base depth: 1m|3m); ii) Upper Pampeano (1m/3m to 8m/12m): medium plasticity stiff silts and clays, eolic deposition, calcareous nodules in a poorly cemented matrix; iii) Middle Pampeano (8m/12m to 25m/30m): medium plasticity very stiff silts and clays, eolic deposition, strong oxide cementation and calcareous nodules. iv) Lower Pampeano (25m/30m to 35m/40m): medium and high plasticity clays, fluvial deposition, poorly cemented matrix, no fissures; v) Puelche (35m/40m to 48m/53m): clean to silty fine to medium grained dense to very dense pliocene sands; and vi) older soils with no effect on excavations and tunnels for metro projects.

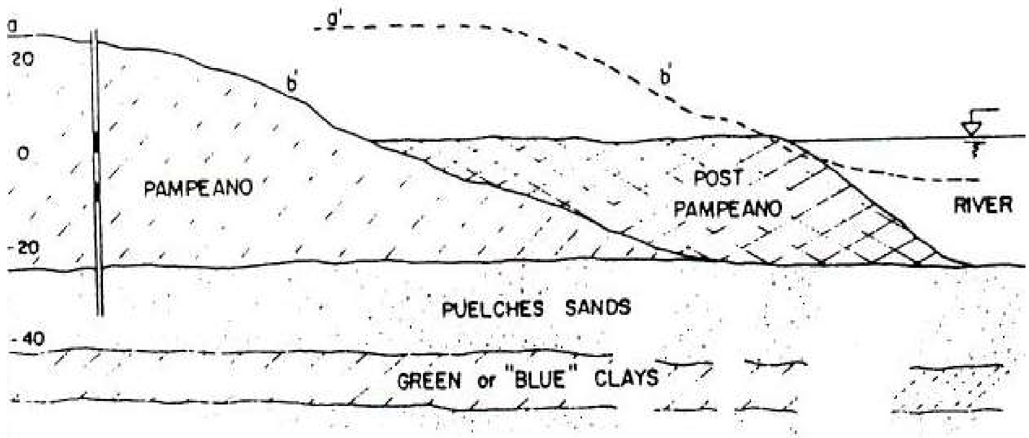



Figure 3. Stratigraphic profile of Buenos Aires City. The project described in this paper lie in the contact between “Pampeano” and “Postpampeano” formations (Núñez 1986).

## 2.2 Origin of the Pampeano Formation

Silt and clay particles were transported by wind and water, and deposited under calm water, forming interbedded layers of fine materials of varying plasticity. The predominant minerals are caolinite, illite, amorphous silica, plagioclases and volcanic ash particles, deposited in quite heterogenous concentrations. Soils plot close to the A line in the Casagrande chart, and classify either as ML, MH, CL or CH.

The reduction of the sea level associated to the last ice age lowered the water table and started a consolidation process. Intense droughts reduced the water content of the deposit that then became unsaturated and heavily overconsolidated by dessication. The preconsolidation stress is induced by dessication is larger than 1.0MPa, yielding an overconsolidation ratio in the range $3.5 < OCR < 4.0$.

Calcium carbonate and magnesium oxides precipitated during the drying process, bonding the particles together. Three degrees of carbonate cementation can be distinguished: i) nodules isolated in a non-cemented matrix; ii) an intermediate cemented matrix with strongly cemented nodules; and iii) the locally called “Tosca”, a cemented matrix embedding very stiff calcium carbonate inclusions, the edges of these inclusions being readily distinguishable from the surrounding matrix.

Where the material was directly exposed to the arid climate, a pattern of fissures developed in the soil mass. Some of these fissures were afterwards sealed with calcium carbonate and other salts transported by rain infiltration. The water table recovery and subsequent fluctuations saturated most of the soil mass. Restrained heave resulted in high lateral stresses and an at-rest coefficient of lateral pressure in the range $0.7 < K_0 < 1.0$.

Particular features of the formation are: i) dessication fissures induce a high secondary permeability; ii) thin layers of non-cohesive loamy sands are found at depth 20m and below; iii) close to the bottom of the formation and right on top of pliocene clean sands, a poorly cemented, sub-stratum of greenish clays acts as a hydraulic seal.

### 2.3 Strength and stiffness

The Pampeano formation is very favorable for deep excavations and underground construction due to its high stiffness, compressive strength, rapid drainage and good frictional behavior when drained. Except for the upper three to six meters, standard penetration resistance is systematically $N_{60} > 20$ with some heavily cemented zones that exhibit very soft rock behavior with $N_{60} \gg 50$ (Figure 4). The undrained unconfined compression strength of saturated, undisturbed samples depends on cementation and ranges from $500\text{kPa} < q_u < 2000\text{kPa}$.

Effective friction angles are rather independent of cementation and are low bounded by the high-confinement friction angle $\phi' = 29°$. While friction angles up to $\phi' = 37°$ have been measured at low confining pressures, values above $\phi' = 34°$ are rarely used for design purposes (Quaglia and Sfriso 2008). An effective cohesion due to cementation is accounted for in practical engineering design, in the range $15\text{kPa} < c' < 50\text{kPa}$.

Plate load tests have been extensively employed for measuring the in-situ soil stiffness. Coefficients of subgrade reaction for a 12" circular plate, measured during the construction of three metro lines, are shown in Figure 5. Average values in un-reloading are $K = 500\,\text{MN/m}^3$ in the Upper Pampeano and $K = 1200\,\text{MN/m}^3$ in the Middle Pampeano. Using elasticity formulas, a secant Young's modulus $E = 110\text{MPa}$ and $E = 260\text{MPa}$ can be calculated for both strata.

Pressuremeter tests have been employed for measuring the in-situ stiffness. High scatter is encountered both in pressuremeter modulus and limit pressure, which could be attributed to the difficulties of performing a circular boring in a soil mass containing cemented nodules. Pressuremeter modulus lies in the range $9\text{MPa} < E_M < 45\text{MPa}$ for the Upper Pampeano and $20\text{MPa} < E_M < 95\text{MPa}$ for the Middle Pampeano. Limit pressure lies in the range $1.8\text{MPa} < p_L < 3.1\text{MPa}$ and $3.3\text{MPa} < p_L < 6.1\text{MPa}$. Pressuremeter Young's modulus are often employed as a first estimate of the Young's modulus at 50% shear mobilization $E_{50}$.

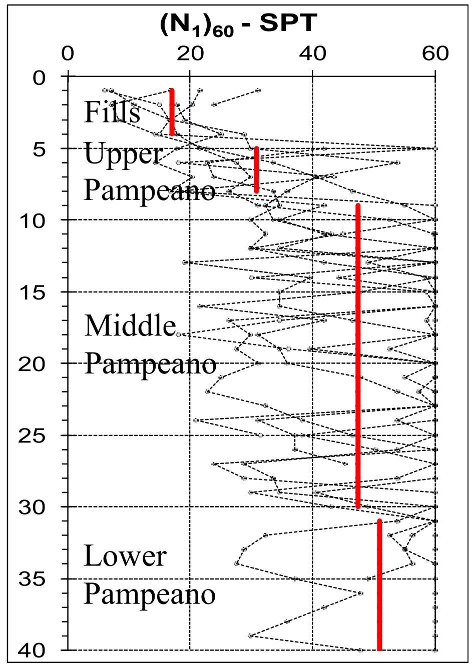



Figure 4. Typical SPT results in the Pampeano Fm. and average values.

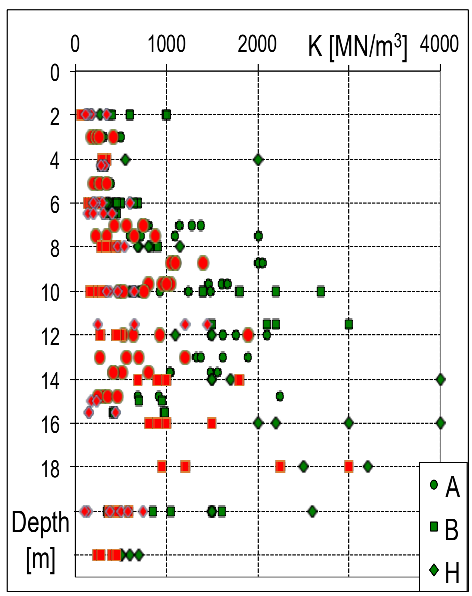



Figure 5. Coefficients of subgrade reaction measured. Red: primary loading. Green: un-reloading.

### 2.4 Permeability

The material itself, silts and clays, has a very low permeability, but, as explained hereinbefore, the formation has a dense pattern of microfissures which produces a much higher secondary permeability. Therefore, no laboratory permeability tests are usually carried out. Pump testing shows that the average permeability of the Middle Pampeano is in the range of $1 \cdot 10^{-5}\,\mathrm{m/s}$.

### 2.5 Earth pressure code requirements

The Building Code of Buenos Aires requires that strutted excavations be designed with a minimum earth pressure which depends on ground conditions and excavation depth (Figure 6).

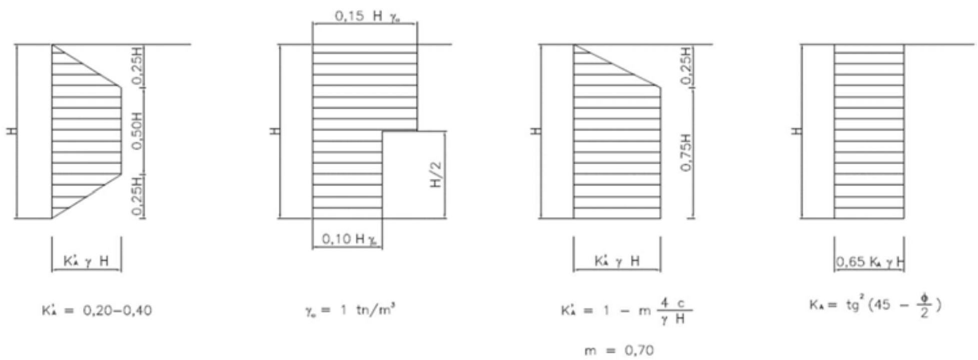



Figure 6. Diagrams of minimum earth pressure for strutted excavations after Building code. i) Stiff clays; ii) Pampeano Fm.; iii) Soft clays; iv) Sands.

### 2.6 Parameters for numerical modeling

Numerical methods have been extensively employed in the design of excavations and underground constructions since 1997. The accumulated experience after the back-analyses of the behavior of more than 10km of tunnels, 10 caverns and about 20 open-trench excavations resulted in a set of material parameters for the HS-Small constitutive model available in Plaxis, as shown in Table 1 (Plaxis 2017).

## 3 CONSTRUCTION PROCEDURES FOR METRO STATIONS

### 3.1 Open trench excavations

Line B Tronador and Los Incas stations were excavated in 1997 to 1999 using open trench excavation techniques. 1.2m diameter bored piles, 22.0m long and separated 2.8m between axes were employed to support precast beams which in turn supported precast formwork slabs. A cast-in-place concrete layer completed the roof slab. Space between piles was covered by a thin arch of shotcrete. A final, non-structural wall covered both the piles and the shotcrete arches. The construction procedure was designed to minimize bending moments in the piles and to allow for the use of precast elements: i) open excavation to a depth of 4.0m using the piles to support the lateral ground acting in cantilever mode; ii) installation of precast beams and formwork slabs; iii) cast of the entire roof slab, which act as a groundlevel strut; iv) reopening of the street to transit; and v) the excavation continued under the slab using the piles to support the ground in propped mode, down to a depth of 12m. Due to delays in the provision of the beams, the open excavation was advanced to a depth of 7m, forcing the piles to work in cantilever mode (Figure 7).

**TABLE 1**. Material parameters for the HS-Small constitutive model.

| Description | Symbol | Unit | Upper Pampeano | Middle Pampeano | Lower Pampeano | Puelche Sands |
|---|---|---|---|---|---|---|
| Unit weight | $\gamma$ | kN/m$^3$ | 19.0\|20.0 | 19.0\|20.5 | 19.0 | 20.5 |
| Peak friction angle | $\phi'$ | º | 29\|31 | 32\|34 | 29\|32 | 35\|38 |
| Effective cohesion | $c'$ | kPa | 5\|25 | 35\|50 | 10\|25 | 0 |
| Dilatancy angle | $\psi$ | º | 2\|4 | 4\|6 | 1\|3 | 5\|8 |
| Shear modulus | $G_0^{ref}$ | MPa | 200\|300 | 300\|450 | 220\|300 | 280\|400 |
| Threshold distortion | $\gamma_{0.7}$ | - | $10^{-4}$ | $10^{-4}$ | $10^{-4}$ | $10^{-4}$ |
| Unloading stiffness | $E_{ur}^{ref}$ | MPa | 120\|180 | 200\|320 | 140\|200 | 180\|250 |
| Secant stiffness | $E_{50}^{ref}$ | MPa | 40\|60 | 80\|110 | 60\|75 | 80 a 100 |
| Oedometer stiffness | $E_{oed}^{ref}$ | MPa | 40\|60 | 80\|110 | 60\|75 | 80\|100 |
| Stress exponent | $m$ | - | 0.3\|0.5 | 0.3\|0.5 | 0.3\|0.5 | 0.5 |
| Poisson ratio | $\nu_{ur}$ | - | 0.20 | 0.20 | 0.20 | 0.20 |
| Precons. pressure | $POP$ | kPa | 600\|900 | 1200\|1800 | 800\|1200 | 200\|400 |
| At rest coefficient | $K_0$ | - | 0.6\|0.7 | 0.7\|0.8 | 0.6\|0.7 | 0.4\|0.5 |
| Permeability | $k$ | $10^{-6}$ m/s | 0.1\|0.5 | 0.1\|0.5 | 0.05\|0.01 | 1\|5 |

A significant lateral deflection was observed at the pile heads ($\delta_h \cong 15|25mm$) and some minor cracks formed in the pavements, but no major instabilities occurred and the construction was completed to success.


Figure 7. Tronador Station. Note piles with no head support working in cantilever mode.

The open pit excavation construction procedure proved slow, expensive and highly disruptive to the urban environment and was abandoned after this experience.

### 3.2 Cut&cover techniques

The first cut&cover station employing piles to support lateral ground pressures was Olleros Station, Line D, completed in 1987, shortly followed by Juramento Station. Since 1997, fourteen stations were excavated using cut&cover techniques, namely Line H stations Venezuela, Humberto Primo, Inclán, Caseros, Parque Patricios, Hospitales and Facultad de Derecho; Line A stations Puán, Carabobo, Flores and San Pedrito; and Line E stations Correo Central, Catalinas, and Retiro. The typical cross-section is shown in Figure 8.

The construction procedure is depicted in Figure 9: i) transit is diverted to one side while piling and construction of a cast-in-place half slab on ground is performed at the other side; ii) the road is reopened and transit shifts side to allow for constructing the second half of the slab; iii) the excavation is performed underground, with minimal transit disruption.

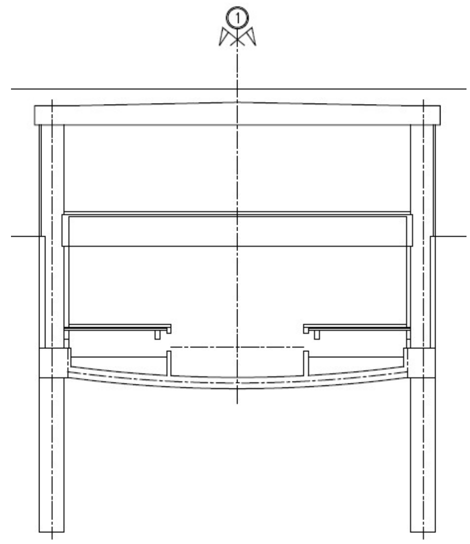


Figure 8. Typical cross-section of a cut&cover station.

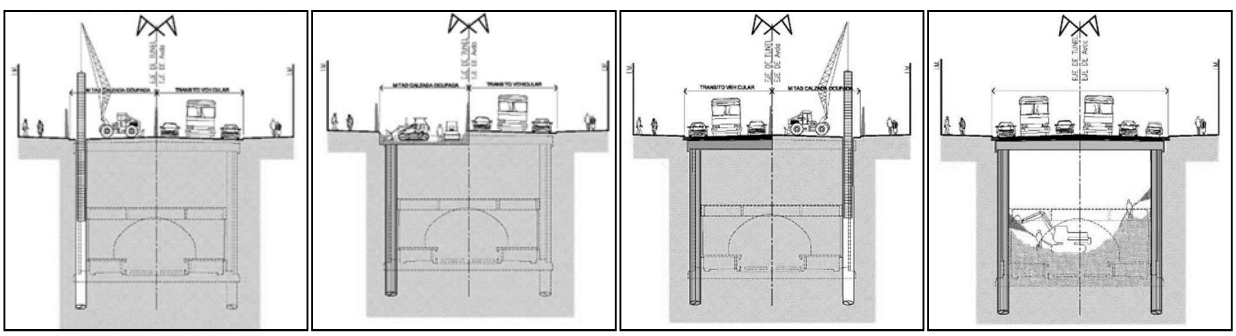


Figure 9. Construction procedure for cut&cover stations.

Using this construction procedure, piles are never forced to work in cantilever mode. Settlements and lateral displacements of the pavements is negligible in excavations in the Pampeano, but could rapidly grow where Post-Pampeano soils are present and not taken into account in the design. The total construction time span is 11 to 14 months for the excavation and installation of all support systems including the floor slab (Figure 10). Disruption to surface transit spans for eight to twelve weeks, depending on logistics and site conditions.

### 3.3 Underground caverns

Seven underground stations were excavated since 1997, namely Line H stations Once, Corrientes, Córdoba, Santa Fe and Las Heras and Line B stations Esteban Echeverría and Roosevelt. Cavern construction procedures evolved significantly during this process, from german tunneling method to crown-bench full-face. Corrientes Station was the landmark achievement of the construction methodology used ever since. It consists in a NATM full-face excavation using two backhoe equipments located at two benches, producing two independent working faces, but keeping the closure of the structural ring within one diameter of the tunnel face. The complete cycle allows for 6m to 8m advance per week resulting in a total duration of the excavation of seven months in 2007 (Sfriso 2007). The procedure, while challenging, proved sound for an ample range of geotechnical conditions.

Figure 10. A cut&cover cavern completed..

# 4 CASE HISTORIES

## 4.1 Line D Underground Maintenance Facility

Line D Maintenance Facility required an excavation 22.9m wide, 51.5m long and 10.2m deep, which was supported on 1.0m dia bored piles, 16.0m long, 7m embedded and separated 2.70m. The roof slab was formed by precast elements, while the curved invert was cast in place and hinged against recesses placed in the pile reinforcement. Only 3.5m of artificial fills overlaid the Pampeano soils and were largely removed during the surface excavation required to install the roof wall. Figure 11 shows the cross-section of the structure. The analysis and design was performed using standard structural engineering procedures, where ground-structure interaction is simulated by Winkler-type springs. Figure 12 and Figure 13 shows two of the structural load cases employed in the numerical model. Figure 14 shows a panoramic view of the structure during the construction of the invert while Figure 15 shows the structural detail of the grooved contact joint between the bored piles and the concrete invert vault. The grooves were made of steel rings welded to the pile reinforcement previously installed during pile construction.

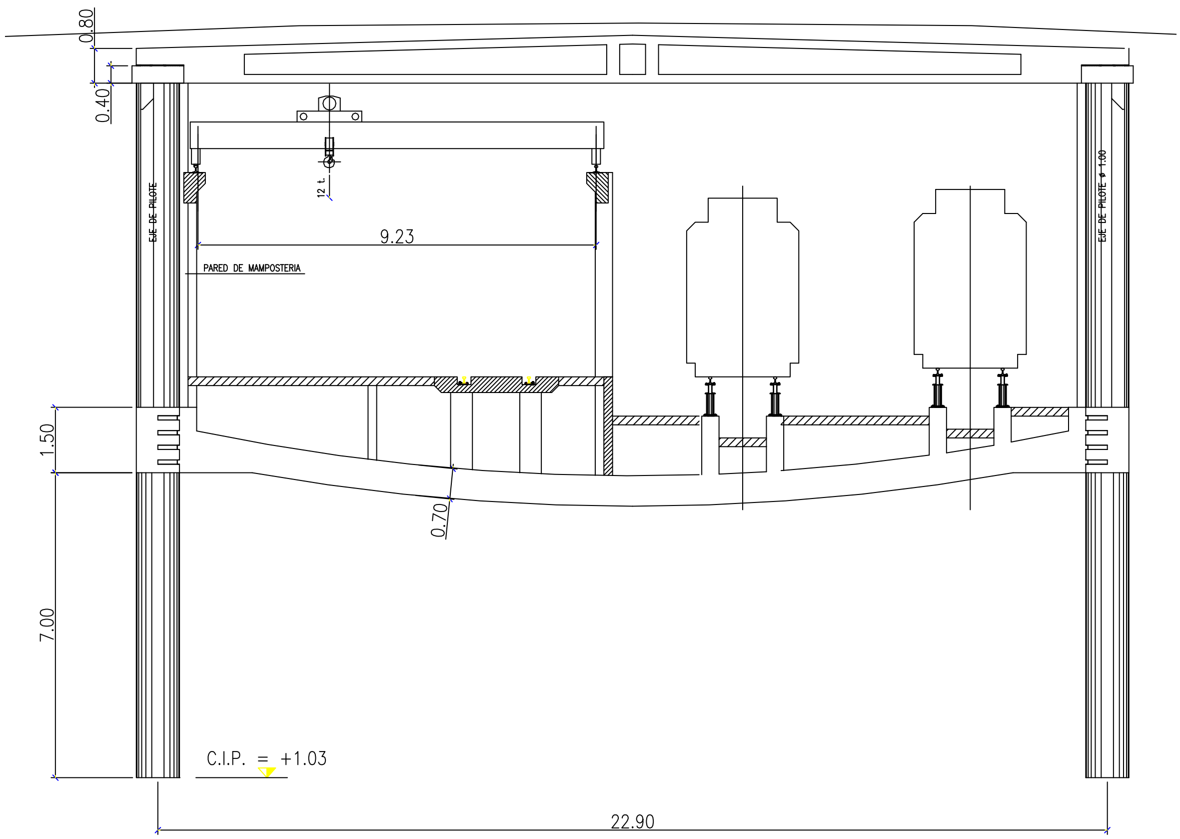



Figure 11. Cross-section of Line D Maintenance Facility.

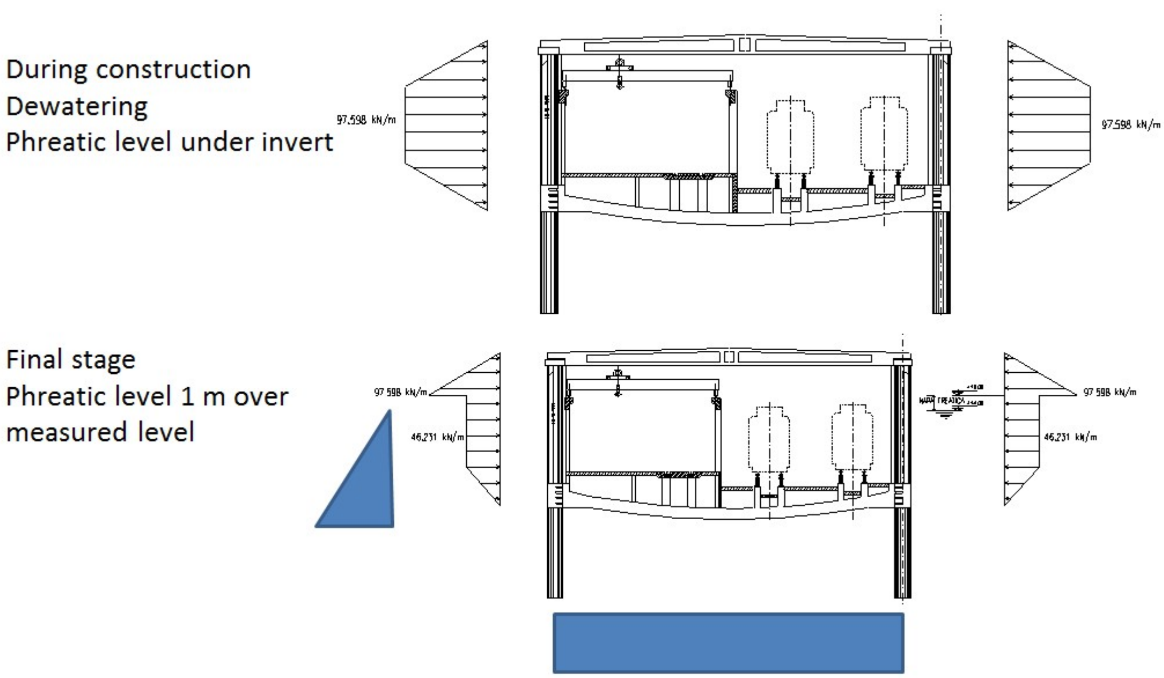



Figure 12. Structural model for designing Line D Maintenance Facility (reproduced with permission from the calculation notes of the project).

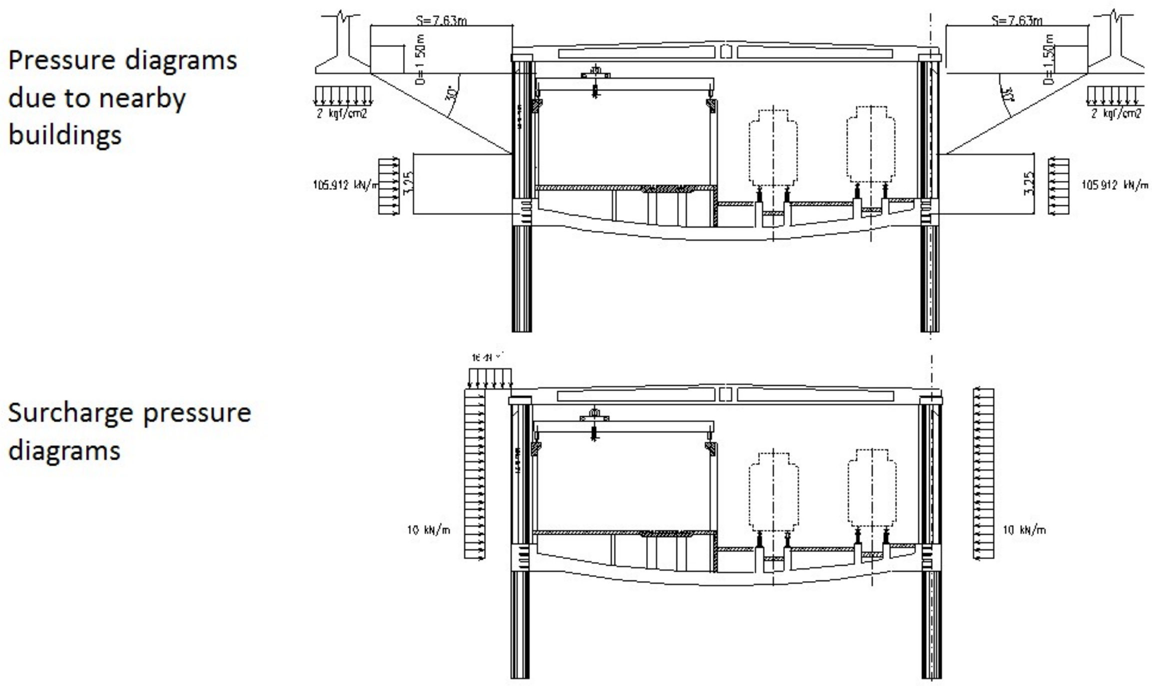



Figure 13. Structural model for designing Line D Maintenance Facility (reproduced with permission from the calculation notes of the project).

Figure 14. Panoramic view of Line D Maintenance Facility during construction of the invert.

Figure 15. Structural detail of the connection of the invert to the lateral bored piles.

A monitoring system comprising points fixed to existing structures was installed prior to the beginning of the drainage system. The settlements were in all cases negligible showing no impact from the excavation on the structures, and furthermore proving that the depression of the phreatic level has no effect whatsoever on the behavior of the Pampeano soils.

### 4.2 Facultad de Derecho Station

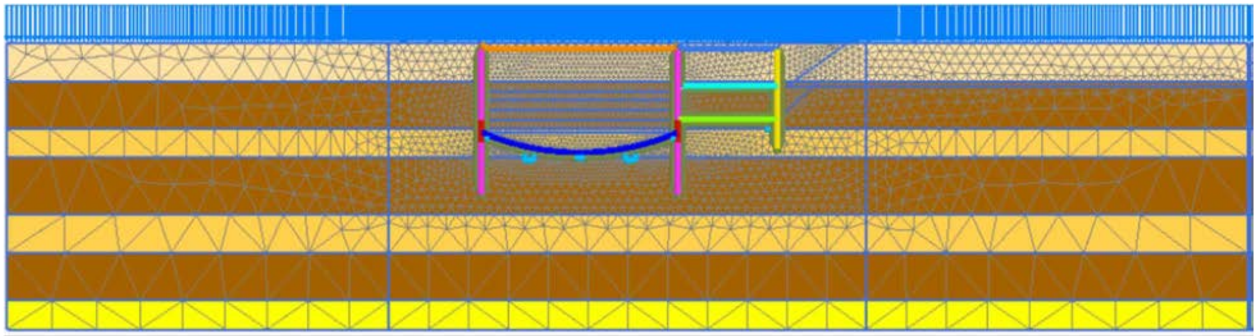
Facultad de Derecho Station is located 20m East of the School of Law of the University of Buenos Aires, close to Figueroa Alcorta Avenue, which runs NW-SE close to the bottom of the slope limiting the upper plains of the city center. The station is 195m long, 19.8m wide and 11.3m deep. The geotechnical profile is: i) 4m of man-made fills; ii) 22m of Pampeano soils: and iii) Puelche sands. While the profile is very similar to that of Line D Maintenance Facility, the man-made fills are of poorer quality and the size of the excavation is significantly higher than the previous case.

Being an excavation of moderate depth, it required a low-risk, conventional structural and geotechnical design. A retaining wall made with bored piles, 1.20m in diameter, 18.5m long and separated 2.50m was employed. Piles were reinforced to provide a yield bending moment of 793kNm/m. There are ϕ1.60m bored piles bored adjacent to the main entrances to the station from a lateral mezzanine with a yield bending moment of 1460kN/m. The ground between piles was supported by a 10cm shotcrete arch. The invert was connected to the side walls between the piles, with no connection to the piles themselves (Figure 16). This structural detail results in time and cost savings, despite the fact that it requires a heavier slab reinforcement. Figure 17 shows a panoramic view of the main building, the open-trench excavation of the mezzanine and the building of the School of Law.

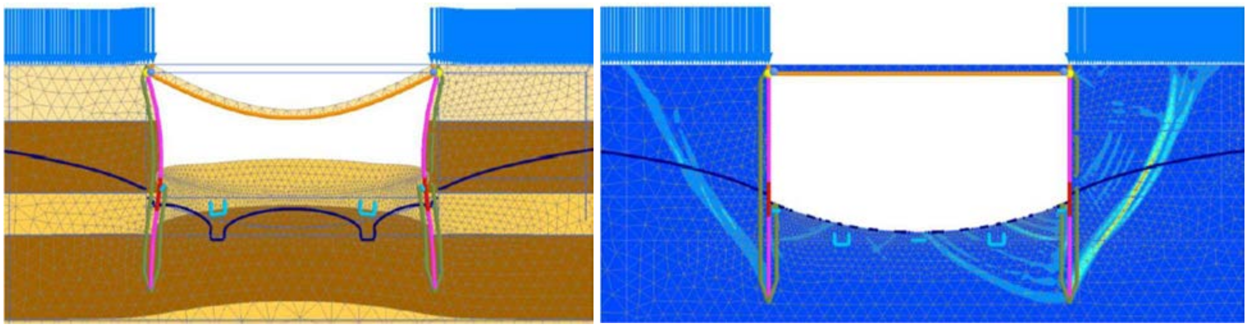
Figure 16. Reinforcement of the slab connected to the (future) wall between the piles.

Figure 17. Panoramic view of the underground station during the open-trench excavation of the mezzanine, and the building of the School of Law.

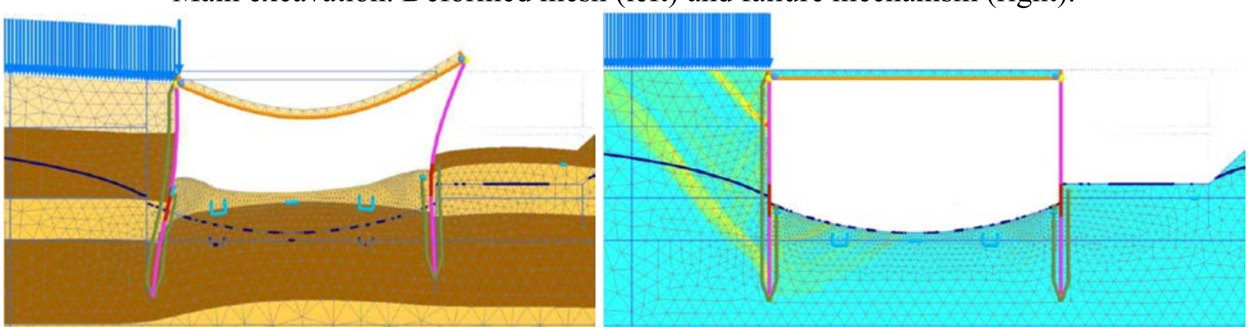
The construction sequence, geotechnical and structural design were performed using numerical models, see Figure 18. Structural design was also verified to comply with earth-pressure diagrams provided by the Buenos Aires construction code, see Figure 19.



Finite element mesh.



Main excavation. Deformed mesh (left) and failure mechanism (right).



Excavation of Mezzanine. Deformed mesh (left) and failure mechanism (right).

Figure 18. Numerical analysis of the construction sequence of Facultad de Derecho Station.

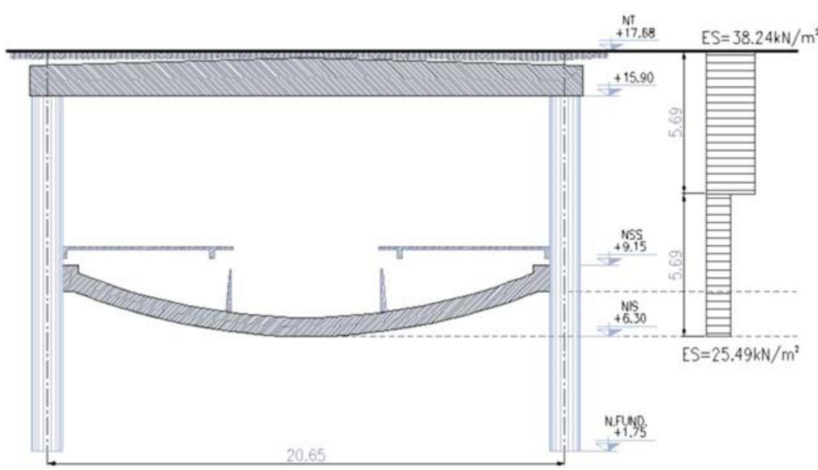



Figure 19. Minimum lateral earth pressure diagram, Buenos Aires construction code.

Construction sequence was eased by the fact that the station is located at the backyard of the university building, which allowed for the casting of the roof slab in full width. The superficial fills and soft soils proved to be a challenge for the adequate construction of the bored piles, see Figure 20 and Figure 21. The maximum settlements measured on points fixed to the Law School building were less than 2 mm.


Figure 20. Construction defects in the upper portion of the bored piles.


Figure 21. Construction defects in the portion of the bored piles excavated in the man-made fills and superficial soft soils. Note the systematic over-excavations and exposure of reinforcement.

## 4.3 Correo Central Station, Line E

Correo Central Station, Line E, is located under Leandro N. Alem Avenue which runs N-W at the toe of the ridge limiting the upper plains of the city center. It was a challenging project because it was the first excavation for a Metro project where the Lower Pampeano clays were exposed and where there was risk of bottom instability due to uplift pressure from the Puelche aquifer. Lower Pampeano soils, never exposed to unsaturation, are greenish instead of brownish, have weak cementation and are of noticeable poorer quality than the Middle Pampeano soils.

The structure was designed to have an all-around PVC geomembrane for watertightness. Piles were designed to support just the lateral ground pressure while a inner wall was installed to resist the water pressure, supported in the floor slab, cast-in-place invert and the mezzanine slab. The cross-section of the structure and the geotechnical profile are shown in Figure 22 (Laiun et al 2014).

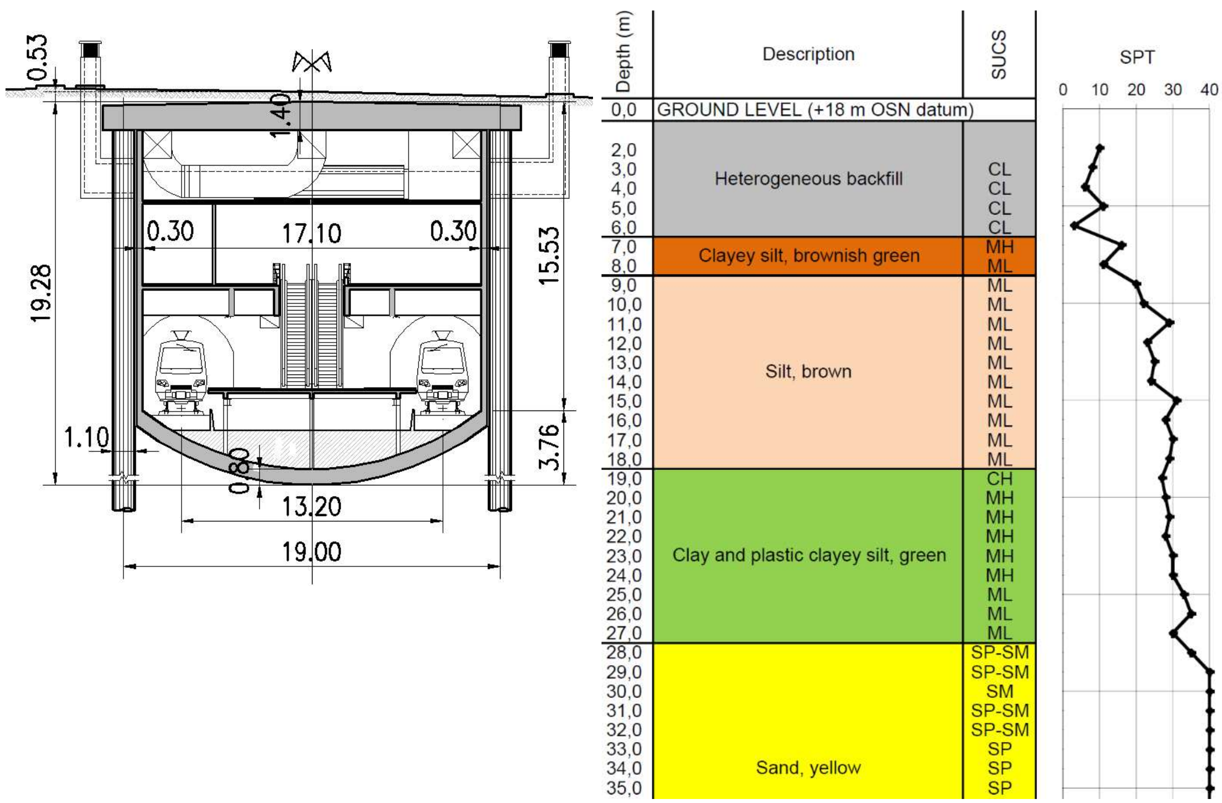



Figure 22. Correo Central Station. Cross-section and geotechnical profile.

The construction procedure was as shown in Figure 9 and Figure 23 but adapted to overcome the adverse geotechnical conditions at the bottom of the excavation. Underground excavation was performed as follows: i) a safe level for general excavation was determined and reached in two stages; ii) two drainage pipes were installed in open trenches; iii) the excavation of the invert was advanced in 2.2m strips and supported with lattice girders welded to the pile reinforcement and shotcrete; iv) the geomembrane was installed; and v) the final invert was cast-in-place. Figure 24 shows the construction sequence, Figure 25 shows the piles, the open box and the concrete invert vault during construction, and Figure 26 shows all stages of the invert construction simultaneously, from ground removal to final cast-in-place invert (Sfriso and Laiún 2012, Laiún et al 2014).

Figure 23. Correo Central Station. Construction procedure of the roof slab-on-piles.

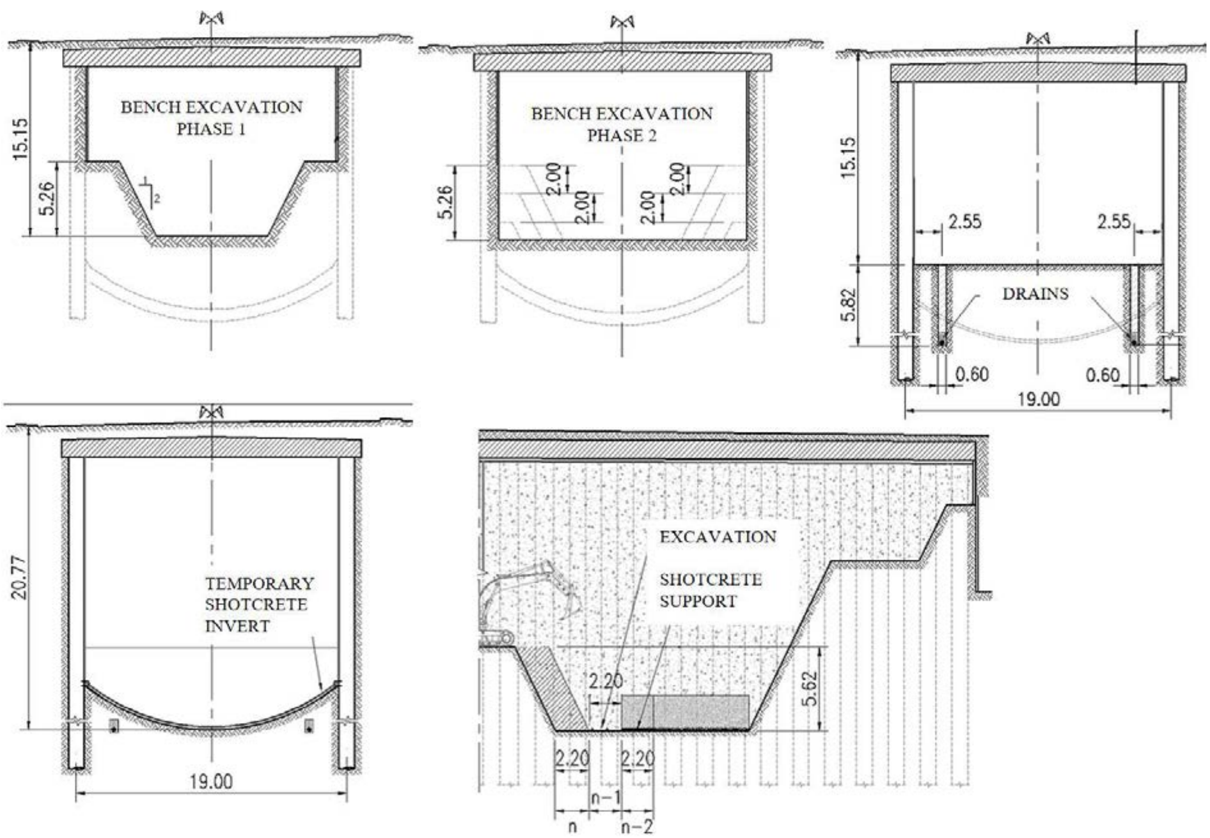



Figure 24. Correo Central Station. Construction procedure for the invert.

Figure 25. Correo Central Station. Phase 2 excavation, pile-supported wall and invert.


Figure 26. Correo Central Station. All construction stages of the invert including the installation of the geomembrane and the casting of the final invert.

The construction was successfully completed with minor delays, mainly due to logistics of the extraction of the excavated soil and transit diversion. The overall behavior was excellent. Nearby buildings showed vertical displacements less than 5 mm. There was no observed distress in ground at the excavation bottom and no water ingress. No cracks, fissures or noticeable tilting of above-ground structures and facilities was observed. An inclinometer was installed between the station and the Correo Central building (nowadays a major concert hall) showed a maximum horizontal displacement of 2mm (Figure 27).

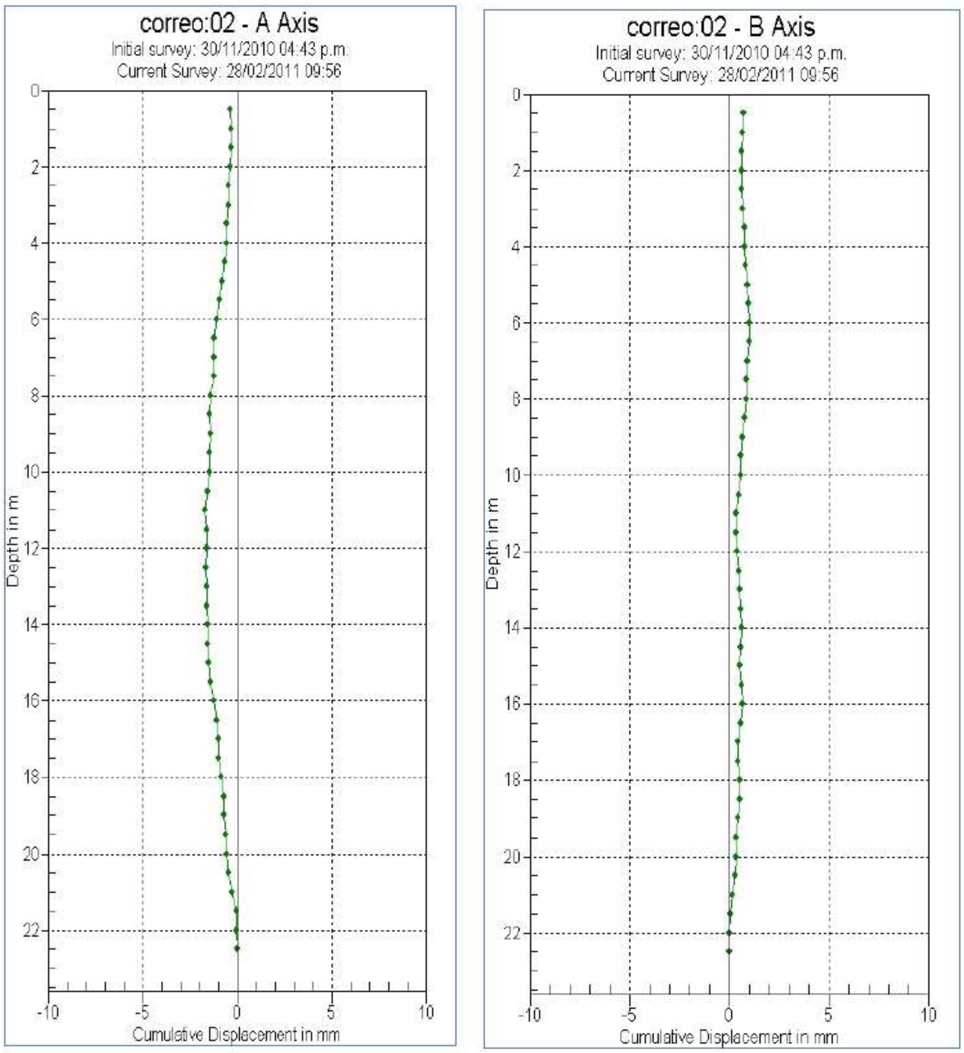



Figure 27. Inclinometer measurements. A-axis is perpendicular to the station axis, B-axis is parallel to it.

## 4.4 Hospitales Station

Hospitales Station, the last station in Line H Southbound, encountered geotechnical challenges with soft surface soils, similar to those of Facultad de Derecho Station, and bottom uplift, similar to Correo Central Station. In this case, piles were cased in the upper 4.0m, resulting in a systematic overexcavation and bulging of the piles at the end of the casings (Figure 28). The construction of the invert was performed in 8.0m wide strips excavated from a safe level (Figure 29).

Figure 28. Bulging of piles at the end of the cased portion.

Figure 29. Strip excavation of the invert.

### 4.5 Pre-Hospitales tunnel, Line H

Hospitales Station is being employed as a terminal station. Not being designed as such, it does not have an emergency stop rail section which is required for safety. While still discussing how to build the tunnel south of Hospitales Station, the Client decided that this stop rail section was urgently required, and ordered the construction of 40m of tunnel past Hospitales Station.

Geotechnical conditions in the site are poor, as must be expected given the location of the project in the flood valley of the Riachuelo river. Figure 30 shows the longitudinal geotechnical profile which shows an increase of the upper soft soil stratum thickness. A 5m to 14m thick layer of young soft soils of fluvial origin rests on top of heavily eroded and decompressed stiff silts and clays of the Pampeano Formation. Puelche sands are found at 21m depth, again introducing the risk of bottom uplift. The loose sands at the top of the profile could be subjet to extreme piping when careless dewatering works are carried out. This geotechnical profile precluded the construction of both NATM tunnels and non-contiguous pile retaining walls.

It was decided that the tunnel should be excavated by cut&cover employing a diaphragm wall as required by the bidding documents. After several weeks of engineering studies, the solution was discarded for three reasons: i) there was limited equipment availability for several months; ii) it was very expensive to mobilize a full diaphragm wall set – cranes, clamshell, bentonite sludge plant – for just 80m of wall; and iii) disruption, as the footprint of the diaphragm wall construction site would have required closing Almafuerte Avenue for several months.

It was soon realized that the diaphragm wall solution posed many technical and logistics challenges: i) it required the total closure of the Almafuerte Avenue, a critical artery for vehicles entering the city in rush hours, for several months; ii) it required the removal of several buried utilities including high-voltage transmission cables, gas pipelines, fresh-water and waste-water ducts, etcetera; and iii) there was a shortage of equipment able to complete the job within the assumed schedule. A secant pile wall was employed instead. Bored structural piles, 1.20m dia, 21.80m long and separated 1.80m between axes were designed to support the ground and water pressure, roof slab loads and all surface loads. Low-strength plain plastic concrete piles of equal size and length were installed as secondary piles to close the gap between the structural piles, producing a continuous wall.

The controlled low-strength material dosification employed in the non structural piles is: i) $w/c = 0.95$; ii) Cement+furnace slag $\approx 310$ kg/m$^3$; iii) Sand $\approx$1550 kg/m$^3$; iv) Additives: hydration stabilizer and improvement of segregation resistance. The unconfined compression strength of the mix is 3 to 6 MPa (at 7 days) and 12 MPa (at 30 days).

Figure 30 shows the cross-section of the Pre-Hospitales tunnel and the geotechnical profile where it is located. Note that the secant pile wall barely penetrates into the very dense Puelche sands, as the net vertical load acting on the piles is negligible due to the uplift water pressure acting at the invert.

The construction procedure is typical of cut&cover excavations with two additions: i) dewatering wells are installed in the inner side of the box and kept operational through the entire excavation; and ii) an intermediate horizontal frame is installed 3.40m below the roof slab before the excavation is advanced to the invert level. This horizontal concrete frame, installed just above the required clearance for the trains is required for two reasons: i) the secant pile wall is not stiff and robust enough to resist the ground and water loads of the full excavation height; and ii) were the piles forced to support the full excavation with poor quality ground they would have required a much deeper embedment and would have high lateral displacements at the bottom of the excavation.

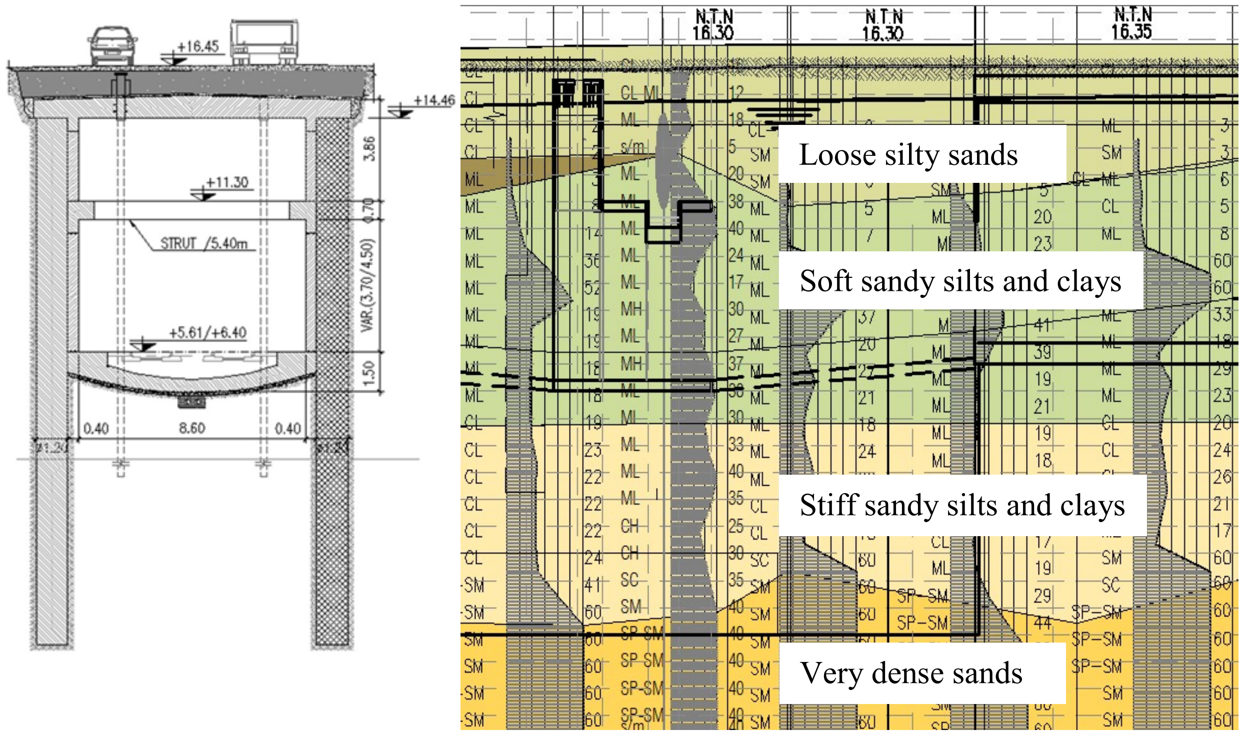



Figure 30. Tunnel pre-Hospitales. Cross section and geotechnical profile.

The long-term watertightness of the structure required the construction of an inner structural concrete wall, 0.40m thick, supported by the piles by means of epoxy-glued rebars, to be built after the tunnel invert is completed. This structural wall acts also as the support of the invert, and must therefore take the high vertical uplift loads produced by water pressure. Figure 31 shows the construction procedure including the drainage wells, intermediate frame and inner structural wall. Figure 32 shows the ground shear mobilization at the end of the construction.

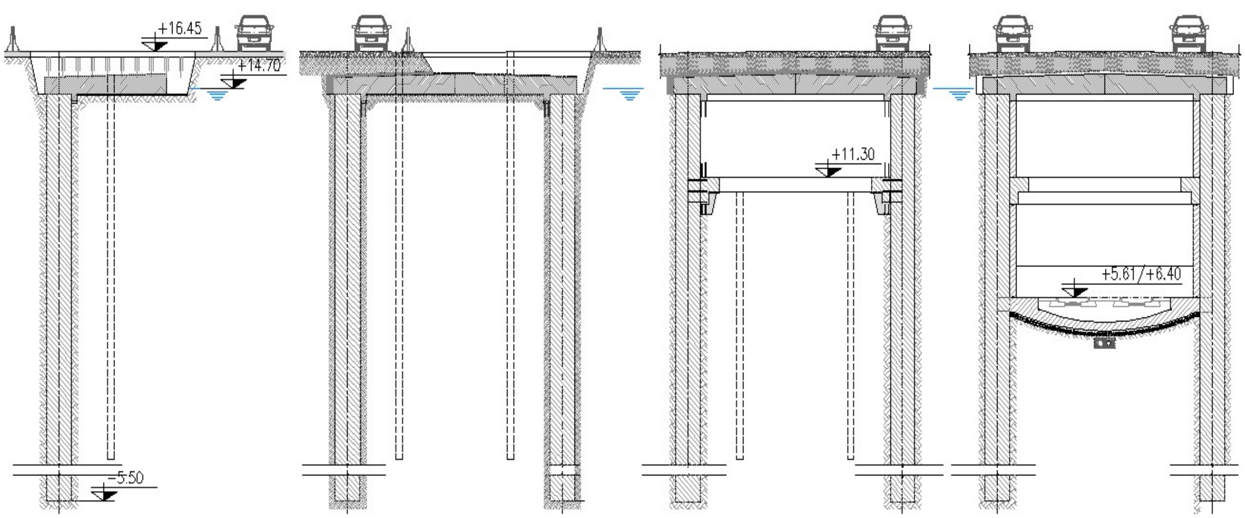



Figure 31. Construction procedure of the Pre-Hospitales tunnel.

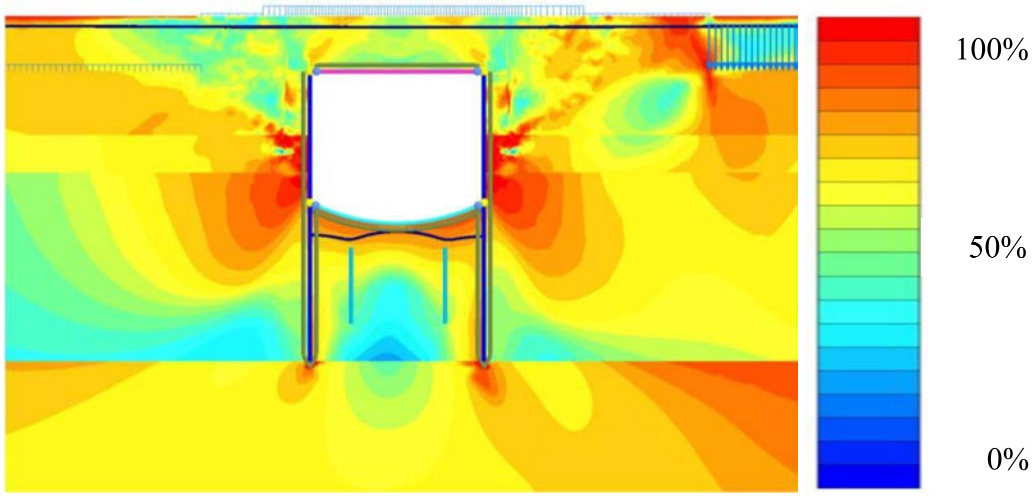



Figure 32. Shear mobilization at end of construction.

Soletanche-Bachy, the foundation specialist contractor, guaranteed that the deviation of verticality would be less than 0.75%. They employed a Soilmec SR-70 equipment with no casing, which is kind of an unconventional installation for performing secant pile walls. Two buckets were employed for the construction of the structural piles: i) an open bucket provided with hardened steel teeth for concrete cutting; and ii) the normal excavation bucket. A concrete template was employed to guide the initial pile excavation. Figure 33 shows the piling rig and the two buckets. Figure 34 shows the concrete template and the secant pile wall completed.

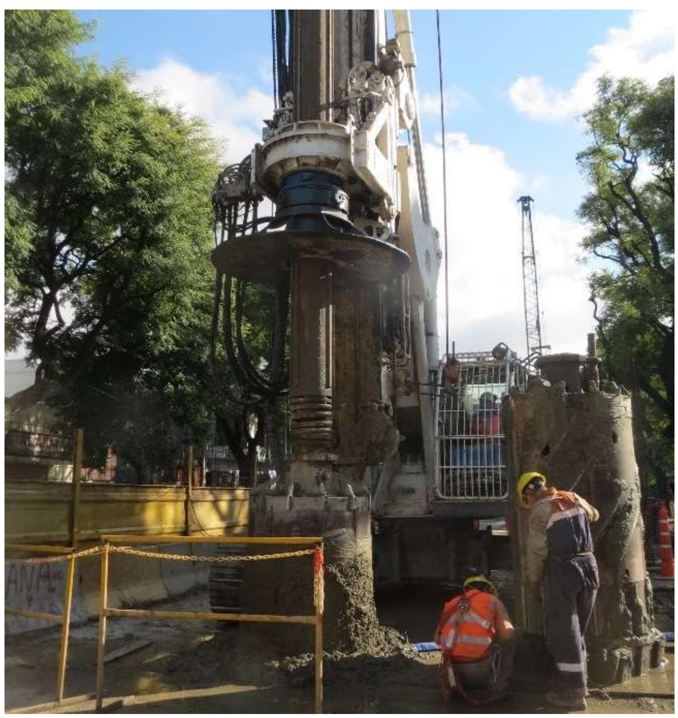


Figure 33. Soilmec SR-70 piling rig provided with two buckets and no casing during the construction of the secant pile wall at Pre-Hospitales tunnel.

Figure 34. Pre-Hospitales secant wall. Construction of concrete template and finished wall.

Figure 35 shows the connection of the slab reinforcement to the piles. Figure 33 shows a panoramic view of the construction of the roof slab and the placement of a CLSM fill on top of the concrete.

Figure 35. Pre-Hospitales tunnel. Connection of the slab reinforcement to the piles.

Figure 36. Pre-Hospitales tunnel. Construction of roof slab and filling with CLSM.

The installation of a 90m perimeter of 22m deep secant piles began on November 2016 and finished on March 2017. It was recommended that the secondary structural piles should be bored within 48 hours after the casting of the adjacent primary soft piles. Unfortunately, this aim was seldom achieved resulting in a lower performance. In average, the secondary piles were bored at a rate of 3m per hour, resulting in less than 1.5 pile per day being executed. At the present time the Contractors are building the second half of the roof slab.

## 5 LESSONS LEARNED AND CONCLUSIONS

During the last twenty years, the construction of a large quantity of underground stations belonging to the expansion plans of the Buenos Aires Metro was an opportunity to acquire a large body of experience and expertise in the design and construction of pile-supported retaining walls.

Monitoring programs and quality control procedures in construction resulted in a set of measured values of ground behavior which have been employed for the calibration of numerical models in a process that has been going on for twenty years now. The result is a set of reliable material parameters that can be employed as a very good estimate for the initial design of new excavations and tunnels.

Employing non-contiguous bored piles for ground suppport of cut&cover excavations was made possible by the good knowledge of the behavior of Buenos Aires City soils and by the good state-of-the-practice in the construction of piled foundations in the city. Cut&cover excavations, when employed in dense urban environments, resulted in a good construction procedure which controlled the geotechnical and general construction risks for existing structures, minimized the environmental impact of constructions and reduced the time lapse of transit disruption.

Five case-histories were presented: Line D Maintenance Facility, Facultad de Derecho, Correo Central and Hospitales Station, and Pre-Hospitales tunnel. All cases are located in transition zones, where the Pampeano soils are overlaid by softer young soils or fills. Four of them were successfully solved by employing non-contiguous pile retaining walls. The subsequent excavation proved that construction of bored piles in soft soils and fills is still a challenge that requires improvement in construction techniques. The fifth case-history is the first application of a secant pile wall to a Metro project in Buenos Aires. A cutting-edge equipments has been able to excavate the piles and build the wall. While there will be an opportunity to inspect the surface quality of these piles, it is forecasted that the secant pile wall solution will be used many more times in future deep excavations in Buenos Aires.

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