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SYNOPSIS: The first time that the New Austrian Tunnelling Method (NATM) was used for underground works in the São Paulo Subway was in 1981. Since that time, significant progress has been achieved in successfully optimizing the support and lowering the construction costs. This paper will describe the latest experience of two single-track parallel tunnels excavated in 1986 through tertiary stiff clay. All the experience accumulated in previous jobs led to design improvements, such as (1) no steel ribs for support along 72% of the tunnels length (2) no temporary invert, and (3) no spiles, forepoles, soil grouting or any other type of ground improvement. Significant cost and time reduction were the practical result. A method for quick and efficient storage and graphical interpretation of instrumentation readings was developed and implemented in a network of microcomputers installed at the construction site, owner's and engineers' offices.

INTRODUCTION

Since 1981 the New Austrian Tunnelling Method (NATM) has been used for tunnel construction in connection with the extension of the North Line of the São Paulo Subway. In the first case, two parallel 60-m long single-track tunnels (6-m diameter) were excavated through tertiary stiff clay and soft organic clay. Face instability problems already related previously (Celestino et al., 1982) occurred in the soft clay. The solution basically consisted on the use of long grouted spiles.

In the second case, a 200-m long double-track (12-m diameter) tunnel was excavated through stiff tertiary clay, with overburden varying between 5 and 13 m. This construction was completely successful (Celestino et al., 1985) and new cost standards for underground works were established for the São Paulo Subway. (Before the use of NATM, only cut-and-cover and shield had been used for local underground construction). The double-track tunnel was excavated underneath poorly constructed old buildings. In the most critical case the foundations of a 5-storey masonry building (with no concrete or steel structure) were only 5 m above the tunnel crown (Mitsuse et al., 1985) and no damage other than minor cracks occurred. Tenants kept occupying the building normally during tunnel construction.

Due to design optimizations of support and final lining of the double-track tunnel, the contract was finished with fund surplus. The Subway Company decided to use those funds for the construction of two 63-m long single-track tunnels, also along the extension of the North Line, the construction of which is currently underway. This paper describes the performance during the construction of these tunnels, design optimizations that could be adopted based on instrumentation results, and the final cost.

It is important to emphasize that design optimizations during construction could only result in cost reduction due to the flexibility of the contract, as described elsewhere (Cruz et al., 1985).

Being NATM an observational method, (Rabcewicz, 1964) optimizations are heavily dependent on efficient instrumentation interpretation. Mathematical models have been used for the jobs mentioned above, and they have been upgraded as construction progressed in order to obtain good agreement between their results and instrumentation readings. Calibrated models helped to evaluate beforehand the consequences of design optimization measures and the resulting safety.

DESIGN DATA

The tunnels were excavated through tertiary stiff fissured clay. Above the West tunnel, quaternary deposits of soft clay with sand lenses occurred. Figure 1 shows a cross section through the portals. The overburden varies from 18 m at portals to 14 m at the end of the tunnels, 63 m away. The thickness of stiff clay above the crowns of the tunnels also decreases slightly to a minimum of 2 m. The pillar between the tunnels is about 5 m wide. The excavation progressed from a shaft towards the dead end of existing tunnels already in operation.

The stiff clay is randomly fissured. In some locations, no fissures could be noticed on the tunnel face; in others, spacing varied from a few to tens of centimeters. Their attitudes were also randomly distributed. They were usually slickensided, having very low strength. Unstable blocks were sometimes formed at the excavation face, and this was one of the critical problems that had to be looked at during construction. The SPT penetration

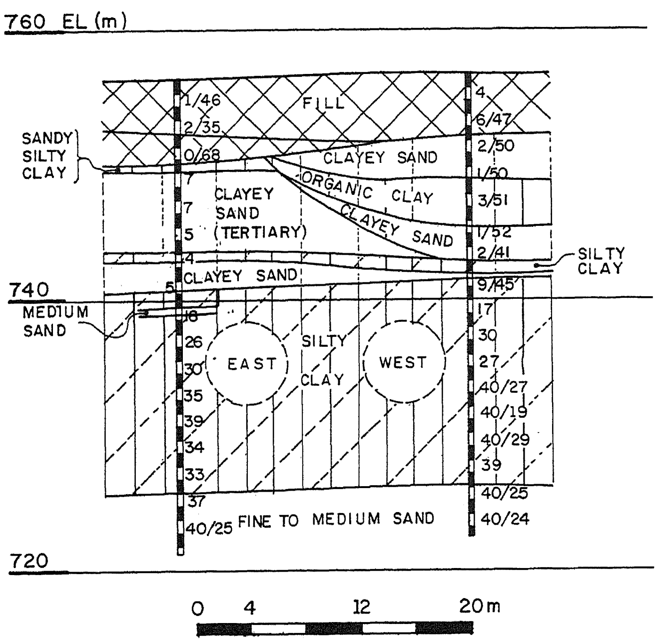

FIGURE 1 - Geological Cross Section Through the Portals

resistance of the clay (blows/foot), also indicated in Figure 1, is in the range of 20 to 40. The modulus of elasticity, and the coefficient of earth pressure at rest, both inferred from instrumentation results during the excavation and mathematical models, were 100 MPa and 1.0 respectively. The latter is also in agreement with theories of earth pressure for overconsolidated clays (e.g. Brooker and Ireland 1965) with an overconsolidation ratio of 5.

Eventhough not important for face stability the deposit of soft clay above the West tunnel played an important role for surface settlements. With a natural void ratio in the range of 2 to 3, the material undergoes appreciable consolidation under minor stress changes.

There are two water tables: one gravitational, approximately at elevation 750 m, and the other one artesian, in the sand deposit below elevation 723 m. Due to the low permeability of the stiff clay, no dewatering was necessary.

In spite of being in urban environment, there were no important buildings directly above the tunnels. However, several buildings were within the area of influence of the work.

Both support and final lining consisted of shotcrete, as indicated in Figure 2, with thicknesses of 15 cm and 10 cm respectively, and 2.2 kg/m2 CA-60B steel wire mesh. According to the design, 4-inch I steel sets would be used for support every 80 cm in the initial and final 10 m long stretches of the tunnels. Both at the portal and next to the dead end of previously excavated tunnels, non-symmetrical loads could be anticipated due to previous

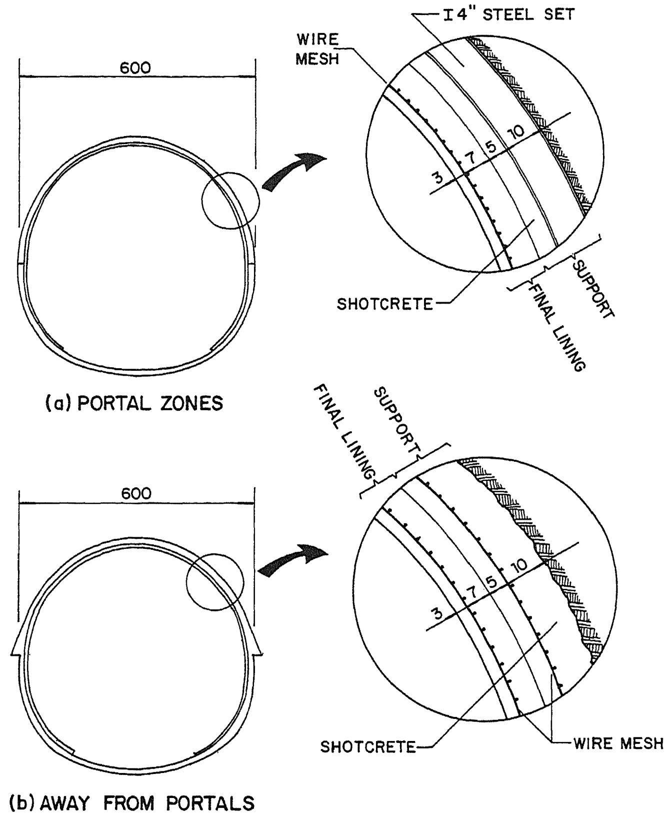

FIGURE 2 - Support and Final Lining (Thicknesses in Millimeters)

disturbance and loosening of the soil mass. Lengths of stretches where steel sets were actually installed were different, as described later on.

The tunnels were top-heading and bench excavated, with 0.8 m advance and distance from bench to face varying from 3.6 to 5.2 m. A core was left at the face in order to minimize problems of unstable blocks formed by slickensided. No global face stability problems were anticipated. Support and final lining were designed with basis on interaction diagrams (bending moment versus normal force) obtained from the mathematical model. The model reproduced the different excavation phases, placement of support and its hardening with time. Long term condition was simmulated by (a) introducing a factor of safety (equal to 2) on the effective strength parameters of the soil, thus causing soil relaxation and increase in load transferred to the shotcrete lining; (b) considering a 2-m increase in the elevation of the water table due to long-term fluctuations; and (c) considering surface and deep loads to be transferred to the soil mass by foundations of future buildings. An example of interaction diagrams is presented in Figure 3. Shown there are bending moment (Md) versus thrust (Nd) envelops for all cross sections of the lining and the moment-thrust interaction diagrams for the support and final lining.

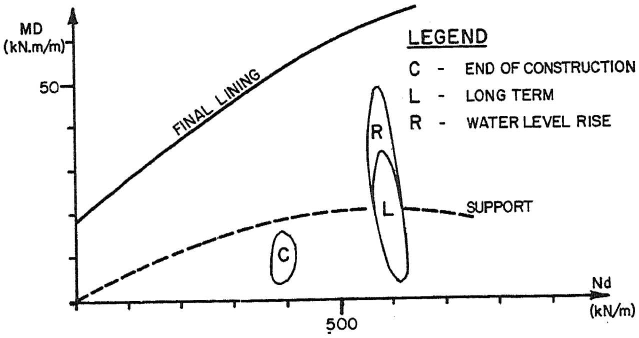

FIGURE 3 - Moment-Thrust Diagram for Support and Final Lining

MONITORING

Instrumentation was very simple and consisted of 7 control sections of surface and deep settlement devices.

A Full control section is shown in Figure 4, with two levels of deep as well as surface settlement devices. Some sections were more simple and had only one surface and one deep settlement devices.

Also shown in Figure 4 are other parameters that will be used later on:

s: maximum settlement;

i: distance to point of inflection of settlement trough, as defined by Peck (1969); referred to as settlement trough width for simplicity;

Vs: volume of settlement trough;

Ve: excavation volume;

π = Vs/Ve (%): percentage of settlement volume.

Settlements of nearby buildings were also controlled. Besides settlements, internal convergence of the tunnels was also measured.

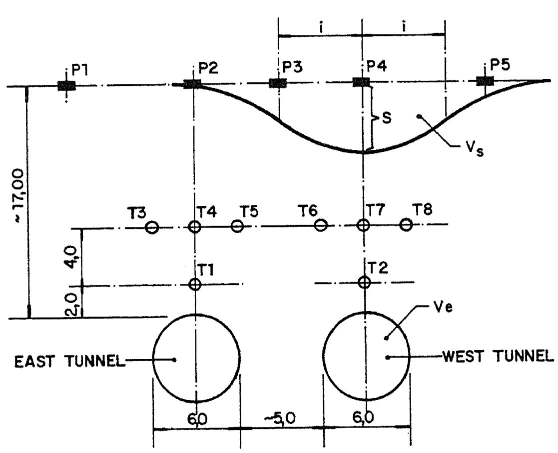

FIGURE 4 - Typical Control Section and Settlement Parameters

In order to speed up the interpretation of instrumention results, a system was developed for storage, retrieval and graphical presentation of instrument readings and general construction occurrences. A network of microcomputers was installed at the construction site, owner's office and engibeers' office so that the data could be remotely transmitted. This system allowed interpretation graphs to be available at the offices a few minutes after reading the instruments at the construction site.

Graphs of readings versus time or readings versus excavation progress can be obtained at scales easily chosen by the user. Annotations of construction occurrences are also shown. Other types of graphs are being implemented now. Domingues et al. (1987) present a general description of the system.

PERFORMANCE DURING CONSTRUCTION AND COST

Eventhough other tunnels have been excavated in soil without steel sets as part of the support both in Brazil (e.g. Negro and Eisenstein 1978; Teixeira, 1985) and abroad (e.g. Ocampo-Franco, 1982), no tunnels had yet been excavated without steel sets for the São Paulo Subway. At the time they were first used, steel sets represented significant progress for the practice of tunnel support. More recently, after other types of support have come up, for instance shotcrete, the use of steel sets has been questioned by some authors (e.g. Kramers, 1978; Rabcewicz, 1979). Important conclusions of the instrumentation program of the Du Pont Circle Station construction, Washington Metro (Brierly and Cording, 1976) show only limited action of steel sets. It was therefore decided to eliminate steel sets at the central portions of the tunnels, except where previous disturbance of the soil mass might cause the need for (a) support shortly after excavation, or (b) support element to withstand concentrated or non-symmetric load.

The West tunnel was excavated first, and only when it reached the end did the East tunnel excavation start. When the face of the West tunnel reached progressive 35 m, settlements started to increase. According to the design, steel sets were not being used for support. It was noticed that the quality of shotcrete (the only support element) was very poor. Time of initial setting and time of end of setting were far beyond design requirements. A large block (4 m wide, 0.9 m high) fell off the roof, and steel sets were again locally adopted. In this mean time, laboratory tests determined that the cement and admixture used were incompatible due to recent increase in the contents of blast furnace slag. Untill new compatible cement and admixture were found, a minimum of 10 hours was established between initiation of subsequent advances, so that the shotcrete could gain enough strength.

Figure 5 shows the contours of equal surface settlements caused by the excavation of the West Tunnel. Values for i, s and π along the tunnel axis are also shown. A deep trough (68.7 mm) can be seen at the location of the unstable block. It is interesting to notice that the

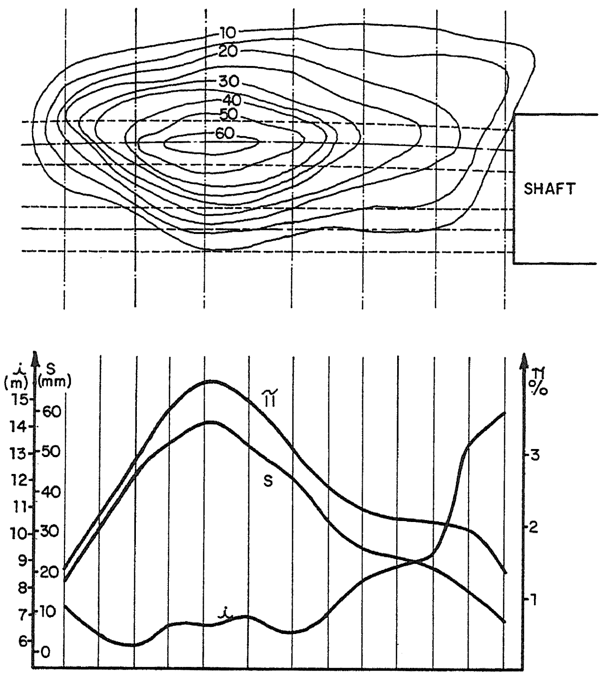

FIGURE 5 - Surface Settlements Caused by West Tunnel: Contours and Parameters i, s and π.

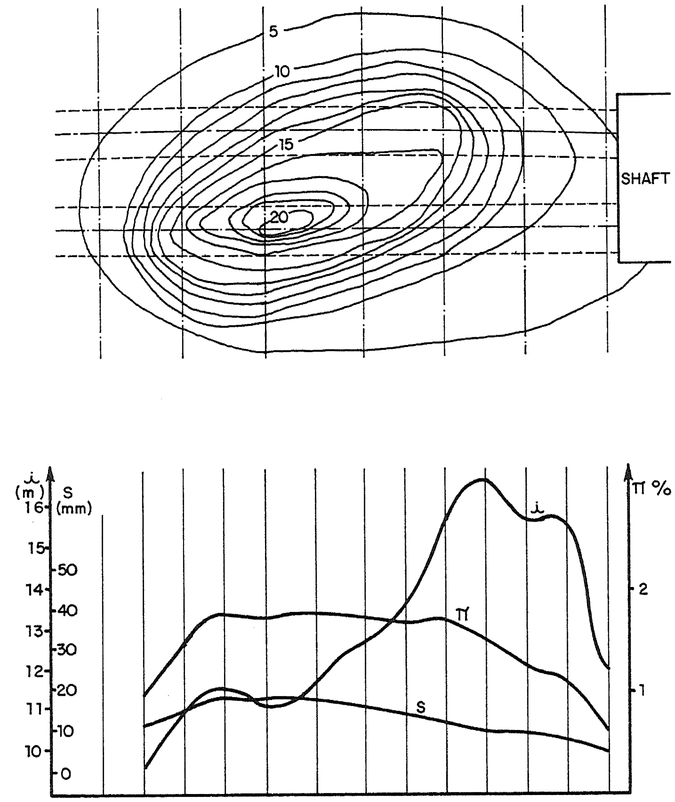

FIGURE 6 - Surface Settlements Caused by East Tunnel: Contours and Parameters i, s and π .

settlement trough becomes narrower at the location of the unstable block, indicating the tendency for plug-like mechanism. Trough width and settlement volume percentage have exactly opposite behaviors (one increases as the other decreases).

When the East tunnel excavation started, there was no more restriction about the time between advances, since new compatible cement and admixture had been found. Steel sets were only used in the initial 10 m according to the design. No problem of unstable block was found. The use of steel sets was conditioned to the maximun observed shear strain in the soil mass, inferred from the reading of the deepest settlement device. The limit was never reached and the excavation progressed successfully without steel sets.

Figure 6 shows settlements caused by the East tunnel. No pronounced peak of π or s exists. It can also be noticed that the settlements are significantly smaller than for the West tunnel. This is true not only at the location of the unstable block, but even in the first 30 m, where no serious problem of shotcrete occurred for the West tunnel.

This tendency is also observed for the deep settlement devices, that do not include consolidation of the soft clay above the West tunnel. East tunnel settlements were only 50% to 65% of West tunnel settlements.

The decrease in settlement caused by the second tunnel is opposite to what had been found by Cording and Hansmire (1975). They compiled data from several tunnels excavated with shield that showed a clear increase in settlement for the second tunnel. It is probable that the already completed shotcrete support of the first tunnel, interacting with the soil mass, is stiffer than the mass by itself. Similar results were found by Celestino et al. (1985), analyzing data of settlements caused by large dimension tunnels excavated in sequences of side and central galleries.

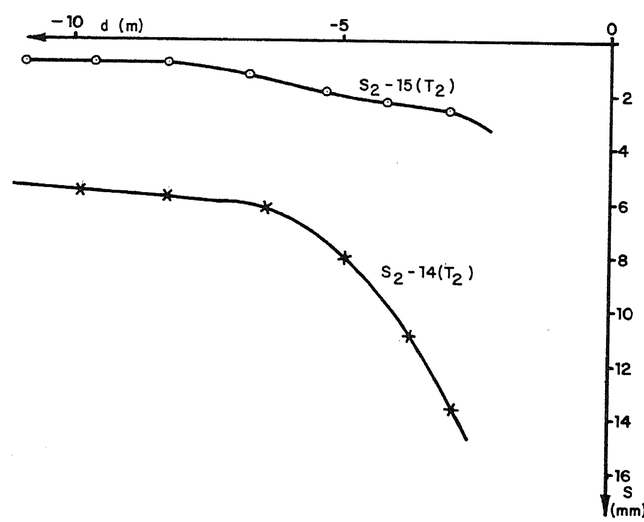

FIGURE 7 - Deep Settlements as a Function of the Distance to Excavation Face: S2-15 Above Pre-Existing Tunnel; S2-14 Above Soil Mass.

Another evidence of this fact was found in connection with the longitudinal arching as the East tunnel approached the dead end of the previously completed tunnel. Figure 7 shows the settlements measured at the deep settlement devices of section S2-15 (above pre-existing tunnel) and S2-14 (ca 10 m before) as function of their distance to the tunnel face. It can be clearly seen again that the interaction between soil mass and tunnel lining shows stiffer behavior (lower settlement) than the soil by itself.

As a result of this observation, the number of steel sets at that location was decreased from what had been specified in the design.

As a result of the measures adopted: (a) no steel sets along 72% of the length of the tunnels, (b) no temporary invert, (c) no ground improvement, and (d) shotcrete for final lining, the cost of the tunnels (excavation, support and final lining including floor slab) came out the lowest ever reached for the São Paulo Subway: only $5,300 per linear meter. For comparison, the cost of the double-track tunnel previously mentioned was of the order of magnitude of $18,000 per linear meter. The first application (single-track tunnels) had a wide variation of cost, due to problems already mentioned above. The lowest cost at problem-free regions was $12,000 per linear meter.

CONCLUSIONS

Despite serious problems with the quality of shotcrete that had to be coped with during the excavation of the second half of the West tunnel, the remainder of the tunnels could be successfully excavated without steel ribs and temporary invert. The cost presented is the lowest for tunnels of the São Paulo Subway.

Settlement data caused by twin tunnels showed tendency contrary to what had been previously reported, i.e. the interaction between the support of the first tunnel and the soil mass seems to contribute to decrease subsequent settements. Settlement caused by the second tunnel was smaller than the ones caused by the first tunnel.

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