SOFT GROUND TUNNELLING IN THE MILAN METRO AND MILAN RAILWAY LINK. CASE HISTORIES.

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SYNOPSIS. The construction of tunnels in a metropolitan area is made complex and delicate by shallow depth and the presence of major surface structures which demand that deformation on the surrounding soil be minimised. Achieving this goal depends heavily on the possibility of deviating the flow of stress in the area surrounding the cavity without producing failure in the soil. In soft ground this is possible only by artificially providing a sufficient amount of shear resistance. Various examples of tunnel construction in soft ground will be shown with special reference to three examples of tunnels completed or under construction in Milan. The first, the Railway Link tunnel, is an example of how excavations are usually carried out in Milan, by the use of soil improvement via chemical and cement injection. The second is a case of the application of jet-grouting technology in the construction of a tunnel in the new Metro Line underpassing a major traffic route. The third concerns the construction of a Railway Link station cavity of unusually large dimension (29 m excavation diameter), and extremely shallow depth (4-5 m); this is being carried out by the "Cellular Arch" method.

INTRODUCTION

The construction of an underground cavity always involves a change in the stress pattern of the surrounding soil. Ground at the excavation face, which has not yet been dug is also affected by these modifications which are 3-dimensional in the frontal area of the tunnel.

At a certain distance from the face the problem can be treated as "plain strain", that is, it is limited to 2 dimensions.

The stability of the cavity and the deformations which its construction causes greatly depend on the soil's ability to bear the new stress pattern without failing. In soft ground the increase in principal compression stress and the simultaneous decrease in lower principal stress can produce soil failure which results in plastic deformation, often in the ground preceeding the excavation face.

With this type of soil it is, therefore, imperative that the shear resistance capacity be artificially improved, in order to avoid any possible instability in the cavity, and to minimise deflection. This is, of course, particularly important when carrying out excavations in an urban area where there are major surface structures.

It can be achieved in two ways:
- providing confining pressure.
- making the ground more cohesive.

The confining pressure can be provided via the installation of resistant structures, for example, lining of shot concrete reinforced with wire mesh, steel ribs and steel fibre. The increase of cohesion is obtained via injections into the ground.

In the past, before current methods of soil improvement were as advanced as they now are, the stability of the cavity could only be assured by a structure strong enough to support a confining pressure near the excavation face.

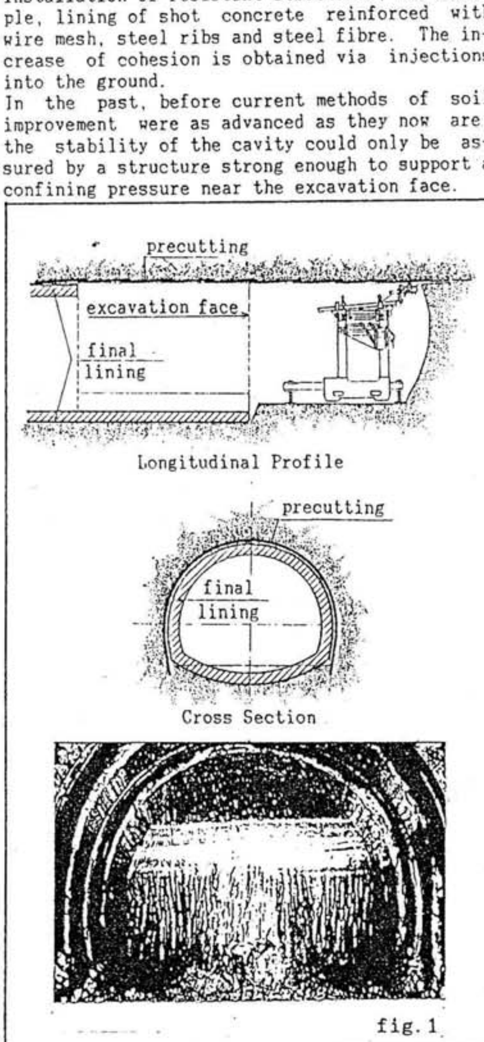
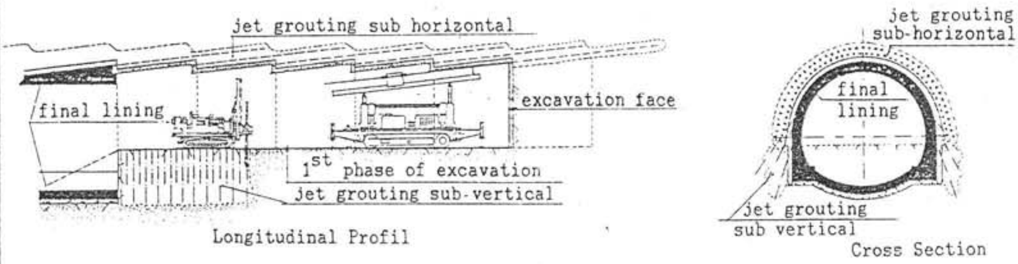


fig. 1



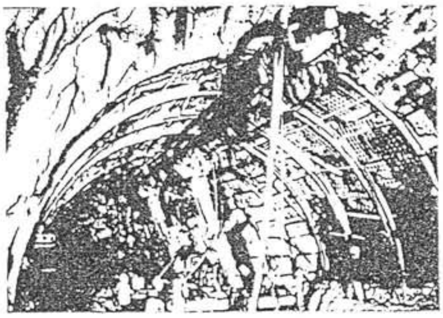
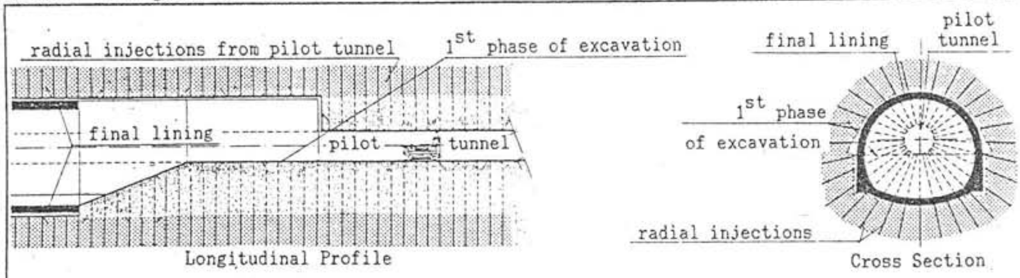


fig.2

In the attempt to guarantee the excavation face's stability, the excavation sections had to be divided and excavated one by one.
Therefore digging was more complicated and difficult and was carried out in several phases. Without going into a description of these operations, and of how they can be counter-productive as regards the control of strain and surface settlements, it must be stressed that the division of the excavation sections does not result in a substantial increase in the safety factor of the stability of the face, or, consequently, in a reduction in final deformation. It should be noted that the basic problem of tunnel construction in soft ground is that of avoiding soil failure in the area in front of the excavation face, as well as controlling the stress and strain behaviour further away from it.
Still using the application of confining pressure, the excavation face can be stabilised via fluid under pressure, as happens for example with earth pressure balanced shields or hydro shields. In between the application of confining pressure and soil improvements are all those methods which bring about a reduction in the principal compression stress at the excavation face, via a pre-built structure which behaves like an arch, directing the deviation of the tension flow deeper. Here we could mention the "predecoupage" method (fig. 1) and consolidation via jet-grouting injections (fig. 2), which we will discuss in more detail later.



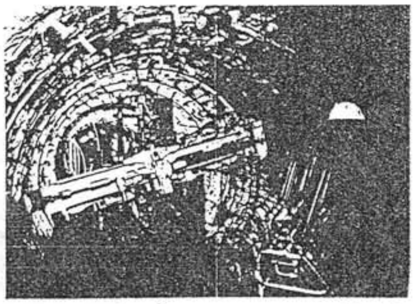
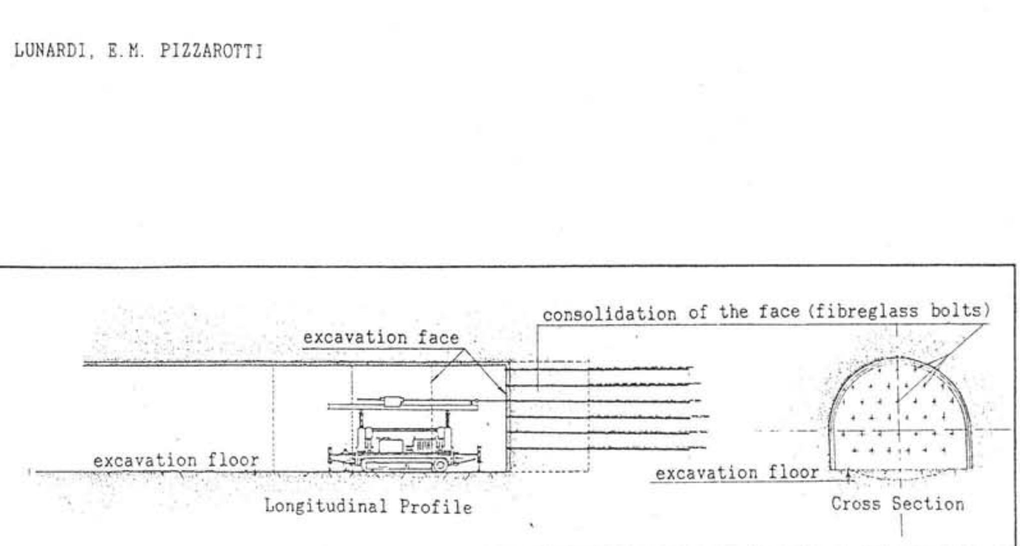


fig.3

Also there is the example of the formation of an arch of consolidated ground surrounding the tunnel (fig. 3), which again is carried out before excavation begins. This is what was done in the first of the case histories, which will be presented shortly.
There are cases in which the only possible solution to stabilise the excavation face is to directly improve the mechanical characteristics of the soil. This can be achieved by, for example, injection. Another alternative, recently carried out by our Company, is the use of fibreglass bolts cemented to the inside of predrilled holes facing out from the excavation face (fig. 4).
Other cases exist, very unusual ones, such as



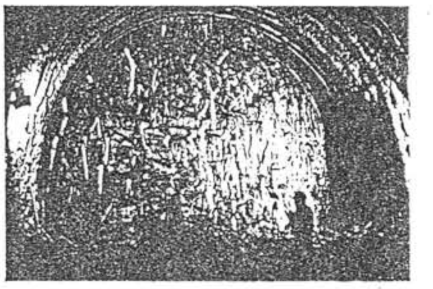


fig.4

that which will be presented in the third case history, where it's not possible to guarantee tunnel stability or control deformation either by applying confining pressure or by improving ground.

UNDERGROUND EXCAVATIONS IN MILAN

The subsoil of Milan is entirely made up of a very thick layer of alluvial deposits (recent diluvium, Quaternary glacial epoch) mostly quite coarse, with alternating levels and strata of variable thickness (from a few centimetres to a few metres) the granulous composition of which varies from areas of silty sand to areas of sandy gravel. The permeability of the soil varies depending on the granulous composition, arriving at a minimum figure of about 10^{-4} cm per sec.

The density of the ground is normally variable according to depth, from quite dense for about the first ten metres (20 to 30 NSPT) to very dense - more than 30 metres depth.

According to these factors, the ground can be classified as not cohesive with an average coefficient of internal friction of 35 degrees and with an elasticity modulus variable depending on the depth, which is best expressed by the function $E = 2000 \sqrt{\sigma_3}$ kg per cm².

The natural weight of the ground can be evaluated as 1.8-2.0 t per m³. The water table is approximately 100 metres above sea level in the entire area. Soil surface levels vary to plus or minus 10 m at around an average of about 120 m above sea level.

The network of underground transport infrastructure currently consists of 3 underground lines, the last of which is in its last stages of construction. The part of line 3 which runs through the centre is already in use.

There is also an underground railway line under construction, (the Railway Link). This will allow commuters travelling by the railway network to cross over directly to the underground system, without overloading public transport overground, or the other railway stations. The first and second underground lines were constructed almost entirely by open excavation. This was possible only because the structures were so near the surface. For line 3 and the Railway Link, which are deeper, the problem of underground excavation had to be faced.

The two above mentioned works took place in the city centre, an area extremely urbanised and, therefore, having a fine network of services mostly at a depth of up to 5 m underground. The method adopted for practically all work on the lines under the city centre consists roughly of the following phases (fig. 5):

1. Construction of a pilot tunnel.
2. Grouting injections into the area surrounding the future cavity, using PVC tubes with valves from the inside of the pilot tunnel.
3. Excavation of the gallery in several phases with simultaneous construction of a pre-lining made up of steel ribs and reinforced shot concrete.
4. Waterproofing and casting the internal lining in reinforced concrete in several

Progressive stages of work on a tunnel with the use of grout injection

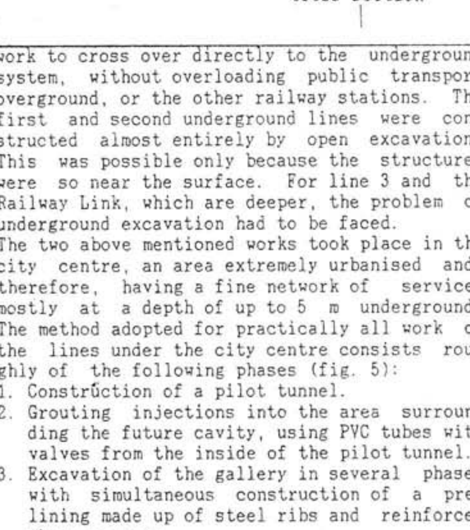


fig.5

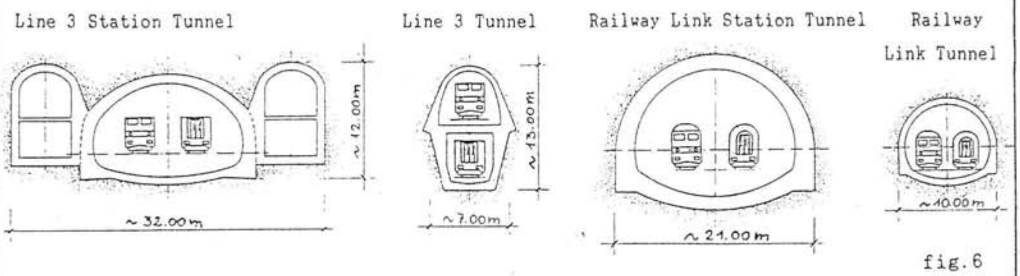
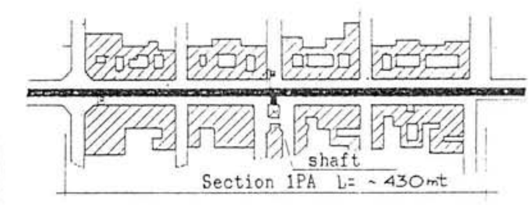


fig.6

phases.
The various grouting mixes used were essentially of two kinds:
CEMENT - triple mix (water, cement, bentonite additive).
CHEMICAL - silicated mix.
The chemical mixes are generally used together with cement. In fact, these latter have notable limits in permeating and treating soil levels with a permeability of less than 10^{-3} cm per sec. in a uniform way.
In general, the grout (cement and chemical) is capable of permeating about 25 to 28 per cent of the theoretical volume of the ground to be treated, corresponding to the volume of the voids in it. In reality, even with very intensive treatment (the injections are placed at a maximum distance of 1.4-1.8 m) the voids are only partly filled by the mix. However, especially in fine soil the improvement of the geotechnical characteristics is achieved via "claquage" that is, filling a fine mesh of fissures with the mix. This causes, as will shortly be seen, the formation of cracks in the soil and, consequently, surface unsettlement.
Subcircular tunnels with double tracks and bullet-shaped tunnels, (with one track above the other), and station tunnels with single or triple tunnel holes have been built using this method (fig. 6).

THE 1PA SECTION OF THE RAILWAY LINK
A typical example follows where the above-mentioned method was used (fig. 7).
The tunnel in question, subcircular with a double track, passes under a major road which bears a great deal of traffic, with a variable overburden of 10-13 m, in a completely built-up area. The tunnel has an internal diameter of 8.80 m. At the beginning of the work, the pilot tunnel had already been completed for the entire stretch of this section. The possibility of building a shaft almost in the centre of the section allowed works to begin outward from the shaft in both directions simultaneously.
After the construction of the shaft and the tunnel which links it to the pilot, the grouting injections had begun (fig. 8).
Note the use of chemical injections together with the cement grout, and the grouted layer beneath the stretches of tunnel under the water table.
This layer's function is to avoid the water reacting with the chemical mix.
For tunnels of these dimensions, the thickness of the grouted layer normally achieved is 2.5-3 m.



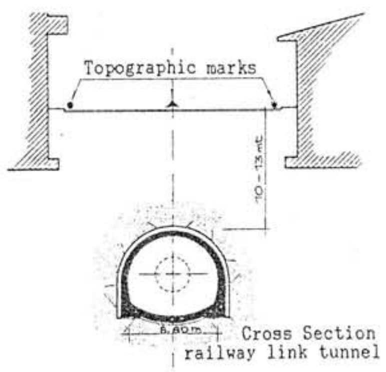


fig.7

Railway Link - Section 1PA

Grout and waterproofing of the ground
distance between sections = 0.90 m

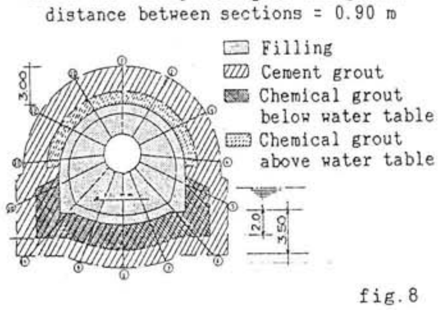


fig.8

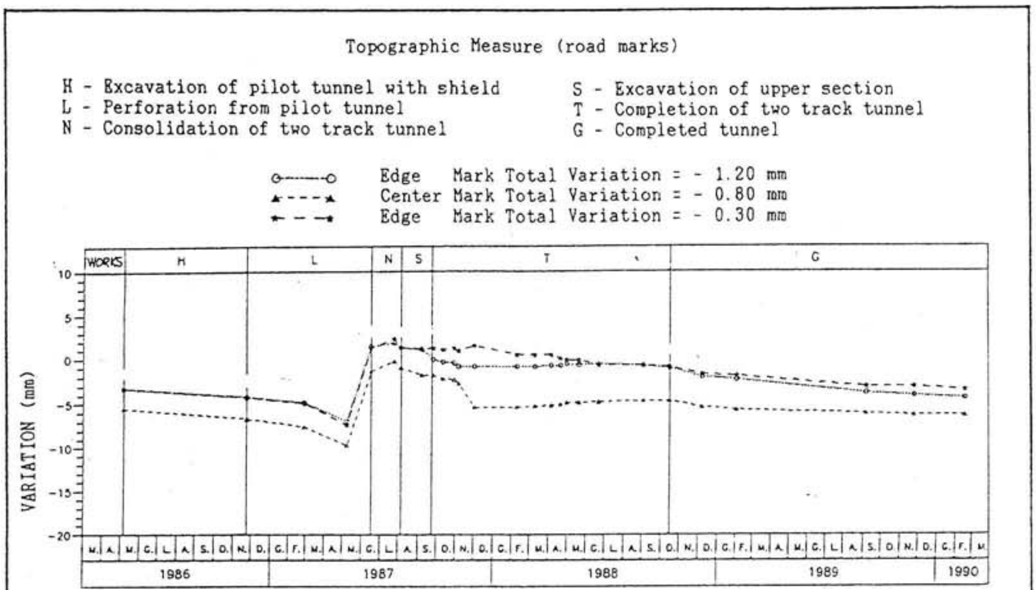


fig.9

Following the completion of the injections, it was possible to proceed from the first phase of the excavation to the floor at the centre of the tunnel with the construction of a lining of shot concrete 20 cm thick, with wire mesh and steel ribs.
The length of each excavation section was equal to the distance between 2 steel ribs, that is, 1 m. There was no problem with stability at the excavation face or in the walls, in this phase of excavation or the next, which consists of the completion of the tunnel section from the floor at the tunnel centre to the base of the posts, and the simultaneous prolongation of the pre-lining. This was done in longitudinal sections of 1 m. The cast lining was completed in two parts: first the invert and then the top section. The thickness of the lining could, at first sight, seem too great. However, the technical norms of MM "Strutture e Infrastrutture del Territorio" S.p.A., the Company which manages construction, demanded that this was carried out for safety reasons:

- not taking into account the improvement of the ground surrounding the cavity,
- considering the weight imposed on the structure by the entire overburden.

For tunnels of this type these limitations demand a lining thickness of 60 cm, reinforced with about 70 kg of steel per m^3 concrete.
Thanks to years of experience on the long-term behaviour of chemical and cement grouting, and contrary to the above-mentioned norms, it was decided, after finite element analysis and carrying out full checks with instruments placed in the lining, to avoid reinforcing the lining for a modest increase in its thickness, as an experiment.
The final lining was, therefore, constructed in simple concrete with a thickness of 70 cm.
In fig. 9, the trend of the surface settlements of the topographic marks positioned according to the cross-section in fig. 7 is shown.
As you will note, the settlements during pilot tunnel excavation are not negligible compared to those which appear during excavation of the major tunnel. The final settlement value falls approximately within that predicted at the planning stage, with a finite element analysis of the excavation phases of the tunnel. In this latter, figures of elasticity modulus of grouted soil were about 2-3 times those of the natural soil.
The correspondence between calculated and actual settlements is an indication of the success of the method, and also of the long-term static benefits of injected ground.

THE RINGROAD EAST SUBWAY IN SECTION 3 OF THE MILAN UNDERGROUND LINE 3

A subway under a major traffic artery was planned as part of an underground tunnel (fig. 10). The previous experience of the same Contractors in jacking with box-shaped structures had produced too many settlements. Thus we avoided choosing this method. The method of injection from a pilot tunnel described above was also avoided because the very reduced overburden (max. 7 m).
In fact, when overburden is small, and thus the natural confine produced by the overburden pressure is decreased, it becomes more difficult to obtain a homogenous treatment, to avoid injected mixes seeping out, and to keep the unsettlements under a certain acceptable level. Thus we turned to technology for help and used the jet-grouting consolidation method, which allows us to achieve, through very high pressure (400-500 bars) injections of a mix of water and cement, grouted soil volumes of controlled dimensions. Such technology can be applied with an extremely reduced overburden in as much as

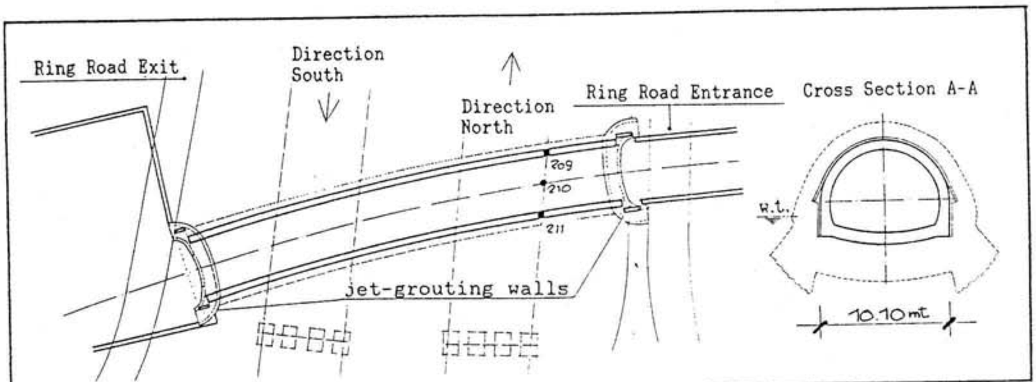


fig. 10

the effect of the injection is felt at not more than a distance of about 60-80 cm from the injection axis. The result is, then, a volume of grouted soil in a column form, surrounded by "claquage", which undergoes a heavy increase of density.

In the case in question, this technology was adopted both for the sub-horizontal injections, to be carried out in advance every 6 metres of excavation, with a length of 12 m and therefore with a constant double overlap, and for the subvertical injections at the side of the posts, carried out from the half section after the first phase excavation (fig. 11).

The bottom of the tunnel was treated, (still acting on the middle section) with cement and chemical injection from tubes with valves with the object of waterproofing for the lower part of the tunnel which was immersed in the water-table. Jet-grouting was also carried out in advance to treat the area under the base of the pre-lining (made up of a layer of shot-concrete reinforced with steel ribs and wire mesh), so that a subfoundation which would not settle could be prepared before the steel ribs were put into operation.

It is interesting to note that the underground section was carried out between two open excavation areas very close to the street embankment. Therefore there was also the problem of beginning the tunnel, by means of subvertical excavation walls. Even in such a case, jet-grouting has proved an advantageous solution to the problem of the initial stages of excavation, allowing the construction of subvertical walls of a notable thickness, made on grouting soil, without anchors, which were than part-demolished during the tunnel excavation.

This latter was carried out on two opposite faces, through the alternation of injection in advance of the top section, and excavation of 6 metres. These operations continued weekly, thereby the excavation of the top section required 16 weeks. Following the excavation of the first areas which showed a tendency for the face to collapse during the injections in advance, some jet-grouting columns were carried out at the excavation face, and permitted us to resolve the problem of face stability definitively. Notwithstanding that the upper part of the tunnel was dug through the street embankment and that in general the soil contained large percentages of silt and silty sand, it was possible to finish the work with settlements which have not in the least affected the flow of traffic on the surface.

Soil deformations and settlements have constantly been checked by geotechnical instruments which have provided the representations appearing in the diagrams (fig. 12).

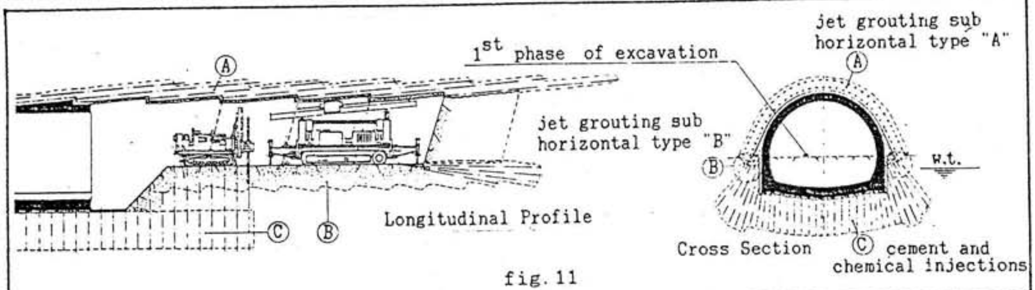


fig. 11

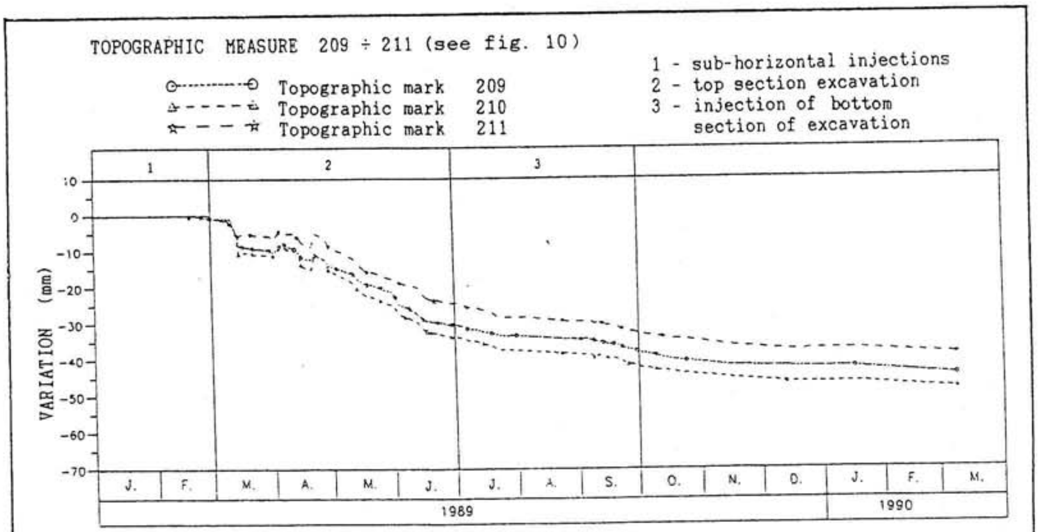


fig. 12

THE "VENEZIA" STATION IN THE 2PB SECTION OF THE RAILWAY LINK

The excavation of cavities in loose ground, exceeding a certain dimension and at shallow depth, is not feasible using traditional methods: in fact it is not possible to consolidate the ground around the cavity in such a way that homogeneity and resistance is sufficiently guaranteed.

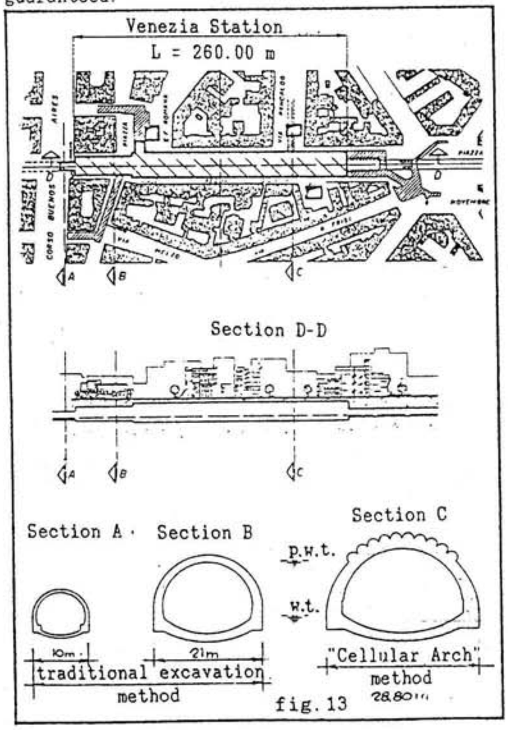


fig. 13

This is the case of the "Venezia" Station of the Milan Railway Link (fig. 13) currently under construction; the external diameter of excavation (28.80 m), the limited overburden (4-5 m), the interference with underground services (particularly an electrical duct of 220.000 Volts capacity, which is extremely sensitive to possible land movements) and the nature of partially saturated soils, are the numerous problems of the construction of this cavity.

To overcome these difficulties a new building method has been developed, the "Cellular Arch", which permits the creation of the complete ground supporting structure and cavity lining even before the excavation is started.

The "Cellular Arch" is a composite structure similar to a semi-circular section grid, in which the longitudinal elements (cells made of reinforced concrete pipes) are connected by means of large ribs (arches).

The main supporting structure in the "Cellular Arch" system is formed by pipes, fixed in the ground along a semi-circular profile and parallel to the longitudinal axis of the tunnel, and transversal arches (fig. 14).

In this way a semi-cylindrical reinforced concrete grid, able to guarantee the perfect canalization of boundary cavity stresses, takes place, artificially providing an "arch effect", necessary to the short and long term stability of the work.

Before applying the Cellular Arch Method to the Venezia Station tunnel, which is 220 m long, and consists of 37 arches dividing 36 fields, each field being 6 metres in length, a test was carried out to verify the feasibility of the new Cellular Arch Method, by fixing in the ground three reinforced concrete pipes.

The soil around the pipes was treated with different grouting mixtures until the best conditions for pipe jacking were obtained.

During the test deformations in the surrounding soil and surface displacements were checked.

As a consequence of the positive results of the

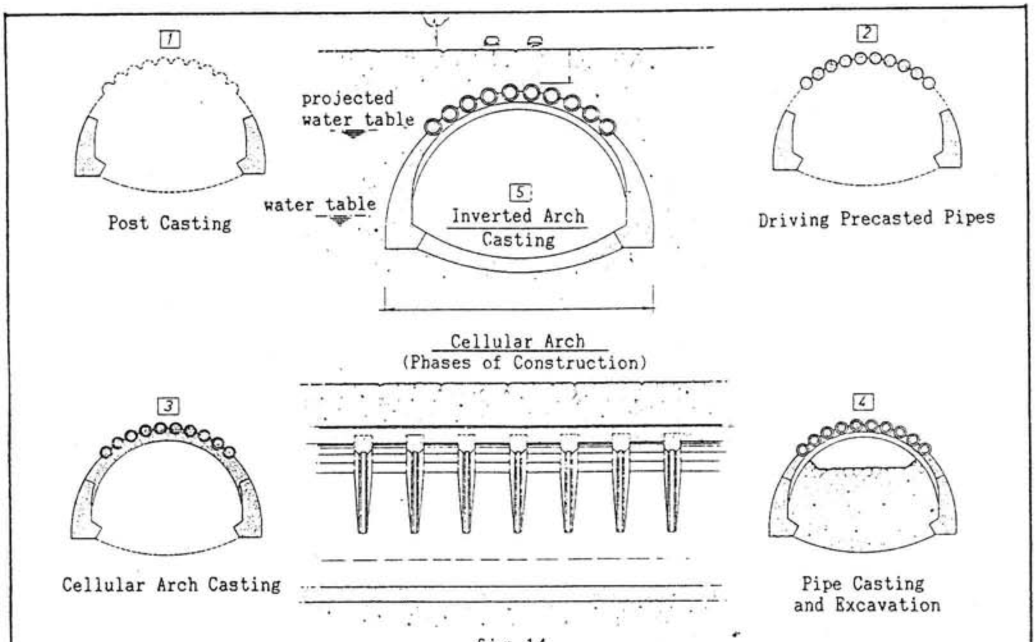
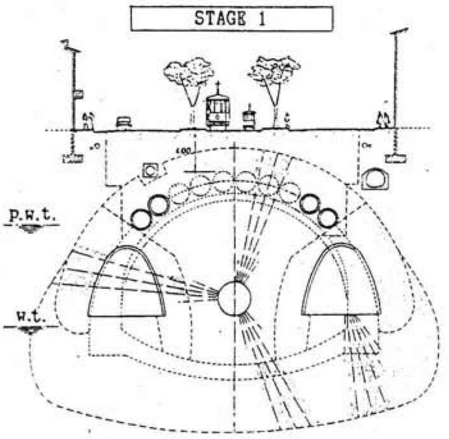


fig. 14

experiments, the final design was completed and construction began.
The construction of the tunnel with the "cellular arch" system consists of these main phases (fig. 15):

a) grouting of the ground around the perimeter of the side drifts, working from the central pilot tunnel; it is worth noting that the grouting of the ground in proximity with the crown of the tunnel creates only a light cohesion in the soil, in order to avoid collapse of the microtunnel excavation face;
b) excavation of the side drifts: these tunnels, 11 m high and 7 m wide, were carried out to allow the construction of the posts. After the first phase of excavation, to the level of the water table, grouting of the 4.5 m thick layer under the inverted arch was carried out. An external layer 2 m thick of soil grouted only by a cement mix was treated to avoid the water reacting with the chemical mix. Subsequently a 4 m excavation was performed to the base of the posts;
c) driving pipe shaft (fig. 16): in an independent site located at the crown profile of the final tunnel, the driving pipe shaft for the hydraulic jacking of the pipes was prepared. The pit, 10 m long and 12 m wide, has walls constructed by means of soil improved by the jet-grouting method;
d) construction of the crown's microtunnels: pipe fixing is carried out by means of equipment consisting of a metal shield 8 m long, divided into three parts, of which the first, 1.3 m in length, movable with a cutting edge, allows the operator to control vertical and horizontal plane movement. The shield has a computer controlled front drilling head, with a circular section about 3 cm smaller than the external shield section, in order to limit the formation of cavities outside the pipes when drilling. The dimension of the pipes are: length 2.0 m, diameter 1.8 m, thickness 0.15 m;
e) construction of the final posts: before casting the posts the external walls and bottom of the gallery were waterproofed with PVC sheets. Finally the moulds, steel reinforcement and the casting take place;
f) excavation of the transversal arches starting from the microtunnels. The excavated material is dumped into the side drifts;
g) positioning of arch moulds and steel reinforcement and filling the longitudinal microtunnels and the transversal arches with concrete. The completion of the arches is the last and most difficult phase in the cellular arch system. In this case it was more difficult because of the triangular section, required for architectonic reasons, and because of the tools needed for the suspended mezzanine. The stages involved in the construction of the arches are the following:
   - the lower half of the pipe is cut with a special cutting disc. The pipe is divided into sections which are removed with the excavated ground during the formation of the arch housing;
   - excavation starts from the lower outer pipes and proceeds towards the top;
   - insertion of special arch moulds, starting from the side drifts;
   - location of steel bars and of mezzanine suspension tools;
   - concrete casting.

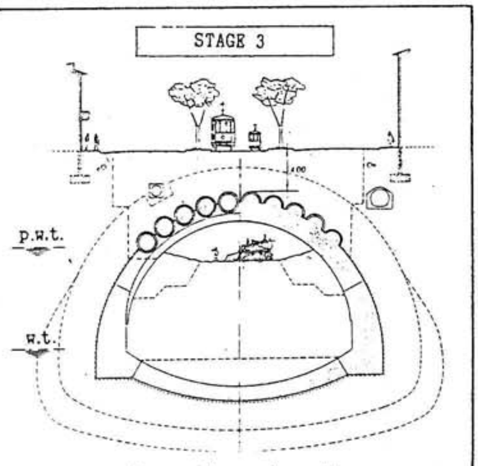
A special and very complex mould was produced to satisfy the architectonic requests



• Grouting pipe and side drift zone.

• Pipe driving starting from sides up to crow and completion of side drift grout.

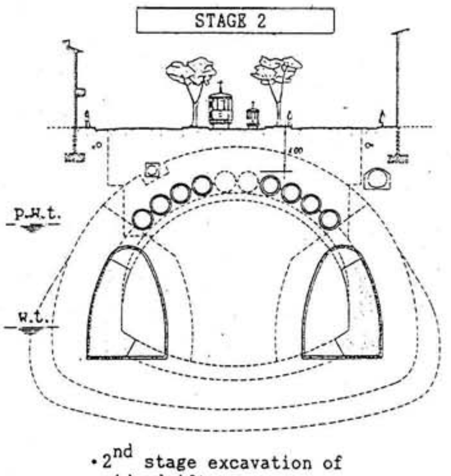
• 1st stage excavation of side drifts and completion of grout.



• Excavation and casting arches and tubes

• Crown excavation in various phases.

• Invert excavation reinforcement and concrete casting of invert.



• 2nd stage excavation of side drift.

• Waterproofing layer of post area; reinforcement and concrete casting of posts.

fig. 15

for the arch;

h) excavation of the final section of the tunnel under the protection of the already active cellular arch;

i) inverted arch casting.

The main advantage of this system in respect of traditional methods is the way the initial soil stress pattern is transformed to the final one without soil failure. The common excavation techniques depend on the three dimensional effect at the excavation face, which contributes to the short term stability of the tunnel. On the other hand, the cellular arch, neglecting the effects of the face, allows minimisation of deformation. In this way shallow tunnels with large diameters can be completed without causing significant vertical displacements. From this point of view, the monitoring programme of the surface displacements, of the deformations in proximity to the gallery, of the existing buildings and of the stresses in the lining during the execution of the whole work, is extremely important.

The programme includes (fig. 17):

- topography check of the marks of a topographical net;
- levellometers to measure the evolution of displacements along the perimeter of the adjacent buildings;
- incremental extensometers and inclinometers located as shown in figure;
- checking points to detect convergence;
- pressure cells and electric transducers for evolution of stress level in the lining.

In fig. 18 the values of the vertical displacements of the topographical marks are shown, situated as indicated in the cross-section in fig. 17.

As you will note, construction has only produced unsettlements due to injections. Currently work has reached stage g, mentioned above.

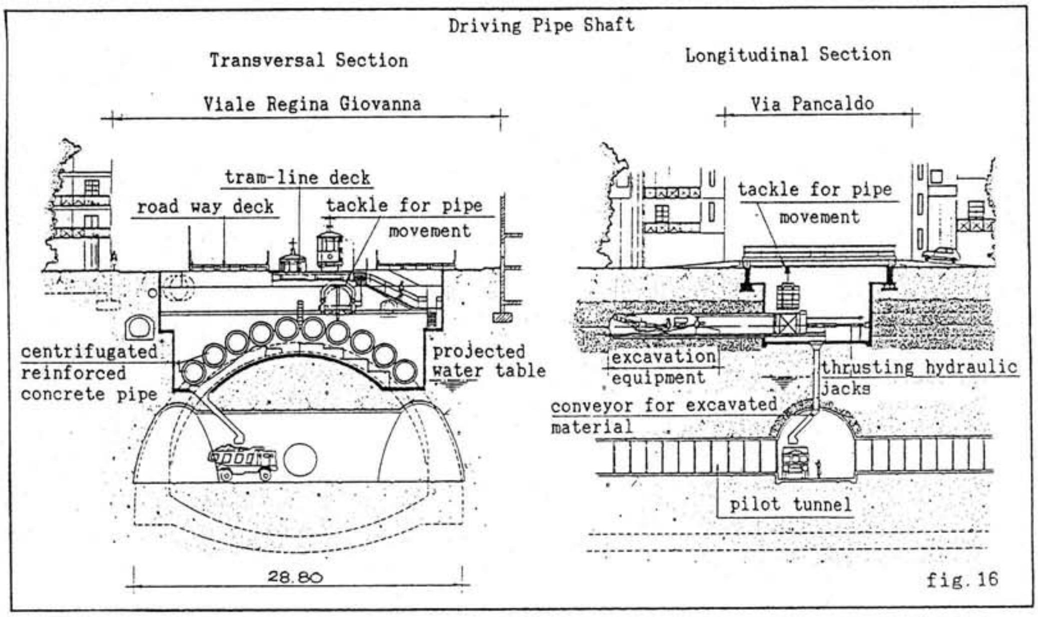
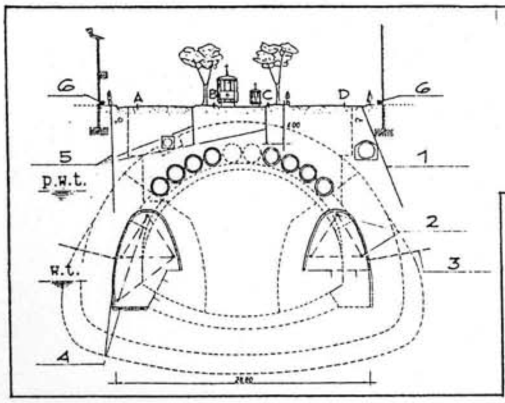


fig. 16



1. inclinometer
2. convergence bolts
3. multibase extensometers
4. pressure cells
5. incremental extensometers
6. levellometers
— topographic marks : A, B, C, D

fig. 17

TECHNICAL DETAILS AND DIAGRAMS REPRODUCED BY KIND PERMISSION OF MM "STRUTTURE E INFRASTRUTTURE PER IL TERRITORIO" S.P.A. CONSTRUCTION MANAGERS OF WORKS PRESENTED.

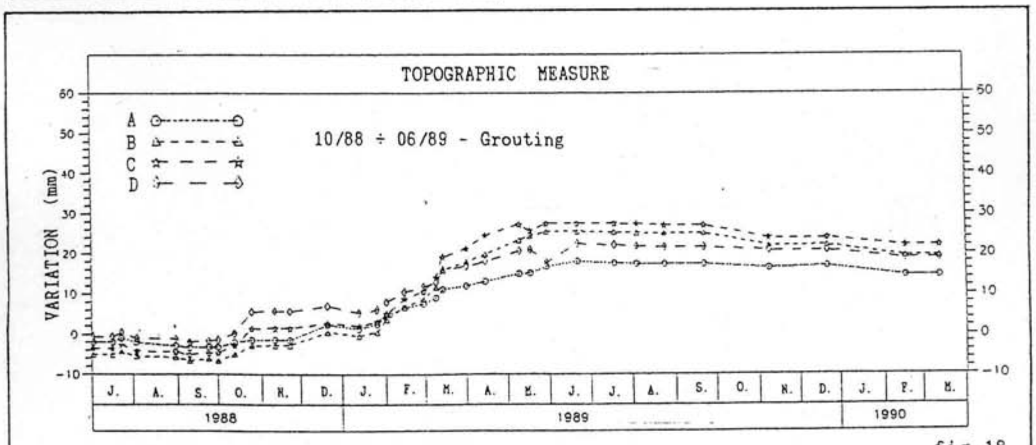


fig. 18