C E N T R A L   I N S T I T U T E   F O R   A D V A N C E D   T R A I N I N G

O F   C O N S T R U C T I O N   E N G I N E E R S

(T S M I P K S)

R U S S I A N - I T A L I A N   S E M I N A R

TUNNELING IN SOIL

Design, Construction Materials and Monitoring

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INTRODUCTION

The tunnel excavation brings changements in the soil balance conditions. As the face advances the deviation of the flow of the natural stress, it occurs at the face and behind the face. This always involves a changement in the stress pattern of the sorrounding soil, passing from a triaxial stress condition, in the frontal area of the tunnel, to a biaxial stress condition, at a certain distance from the face. The increasement, in the principal compression stress and the simultaneous decrease in the lower principal stress (confining pressure) can be beared from the soil, which gives way to an elastic condition, or can produce soil failure which gives results in the plastic deformation, often in the ground preceeding the excavation face. That means that the stability of the cavity and the deformations which its construction cause, greatly depends on the soil's ability to bear the new stress pattern without failing.

In soft ground it is, therefore, imperative that the soil strength should artificially improve, in order to avoid any possible instability in the face and in the cavity, and to minimize the deformation. This is, of course, particularly important when carrying out excavations in an urban area where there are major surface structures.

Stabilization techniques for underground works have been recently provided with new operative systems, which are named actions for preservation. Their aim is to preserve the face and the cavity integrity as much as possible: the excavation face is a structural element, whose stability is essential for a successful work.

It can be achieved in two ways:

- providing confined pressure;

- making the ground more cohesive.

The confining pressure can be provided via the installation of resistant structures ahead of the tunnel face to support the surraunding soil mass during tunnel excavation. The increase of cohesion is obtained throughout injections into the ground.

In the past, before current design and construction methods were advanced as they are now, the engineers studied the stability of the cavity dimentioning a lining strong enough to support a confining pressure near the excavation face. The face's stability in soft ground was garanteed by dividing the excavation sections and excavated them one by one (as forseen by NATM). Therefore digging was more complicated and difficult and was carried out in several phases.

These methods often bring about large final deformations along the profile of the excavation and heavy forces, on the lining, whose effects are often uncontrollable. The poor knowledge of the involved mechanism led, in the past, to consider the face problems in an inadeguate manner. Infact, only the impossibility of carrying on the excavation was the clearest result of the core instability, disregarding the possible consequence on short and long-term stability of the cavity.

It should be noted that the basic problem of tunnel construction in soft ground is to preserve the core integrity on the working 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 stabilized throughout fluid under pressure, as it happens for example with earth's pressure balancing shields or hydroshields.
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, throughout pre-built structure which behaves like an arch, directing the deviation of the tension flow deeper. Here we could mention the "predecoupage" or "Premill" method and consolidation throughout jet-grouting sub-orizontal.

The first one consists in a concrete shell performed around the tunnel section before the letter is excavated. The shell is made by cutting a slot with a saw-type milling machine that rotates along a mobile frame shaped to the tunnel profile.
Each slot is immediately filled with high-strengh fast-hardening shotcrete. The standard length shell range is between 3 and 5 metres while the thickness is between 18 and 30 cm. In this way a concrete pre-lining ahead the face is made which is able to grant a radial pre-confinement of the ground, preventing its loosening.
Once the advancing face is passed, it is strengthened by casting quickly the invert and than the final lining.

The jet-grouting consolidation method allows us to achieve, through very high pressure (400 $\div$ 500 bars) injections of a mix of water and cement, grouted soil volumes of controlled dimensions. In adesion less soil a grouting soil arch is formed before the excavation strage by jet-grouting sub-horizontal columns carrying them out near each other, around the tunnel perifery.

There is also the example of the formation of an arch of consolidated ground surrounding the tunnel which again is carried out before excavation begins.

There are cases in which the only possible solution to stabilise the excavation face is to improve directly 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 pre-drilled holes facing out from the excavation face. This is what was done in the first of the case histories, which will be presented shortly.

Other cases exist, very unusual ones, such as that which will be presented in the last case history, where it's not possible to guarantee the tunnel stability or to control deformation either by applying confining pressure of by improving ground.

Now we can see some case histories in which all these methods are been applied.

TUNNEL FACE REINFORCEMENT: THE ROME-FLORENCE SUBJECT OF THE HIGH SPEED RAILWAY LINE

The first application of this method was in Italy in 1988 during the construction of the last lots of the Roma-Firenze high speed railway line, stretch Arezzo South-Figline. This stretch runs through the Valdarno lacustrine basin, characterized by the deposition of soft silty-sandy and clayey soils.

The soil investigations along the tunnel layout had shown the prevailing presence of saturated sandy-silty soils exhibiting a low consistency.

The extrusive tests had proved the instability of the heading even under a small overburden.

These soil conditions had caused serious difficulties during the divided section excavation in the adjacent lot tunnels, involving uncontrollable convergencies on the excavation profile.

In order to avoid prosecution in already collapsed soil it was felt that it was necessary to increase as much as possible the excavation rate, with suitable measures to preserve the core integrity on the working face.

Consequently the design has been based on a full face excavation with a previous consolidation by an adequate pattern of fiberglass tubes thoroughly cemented into the ground ahead of the face, immediate reinforced shotcreting, and invert casting at a maximum distance of about 1.5 times the tunnel diameter.

This procedure enables to stop promptly or to prevent the deformations involved by the excavation, since soil's release around the cavity (unavoidable when divided section tunnelling is carried out) is prevented and excavation rates may be increased and kept constant.

The stabilization is aimed to preserve the natural soil structure, acting ahead of the face when this is a fix point in a triaxial constraint condition.

Besides a soil strength improvement, the function is to prevent an excessive decrease of confining pressure when the cavity outline deformations may be still under control.

Since in the low overburden sections the cave-in risk was greater, the design has provided for the realization of a bearing arch formed by consolidated soil around the cavity perifery after each excavation stage, the core stabilization.

The mean rate of excavation and pre-lining (with reference to each beading) was 10 m per week, that is about the double in relation with the rate recorded from divided section tunnelling.

After this important experience other application were done and some of these combining the Mechanical Precutting Technology.

MECHANICAL PRECUTTING TECHNOLOGY

In the mechanical precutting method a concrete shell is performed around the tunnel section before the latter is excavated. The shell is made by cutting a slot with a saw-type milling machine that rotates along a mobile frame shaped to the tunnel profile. Cutting starts from the side walls and progresses up to the roof apex. Each slot is immediately filled with high-strength fast-hardening shotcrete. Cutting extends below the tunnel's base level so as to guarantee that the foot of the shell is firmly anchored. Each Premill shell is 3-5 m deep, 15-35 cm thick, and is shaped liked a truncated cone such that it overlaps the previous one. In this way an arch is built into the soil, ahead of the tunnel face, to confining the surrounding soil mass during tunnel excavation. Mechanical precutting secures its objective of appreciably improving soil behaviour during excavation through a dual-acting mechanism:

- the radial pre-confining pressure, by avoiding drastic decrease, even temporary, of the minor principal stress r3, reduces the plasticisation around the excavation since the flow of stresses is channelled along the tunnel boundary through a "medium" with better characteristics than those of the natural soil. In this way an arch effect is developed ahead of the excavation face ensuring the transverse stability of the tunnel;
- the shotcrete shell protruces about 4 m beyond the face and serves to push the area in which any extrusion phenomena may originate further forward into the mass of soil, thereby delaying the occcurrence of such phenomena and reducing their effects.

Good stability of the excavation face is a vital prerequisite for this method's success. If the Premill shell by itself cannot guarantee this stability, it must be supplemented by face reinforcement and, where pore pressures are present, by using a ring of drainage holes.

The precutting technique was applied during the 1970s in France for divided-section profile headings or in small diameter tunnels in rocky lithotypes.
Its main purpose was to prevent the spread of vibrations set up by blasting.

It was in Italy, however, that the first application of precutting technique was carried out on full sections (85 m2), in rail tunnels along the Sibari-Cosenza line on cohesive and semi-cohesive soil. The method was subsequently used, again in Italy, on over 6 Km of tunnelling and proved successful.
It has served for advancing full-section headings at a production rate in the order to 3 m/day in tunnels of 70-100 m2 in section. It has shown itself compatible with a work schedule, it ensures safety during operations, and has succeeded in soil where other techniques have failed.

Continuous checks made during excavations have demonstrated that this method offers appreciable advantages in the way it limits deformation. Accordingly, in the case of shallow tunnels, it is effective in containing surface settlements. The average values noted for convergence were in the order of centimetres and those for surface settlement, in the shallowest stretches, were in the

order of millimetres. As such, they were lower than the values that normally characterise traditional excavation methods in similar soils.

TUNNEL IN URBAN AREA - 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 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 5 m underground. The method adopted for practically all work on the lines under the city centre consists roughly of the following phases:

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 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 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.

THE 1PA SECTION OF THE RAILWAY LINK

A typical example follows where the above-mentioned method was used in the construction of the 1PA section of the Railway Link.

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.

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.

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 net 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

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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 final lining was constructed in simple concrete with a thickness of 70 cm.

In figure, the trend of the surface settlements of the topographic marks positioned on the street.

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.

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. 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 of 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. Such technology can be applied with an extremely reduced overburden as long as the effect of the injection is felt at a distance of no more then about 60-80 cm from the injection axis.

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.

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 net), 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 the excavation of 6 metres. 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.

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

The excavation of cavities in loose ground, exceeding a certain dimension 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.

This is the case of the "Venezia" Station of the Milan Railway Link currently under construction; the external diameter of excavation (28.80 m), the limited overburden (4-5 m), the interference with underground services 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.

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.

The Cellular Arch Method applyed to the Venezia Station tunnel, which is 220 m long, consists of 37 arches dividing 36 fields, each field being 6 metres in length.

The construction of the tunnel with the "cellular arch" system consists of these main phases:

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: 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 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.

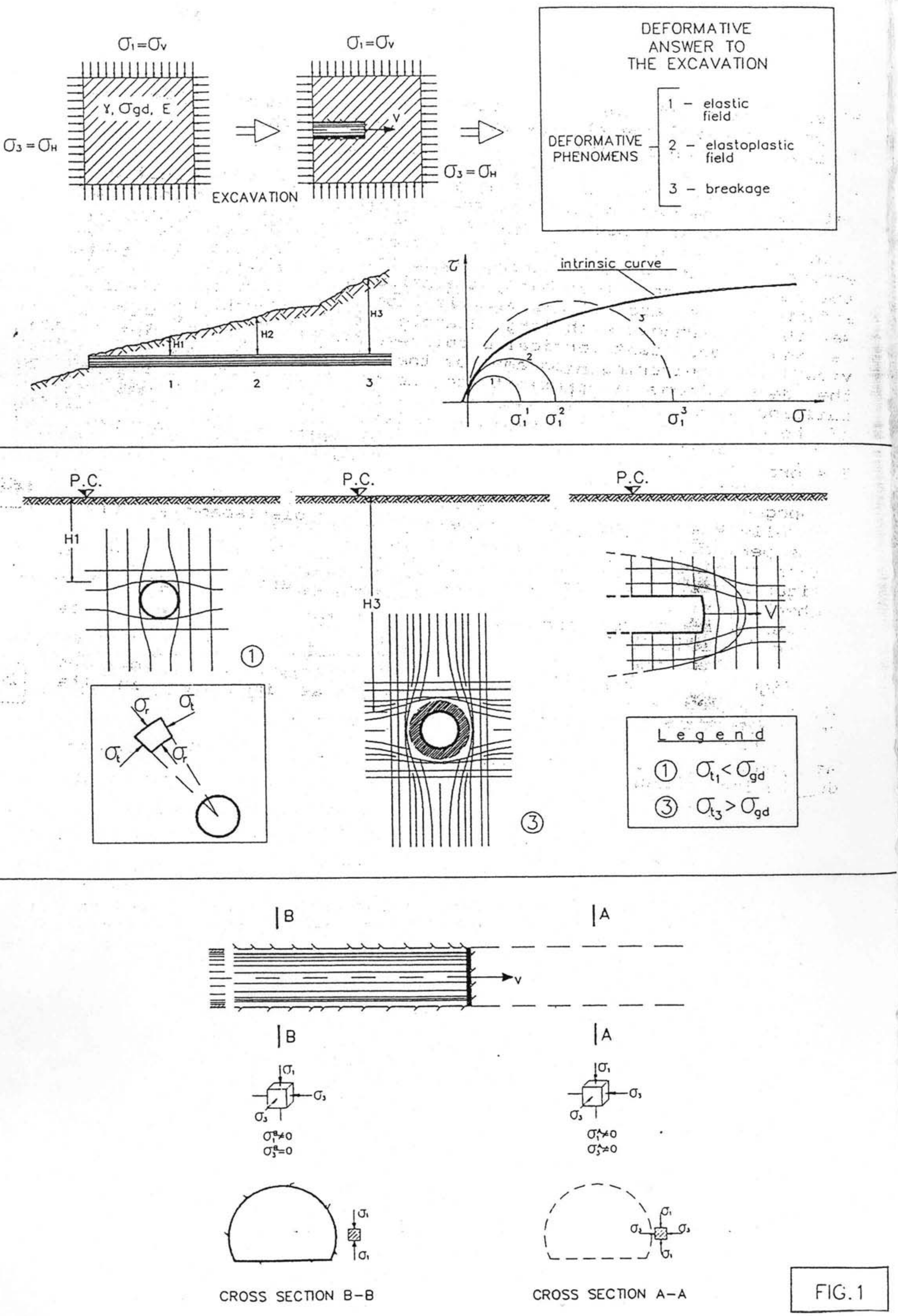
The main advantage of this system in respect of traditional methods is the way the initial soil stress pattern is transformed to the final one without soil failure. The common excavation techniques depend on the three dimensional effect at the excavation face, which contributes to the short term stability 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:

- 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 figure the values of the vertical displacements of the topographical marks are shown, situated as indicated in the cross-section.

As you will note, construction has only produced unsettlements due to injections.



σ1=σv
γ, σgd, E
σ3=σH
EXCAVATION
V
DEFORMATIVE ANSWER TO THE EXCAVATION
DEFORMATIVE PHENOMENS
1 - elastic field
2 - elastoplastic field
3 - breakage
H1
H2
H3
1
2
3
τ
intrinsic curve
σ
P.C.
σr
σt
V
Legend
① σt1 < σgd
③ σt3 > σgd
B
A
σ1B≠0
σ3B=0
σ1A≠0
σ3A≠0
CROSS SECTION B-B
CROSS SECTION A-A

FIG. 1

NEW AUSTRIAN TUNNELLING METHOD

(Rabcewicz–Pacher)

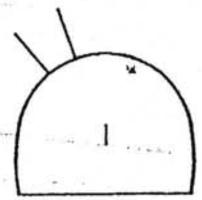
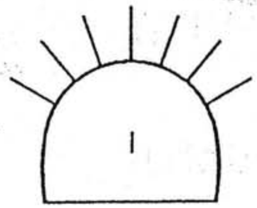
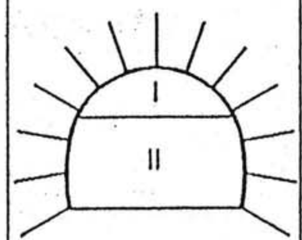
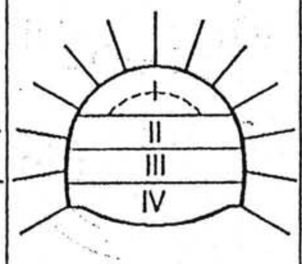
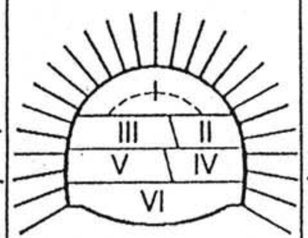
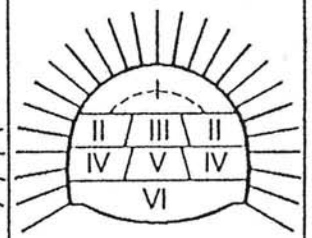
CLASSES OF ROCK MASS	I	II	III	IV	Va	Vb
						

FIG.2

SURFACE SETTLEMENTS

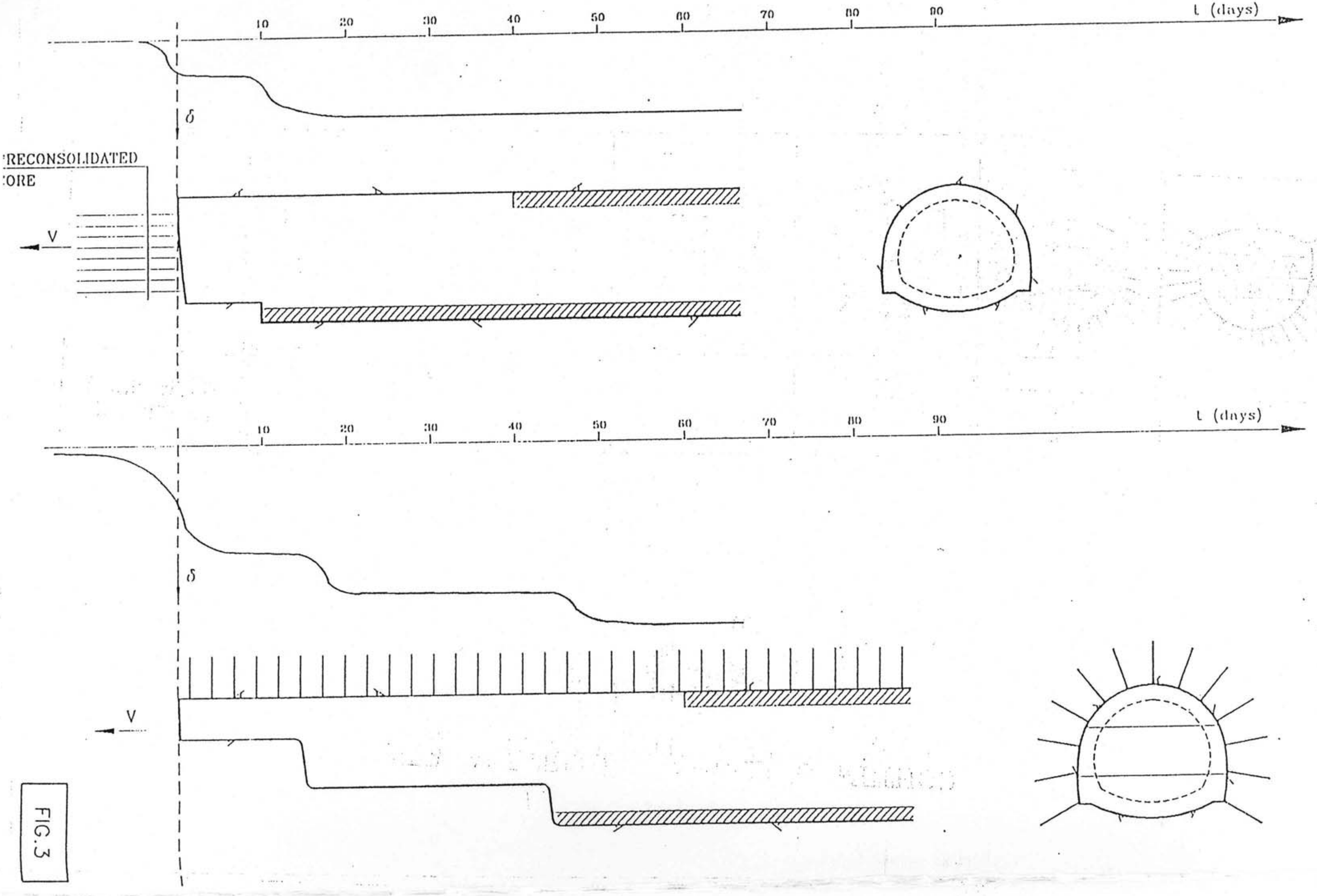
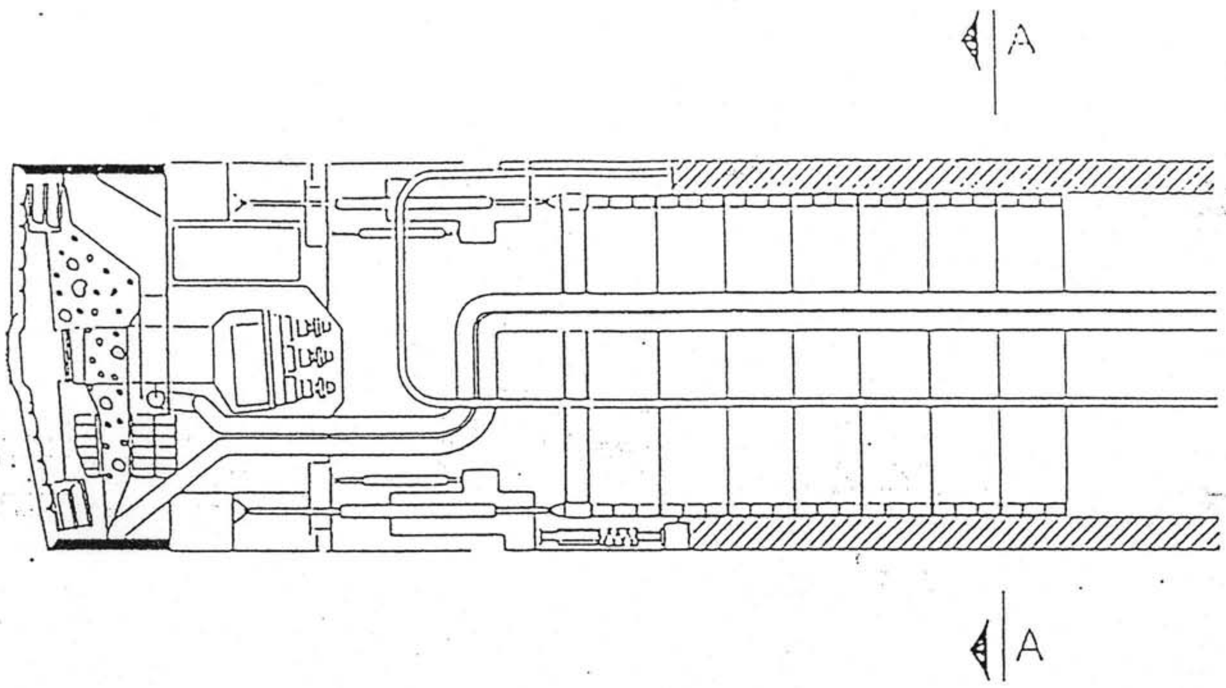


FIG.3



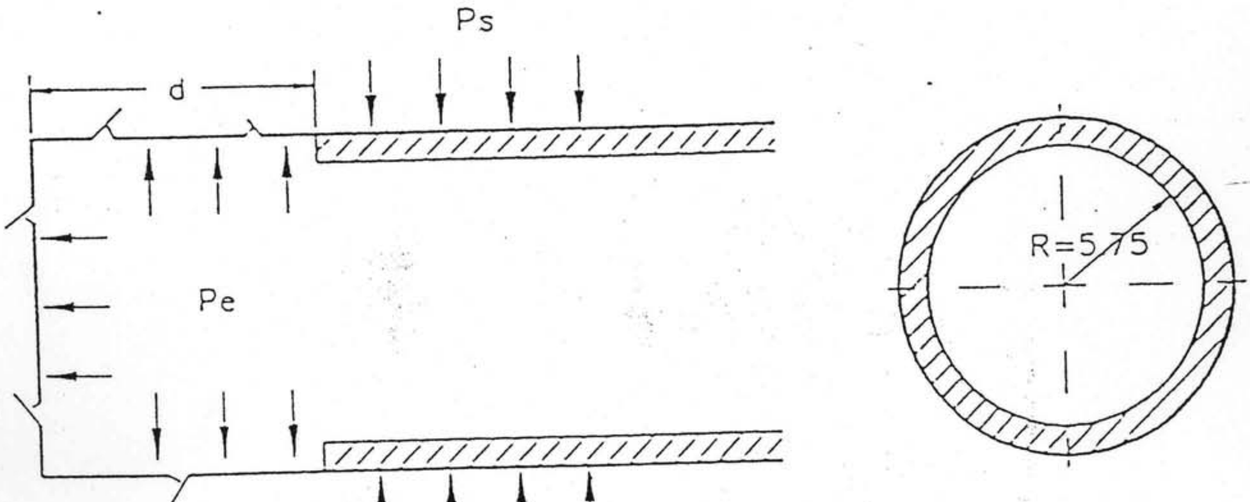
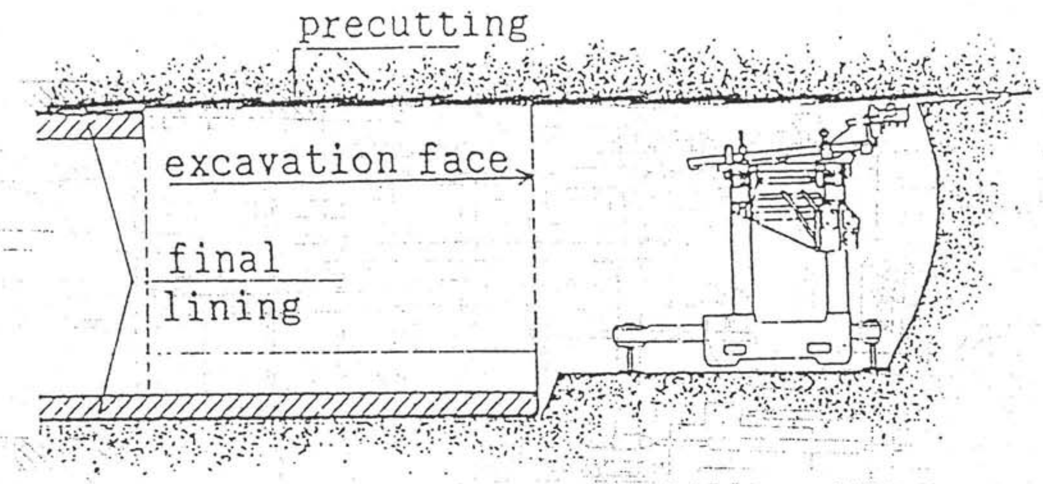
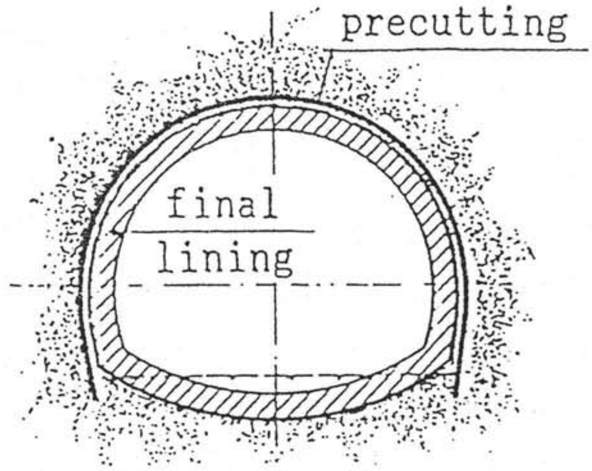


FIG.4



Longitudinal Profile



Cross Section

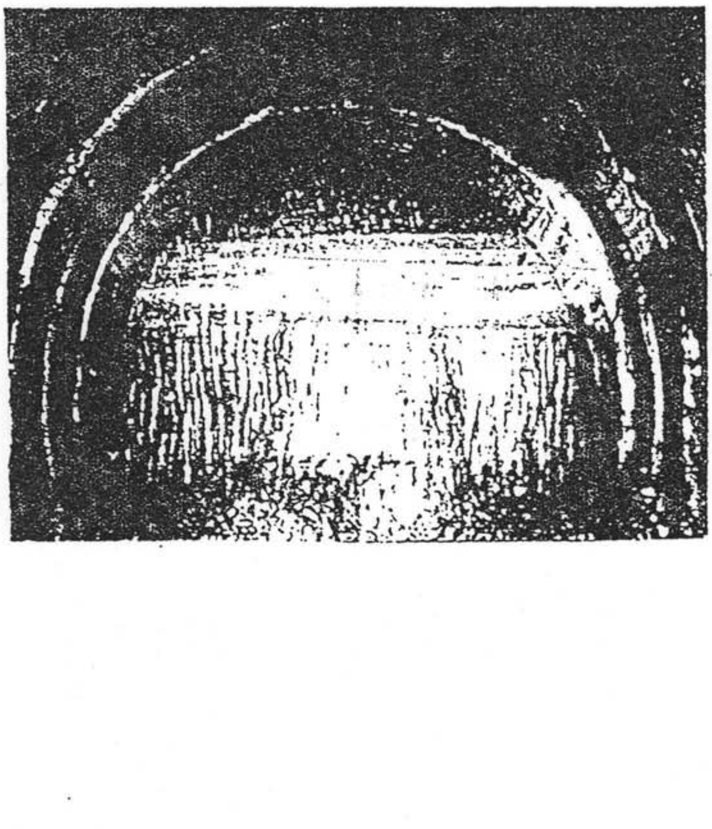
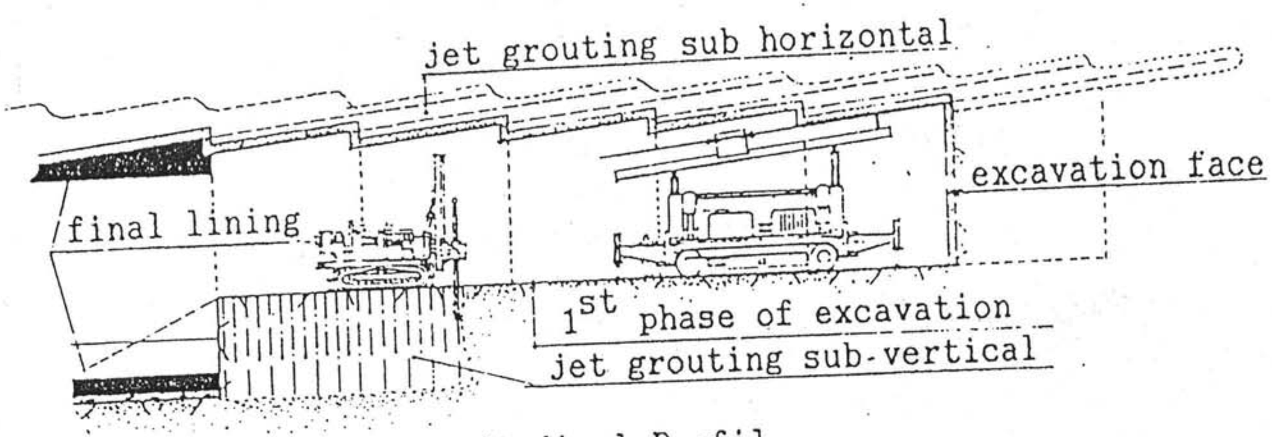
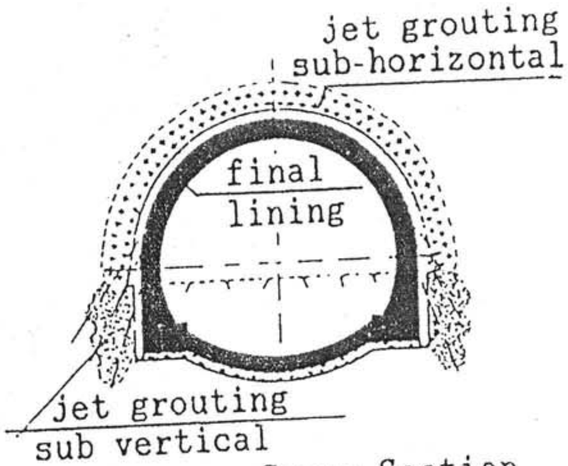


FIG.5



Longitudinal Profil



Cross Section

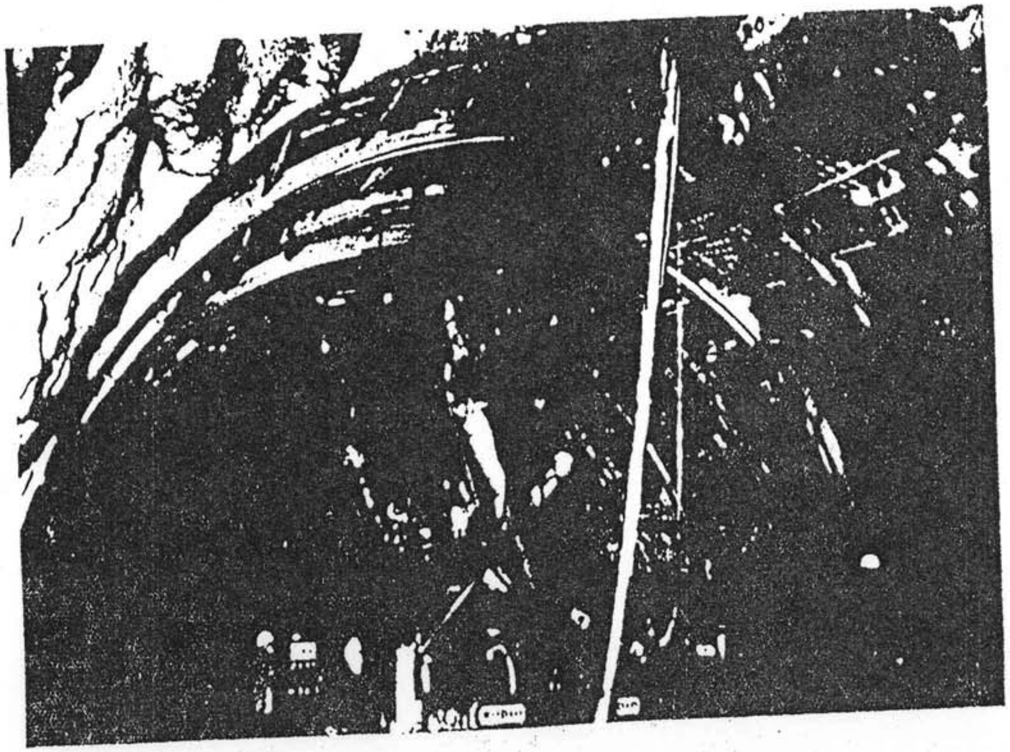


FIG.6

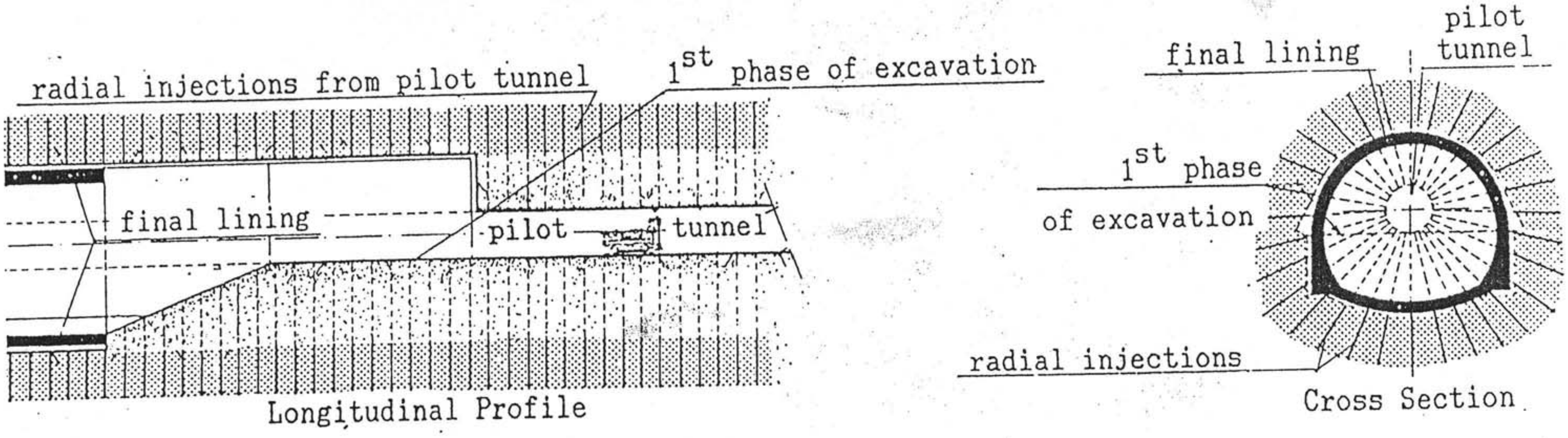
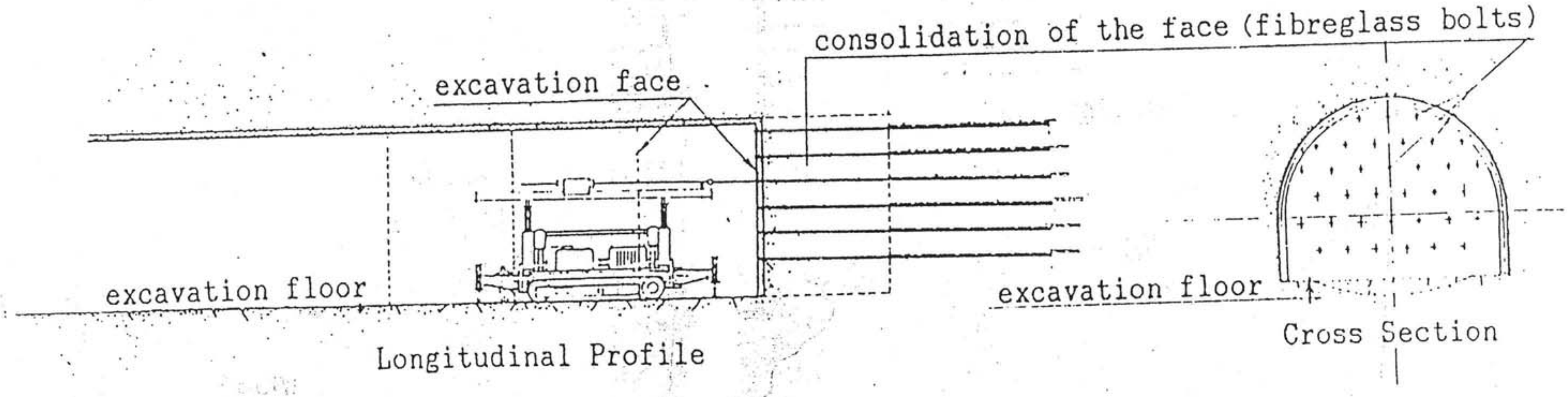

radial injections from pilot tunnel
1st phase of excavation
final lining
pilot
tunnel
Longitudinal Profile
pilot
tunnel
final lining
1st phase
of excavation
radial injections
Cross Section

FIG.7


consolidation of the face (fibreglass bolts)
excavation face
excavation floor
Longitudinal Profile
excavation floor
Cross Section

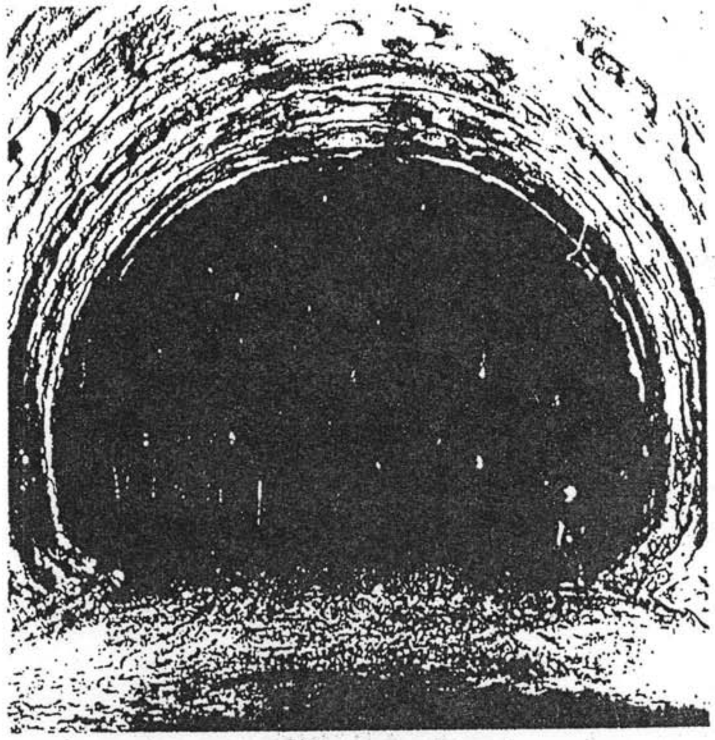


FIG.8

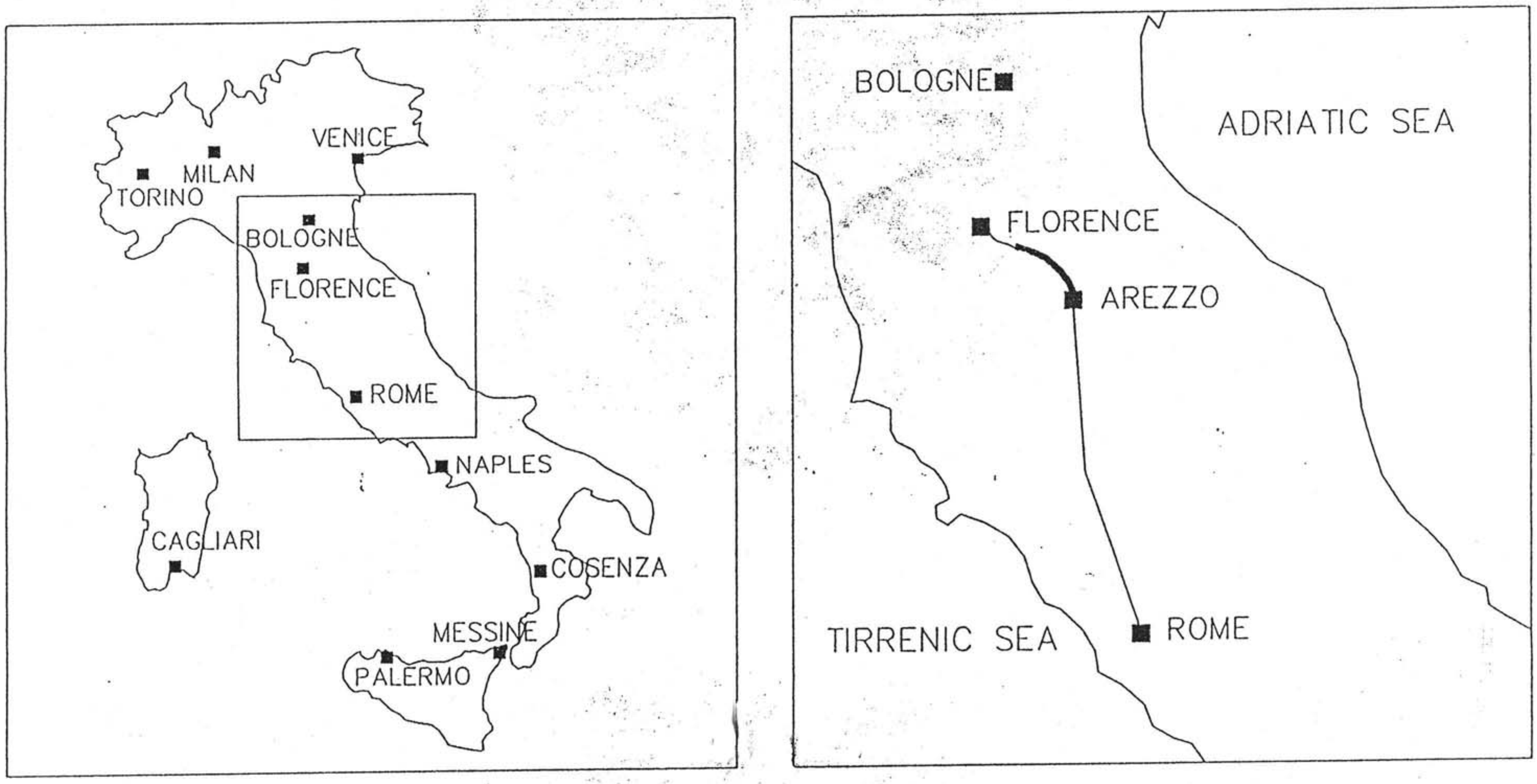

VENICE
MILAN
TORINO
BOLOGNE
FLORENCE
ROME
NAPLES
CAGLIARI
COSENZA
MESSINE
PALERMO
BOLOGNE
ADRIATIC SEA
FLORENCE
AREZZO
TIRRENIC SEA
ROME

FIG.9

HIGH SPEED RAILWAY LINE ROMA–FIRENZE
AREZZO SUD–FIGLINE VALDARNO SECTION

5th LOT

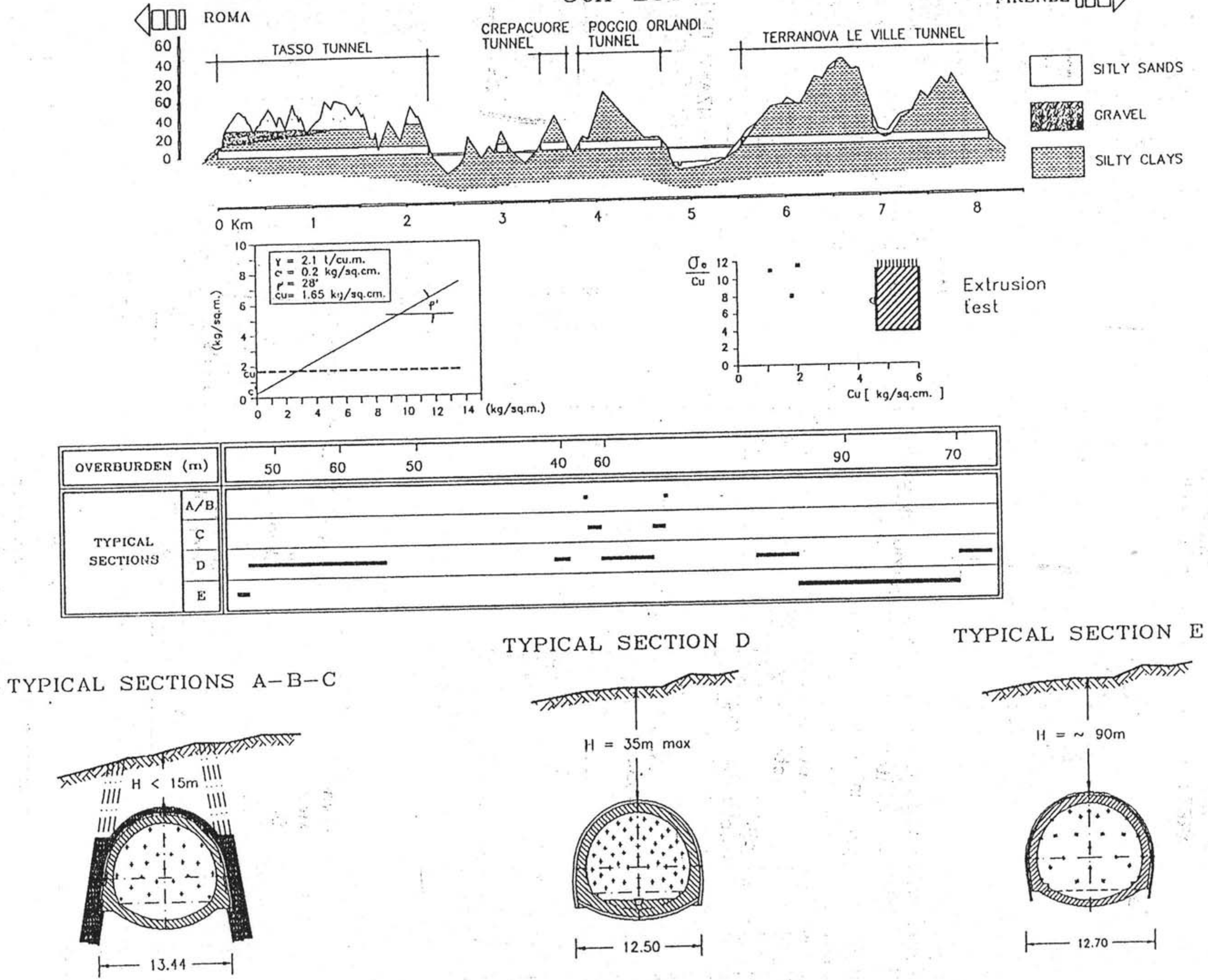
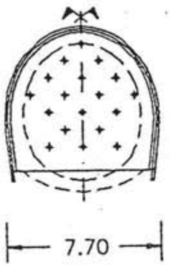


FIG.10

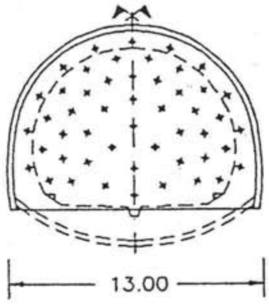
TUNNEL FACE REINFORCEMENT: APPLICATIONS IN ITALY

ROMA–FIRENZE express railway line
AREZZO SUD–FIGLINE VALDARNO stretch
Talleto and Caprenne tunnels



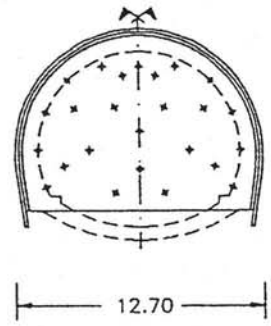
Owner:	F.S.
Main contractor:	Consorzio FE.S.P.I. (Ferdir)
Specialist contractor:	Rodio S.p.A.
Design:	Rocksoil S.p.A.
Max. overburder:	60 m
Excav.section:	60 m2
Soil:	Clayey silts whit sands
Treatment length:	3482 m

ROMA–FIRENZE express railway line
CREPACUORE and POGGIO ORLANDI tunnels



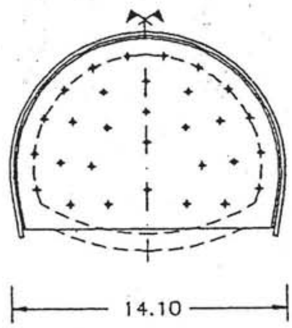
Owner:	F.S.
Main contractor:	Consorzio FE.S.P.I.(Fondedile)
Specialist contractor:	Italsonda S.p.A.
Design:	Rocksoil S.p.A.
Max. overburder:	60 m
Excav.section:	116 m2
Soil:	Clayey silts whit sands
Treatment length:	839 m

ROMA–FIRENZE express railway line
TASSO and TERRANOVA LE VILLE Tunnels



Owner:	F.S.
Main contractor:	Consorzio FE.S.P.I. (Ferdir)
Specialist contractor:	Rodio S.p.A.
Design:	Rocksoil S.p.A.
Max. overburder:	90 m
Excav.section:	116 m2
Soil:	Clayey silts whit sands
Treatment length:	3994 m

CATANZARO ring road link
SAN GIOVANNI Tunnel



Owner:	A.N.A.S.
Main contractor:	Consorzio Sincat
Specialist contractor:	Rodio S.p.A.
Design:	Rocksoil S.p.A.
Max. overburder:	35 m
Excav.section:	140 m2
Soil:	Stiff sand and clay st.
Treatment length:	350 m

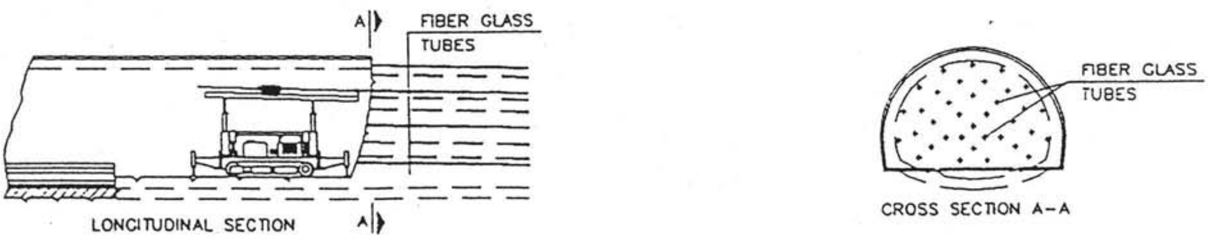
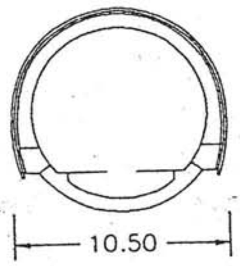


FIG. 11

MECHANICAL PRECUTTING METHOD: APPLICATIONS IN ITALY

SIBARI–COSENZA railway line
Tunnels 1,2,3,4

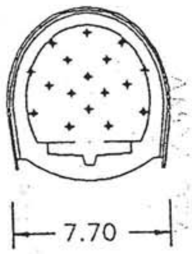
L= 3.50m
O= 0.50m
T= 0.14m



Owner:	F.S.
Main contractor:	ASFALTI SINTEX S.p.A
Specialist contractor:	RODIO S.p.A
Design:	ROCKSOIL S.p.A.
Max. overburder:	110 m
Excav. section:	80 m2
Soil:	clay and silts
Treatment length:	3000 m

ROMA–FIRENZE express railway line
AREZZO SUD–FIGLINE VALDARNO stretch
Talleto e Caprenne tunnels

L= 3.50m
O= 0.50m
T= 0.14m



Owner:	F.S.
Main contractor:	CONSORZIO FE.S.P.I.
Specialist contractor:	RODIO S.p.A
Design:	ROCKSOIL S.p.A
Max. overburder:	60 m
Excav. section:	70 m2
Soil:	clayey silts with sands
Treatment length:	1800 m

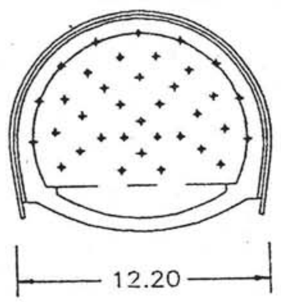
)TARGIA–SIRACUSA railway line SIRACUSA tunnel — b)ROMA–FIRENZE express railway line Tasso and Terranova Le Ville tunnel

)TARGIA–SIRACUSA
railway line
SIRACUSA tunnel

b)ROMA–FIRENZE
express railway line
Tasso and Terranova
Le Ville tunnel

L= 3.00m
O= 0.50m
T= 0.18m

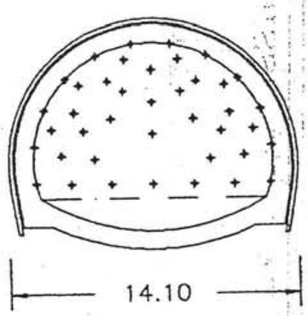
L= 3.50m
O= 0.50m
T= 0.18m



wner:	F.S.	
ain contractor:	a)COLLINI,LODIGIANI ROMAGNOLI	b)CONSORZIO FE.S.P.I.
pecialist contractor:	RODIO S.p.A	
esign:	ROCKSOIL S.p.A	
ax. overburder:	40 m	90 m
xcav. section:	116 m2	109 m2
oil:	ialoclast. and limestone	clayey silts
reatment length:	300 m	1600 m

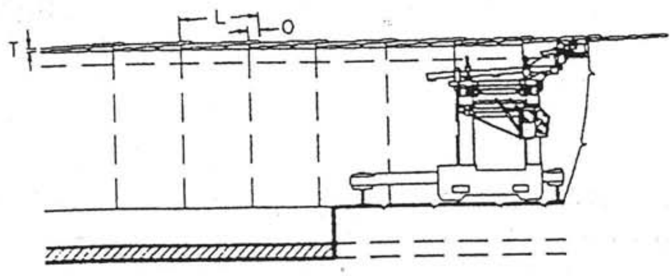
A.N.A.S.
CATANZARO ring road link
San Giovanni tunnel

L= 4.00m
O= 0.50m
T= 0.20m



Owner:	A.N.A.S.
Main contractor:	SINCAT S.c.a.r.l
Specialist contractor:	RODIO S.p.A
Design:	ROCKSOIL S.p.A
Max. overburder:	35 m
Excav. section:	140 m2
Soil:	stiff sand and clay
Treatment length:	350 m

LONGITUDINAL PROFILE



L = shell length

O = overlapping

T = shell thickness

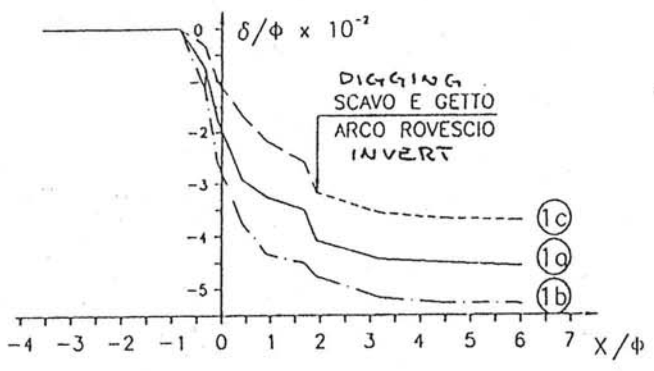
FIG.12

FASE DI VERIFICA

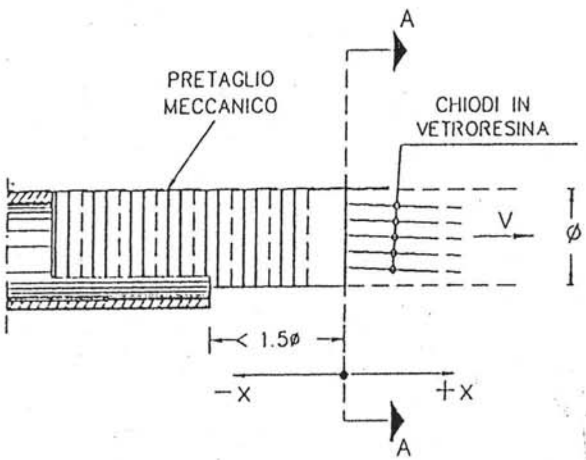
LINEA DIRETTISSIMA ROMA–FIRENZE

TRATTA AREZZO SUD – FIGLINE VALDARNO GALLERIA TASSO – FINESTRA LATO ROMA –

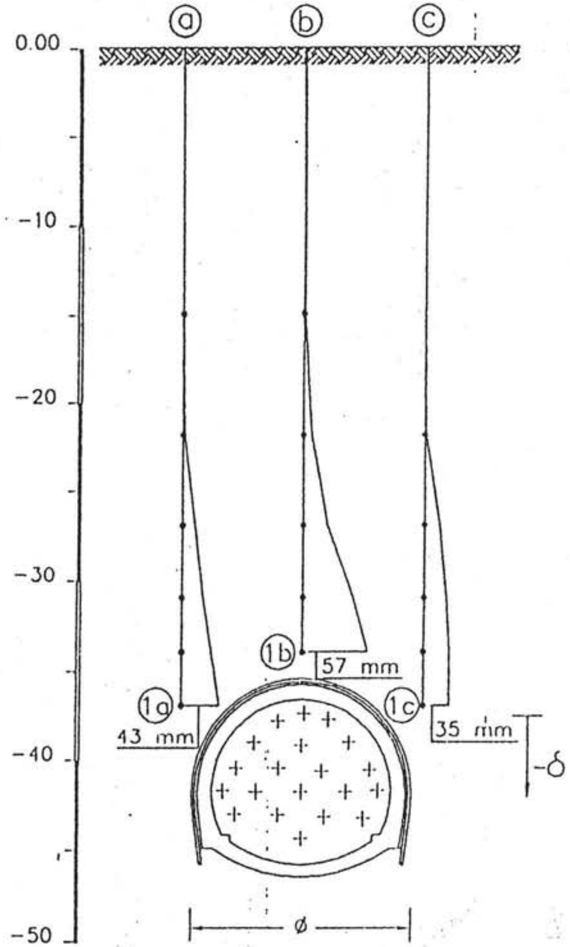
–SEZIONE TIPO "E"–



MISURE DI ABBASSAMENTO



ESTENSIMETRI MULTIBASE



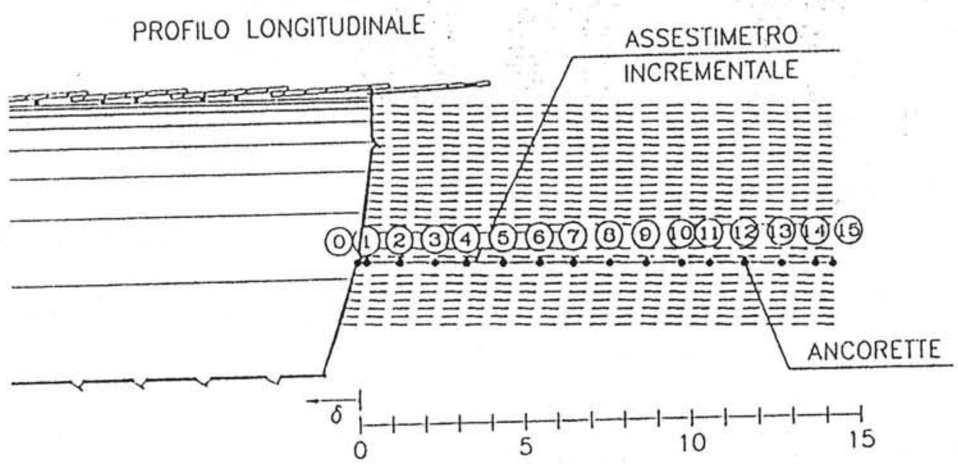
SEZIONE A–A

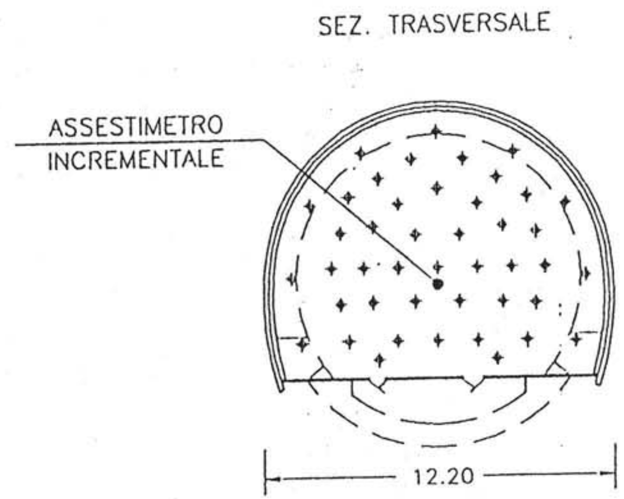
P. LUNARDI – CIFI 90

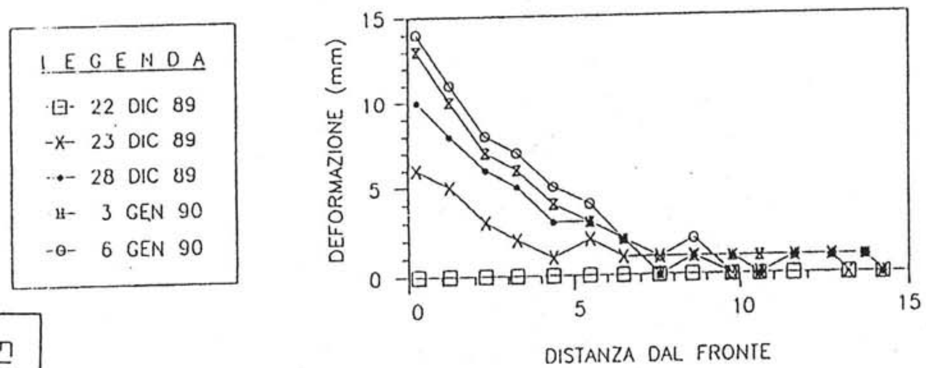
FIG.13

FASE DI VERIFICA

LINEA DIRETTISSIMA ROMA – FIRENZE
TRATTA AREZZO SUD – FIGLINE VALDARNO
MISURE DI DEFORMAZIONE AL FRONTE DI SCAVO







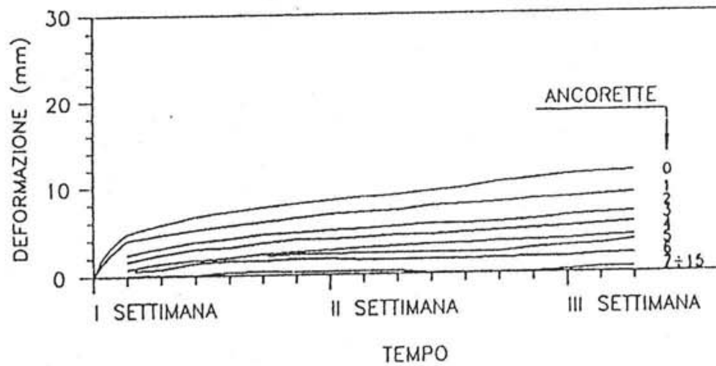
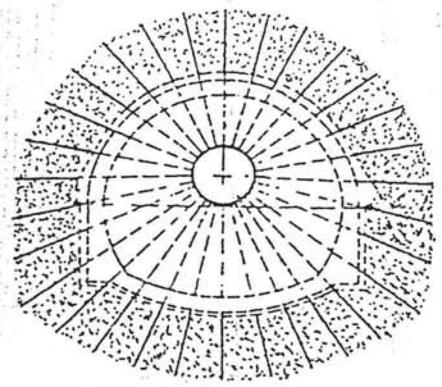


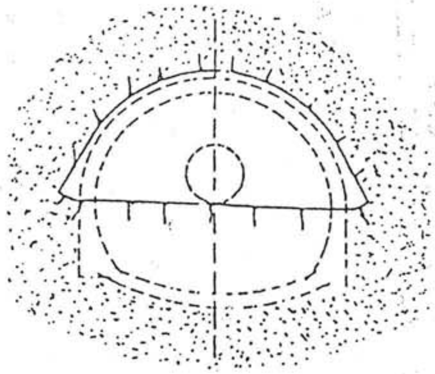
FIG. 14

P. LUNARDI – CIFI 90

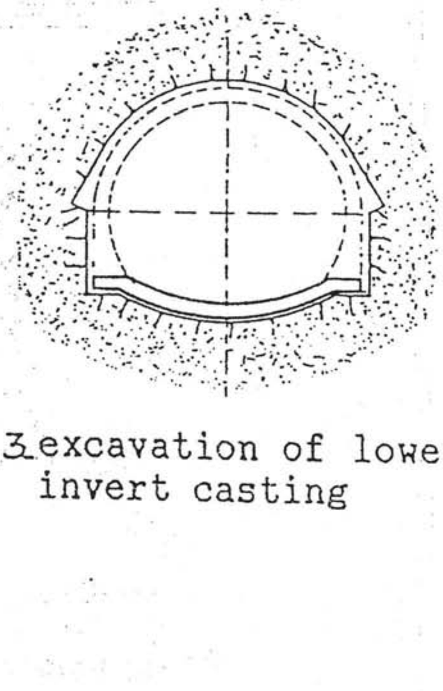
Progressive stages of work on a tunnel
with the use of grout injection



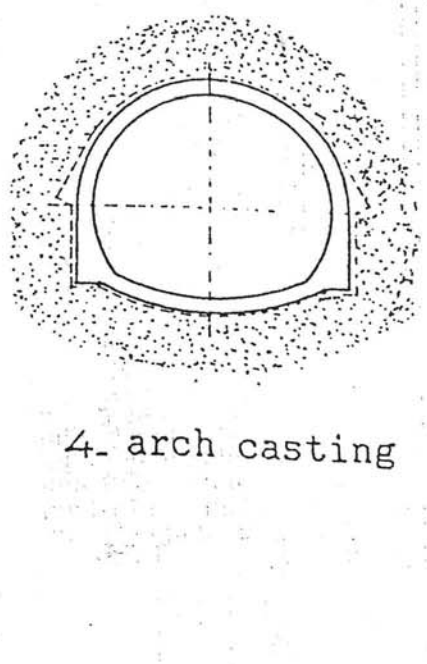
1_ radial injections
from pilot tunnel



2_ excavation
of half section

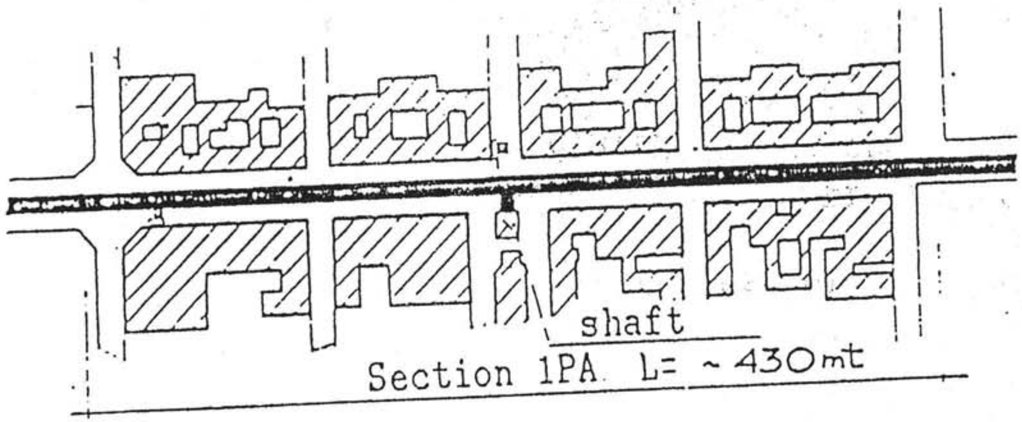


3_ excavation of lower part
invert casting



4_ arch casting

FIG. 15


shaft
Section 1PA L= ~430mt

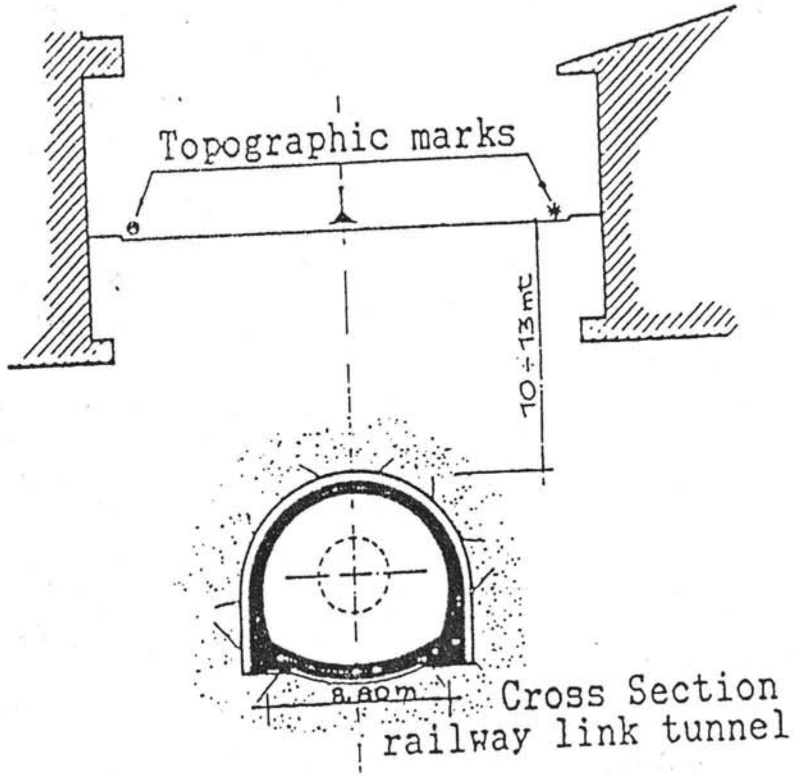

Topographic marks
10 ÷ 13 mt
8.80m
Cross Section
railway link tunnel

FIG.16

Railway Link - Section 1PA

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

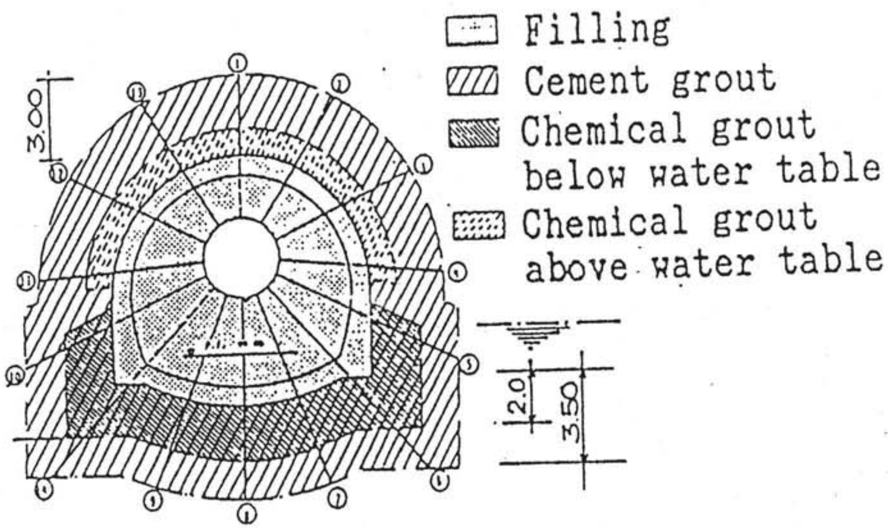


FIG. 16B

Topographic Measure (road marks)

H - Excavation of pilot tunnel with shield
L - Perforation from pilot tunnel
N - Consolidation of two track tunnel

S - Excavation of upper section
T - Completion of two track tunnel
G - Completed tunnel

Edge Mark Total Variation = - 1.20 mm
Center Mark Total Variation = - 0.80 mm
Edge Mark Total Variation = - 0.30 mm

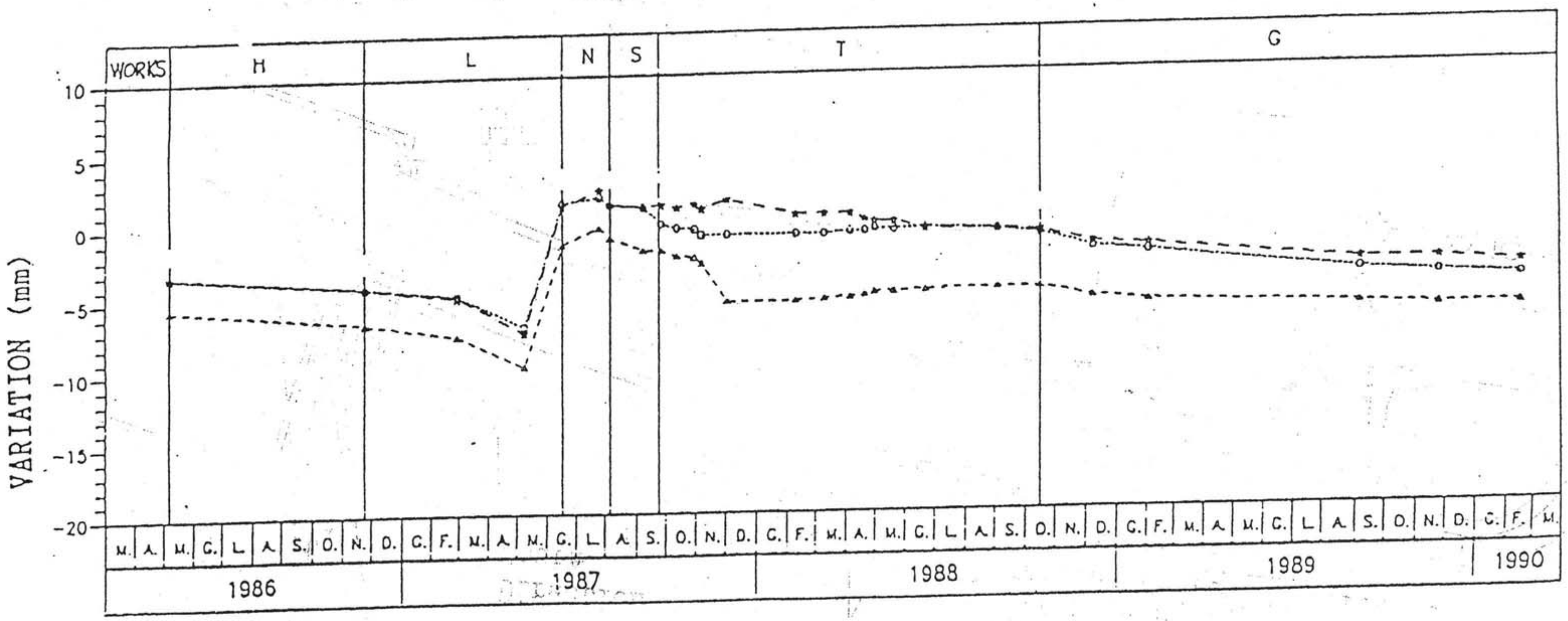
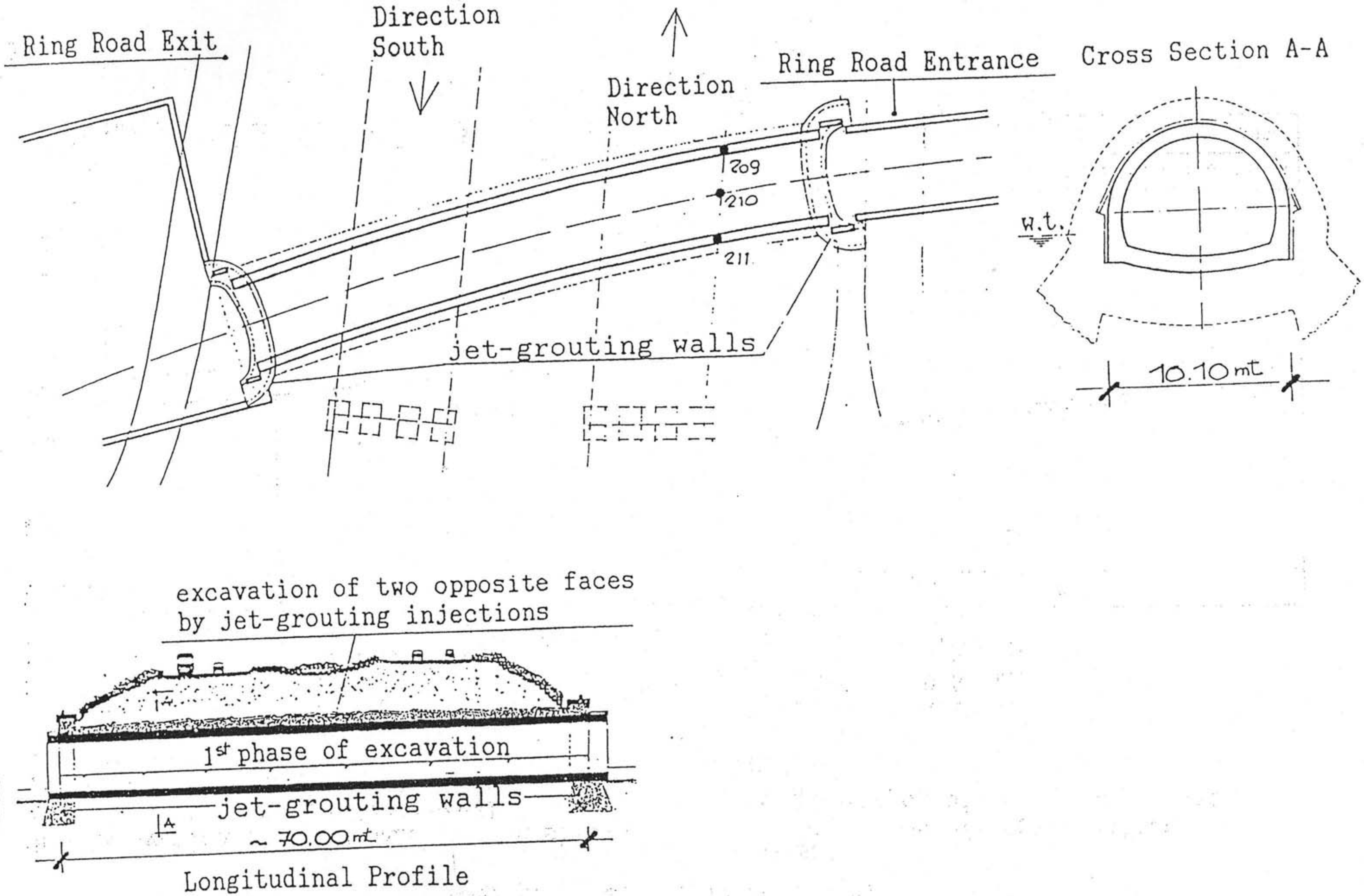
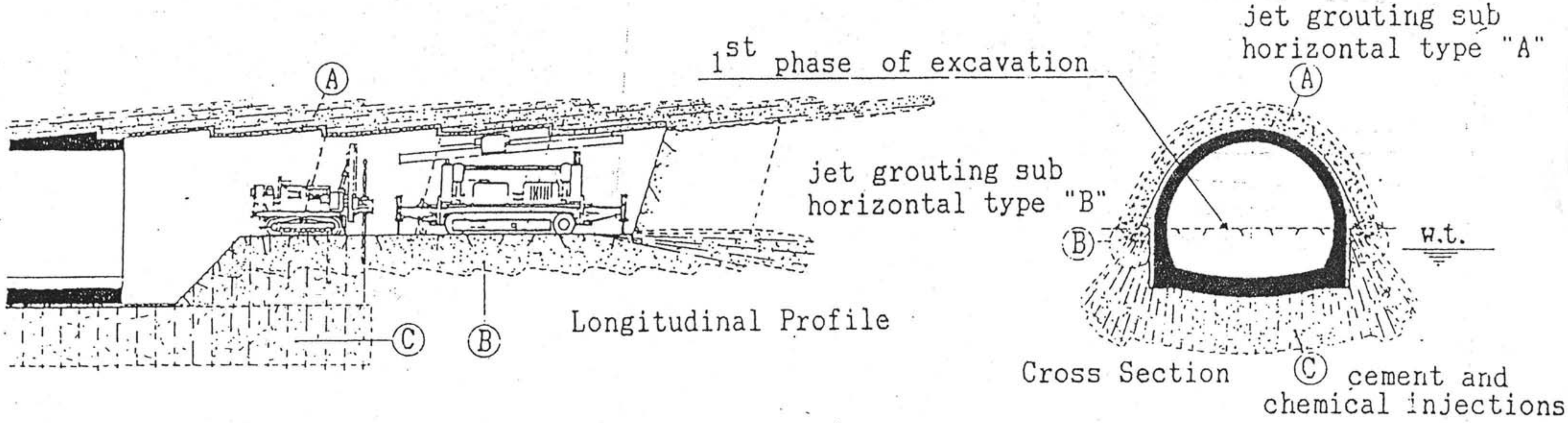


FIG.17



Ring Road Exit
Direction
South
Direction
North
Ring Road Entrance
Cross Section A-A
209
210
211
w.t.
jet-grouting walls
10.10 mt
excavation of two opposite faces
by jet-grouting injections
1st phase of excavation
jet-grouting walls
A
~ 70.00 mt
Longitudinal Profile

FIG. 18



1st phase of excavation
jet grouting sub horizontal type "A"
jet grouting sub horizontal type "B"
A
B
C
w.t.
Longitudinal Profile
Cross Section
C cement and chemical injections

FIG. 18B

TOPOGRAPHIC MEASURE 209 ÷ 211 (see fig. 10)

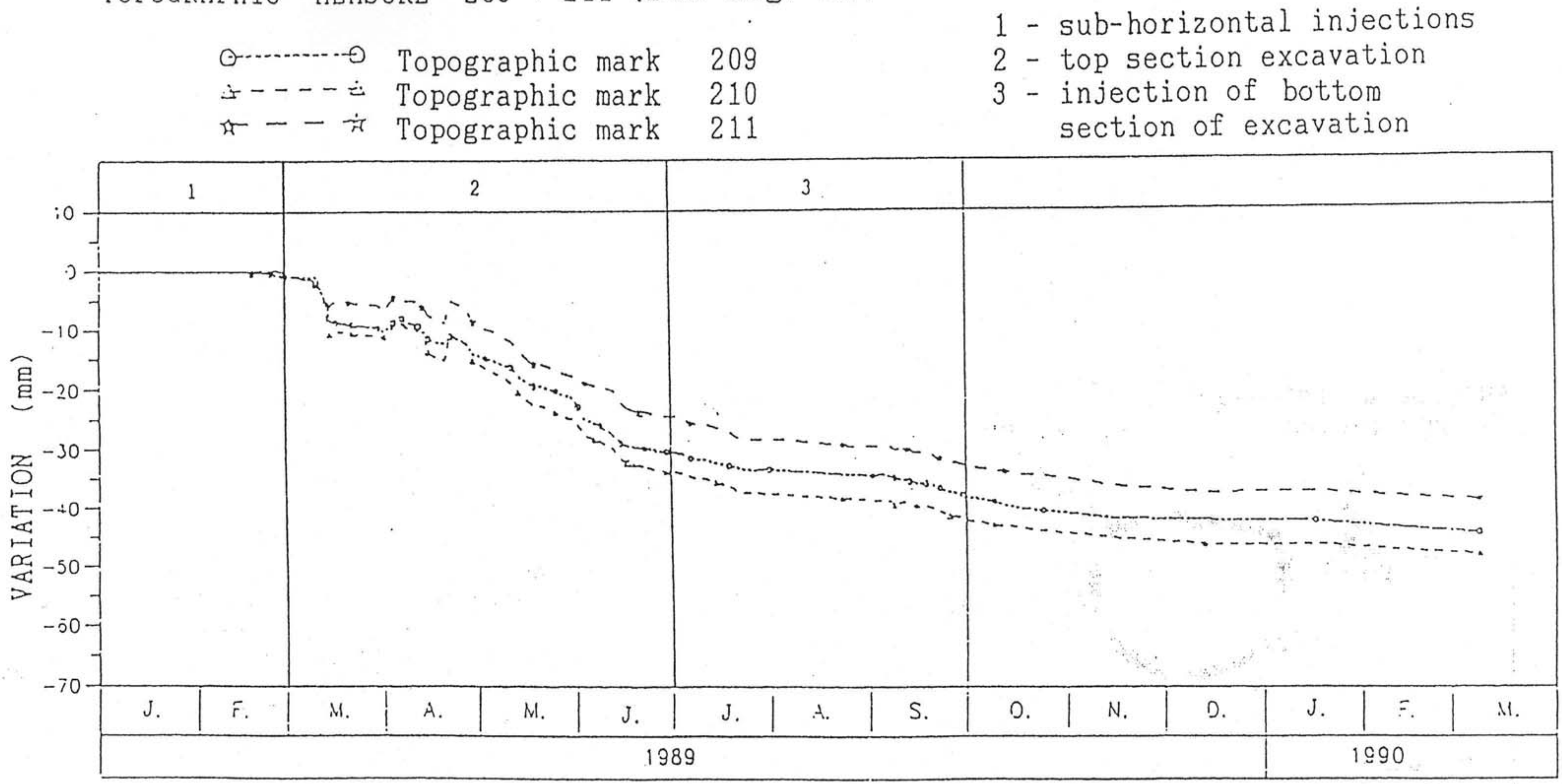


FIG.19

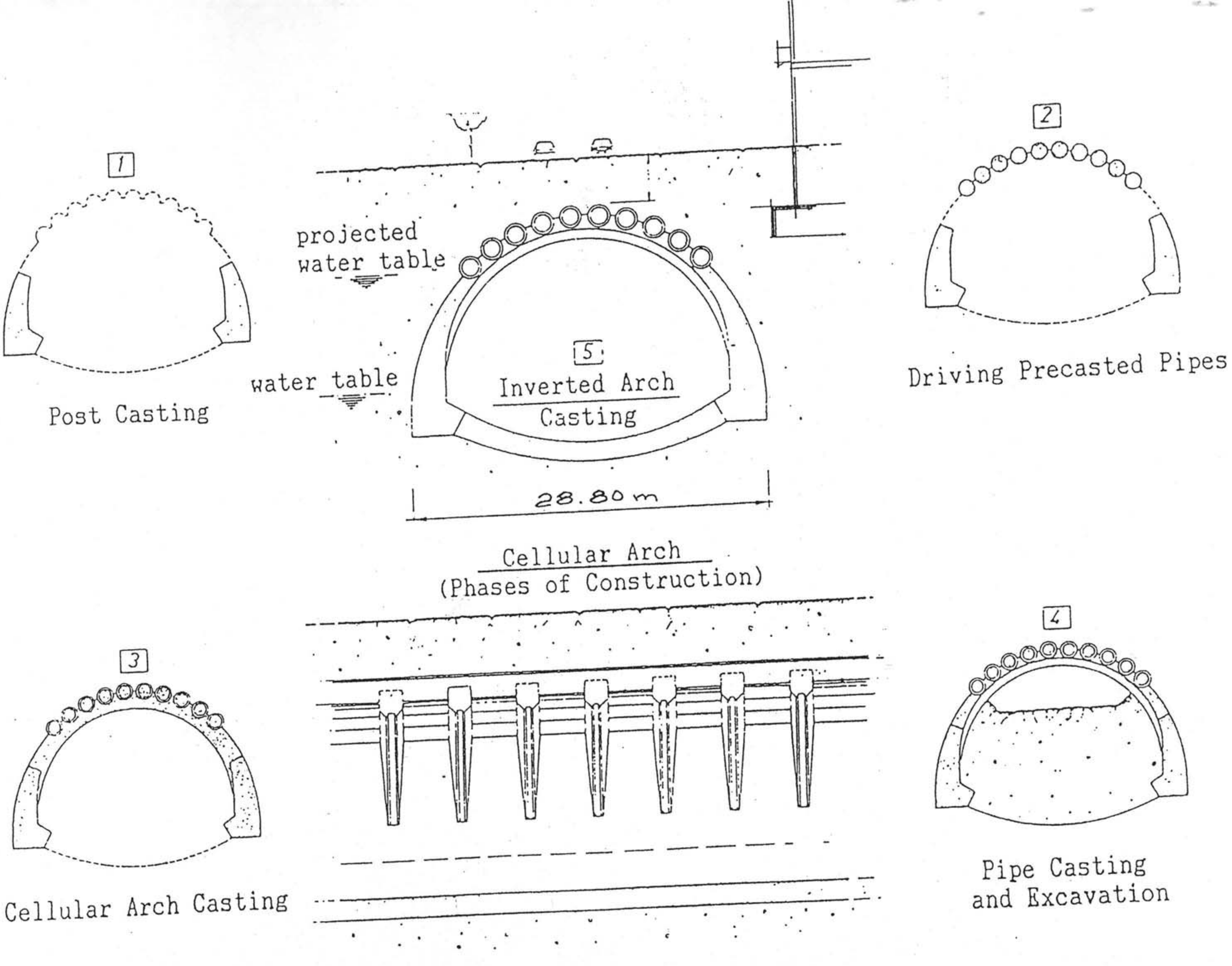
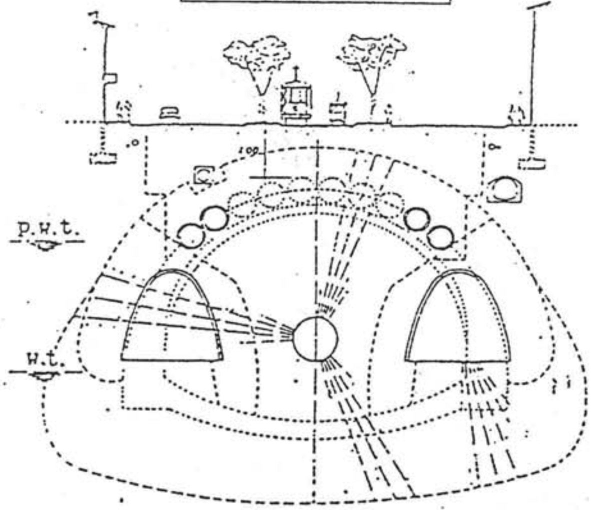

1
Post Casting
projected
water table
water table
5
Inverted Arch
Casting
28.80 m
2
Driving Precasted Pipes
Cellular Arch
(Phases of Construction)
3
Cellular Arch Casting
4
Pipe Casting
and Excavation

FIG.20

STAGE 1

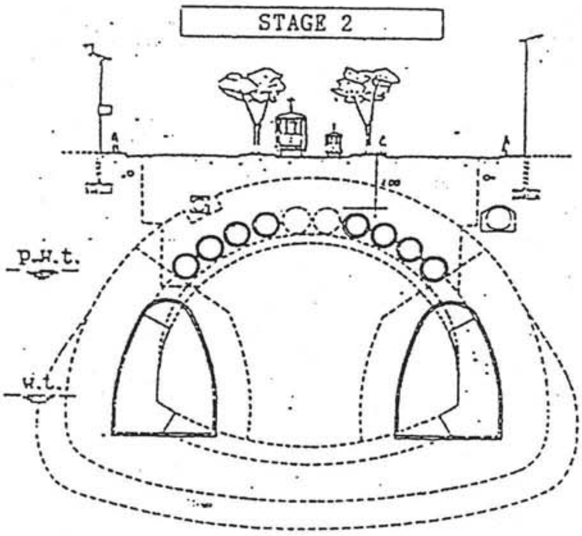


• Grouting pipe and side drift zone.

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

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

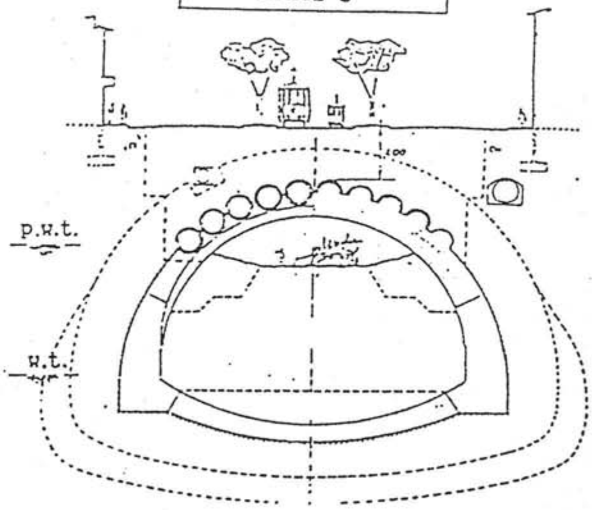
STAGE 2



• 2^{nd} stage excavation of side drift.

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

STAGE 3



• Excavation and casting arches and tubes

• Crown excavation in various phases.

• Invert excavation reinforcement and concrete casting of invert.

FIG.21

Driving Pipe Shaft

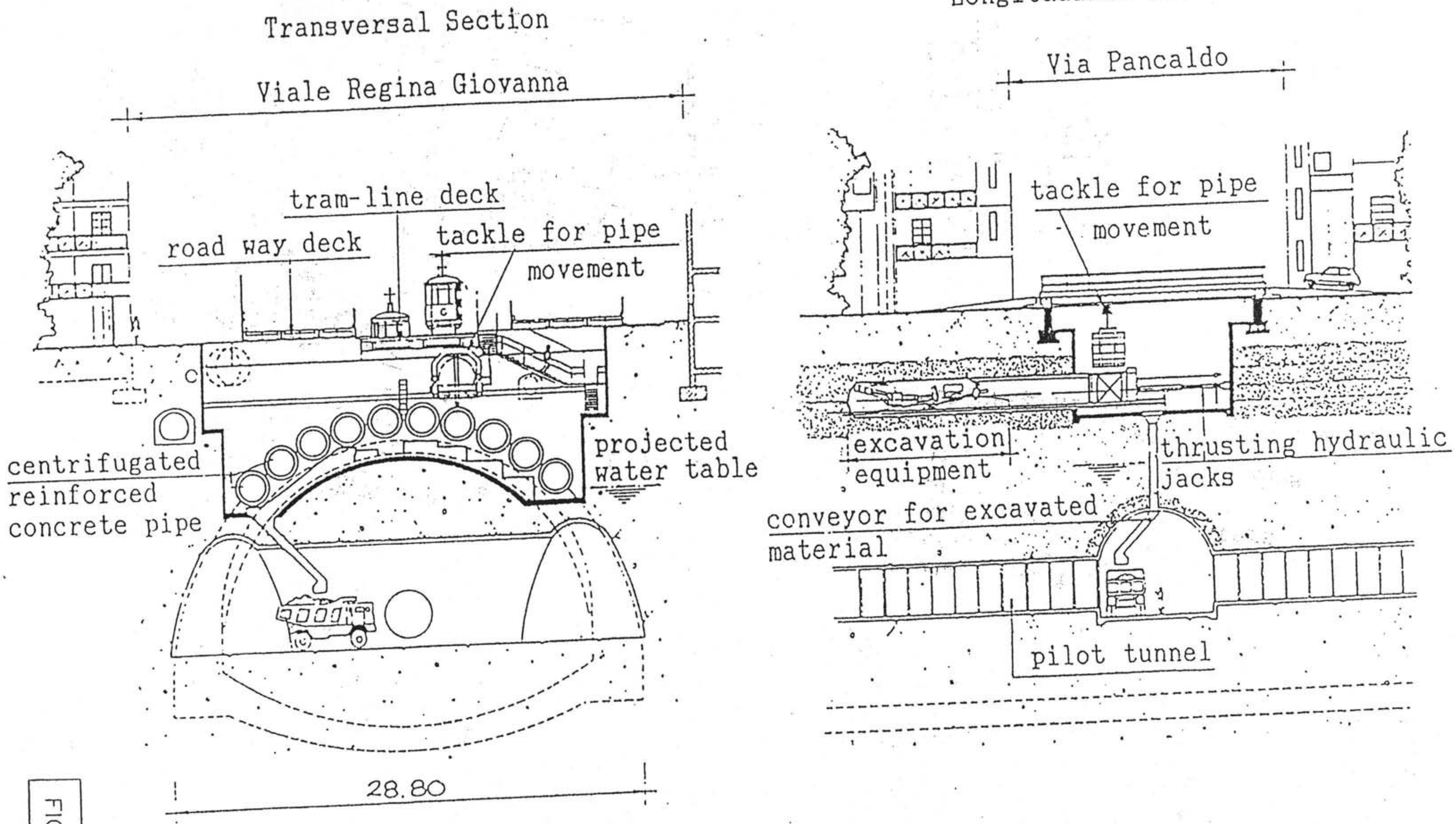
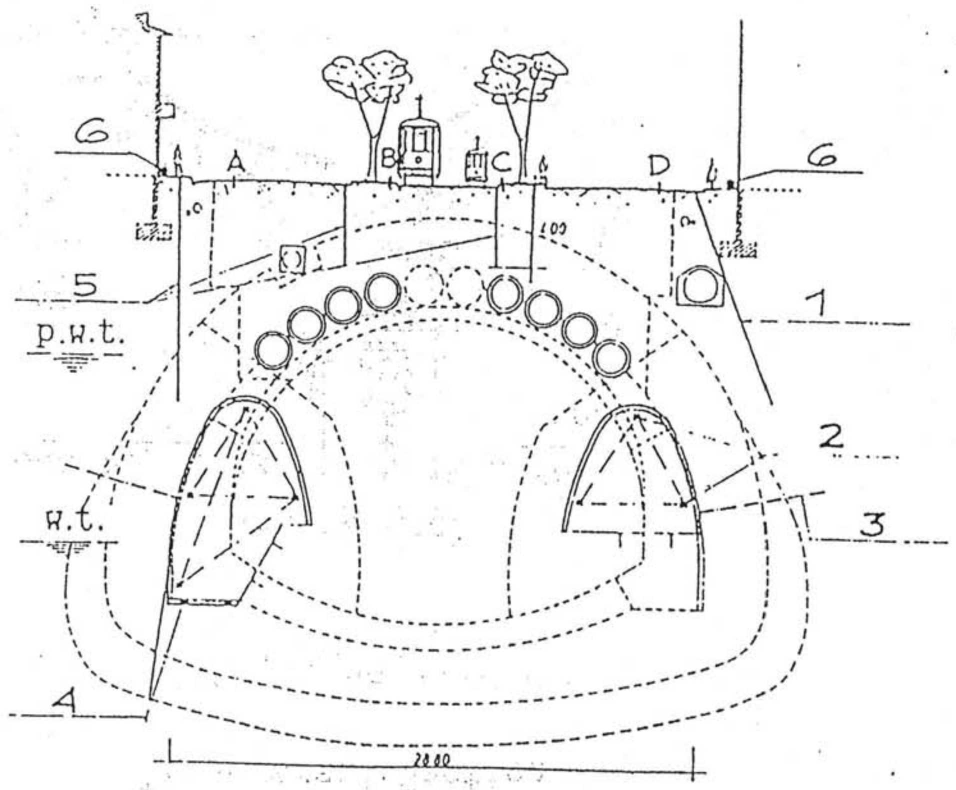
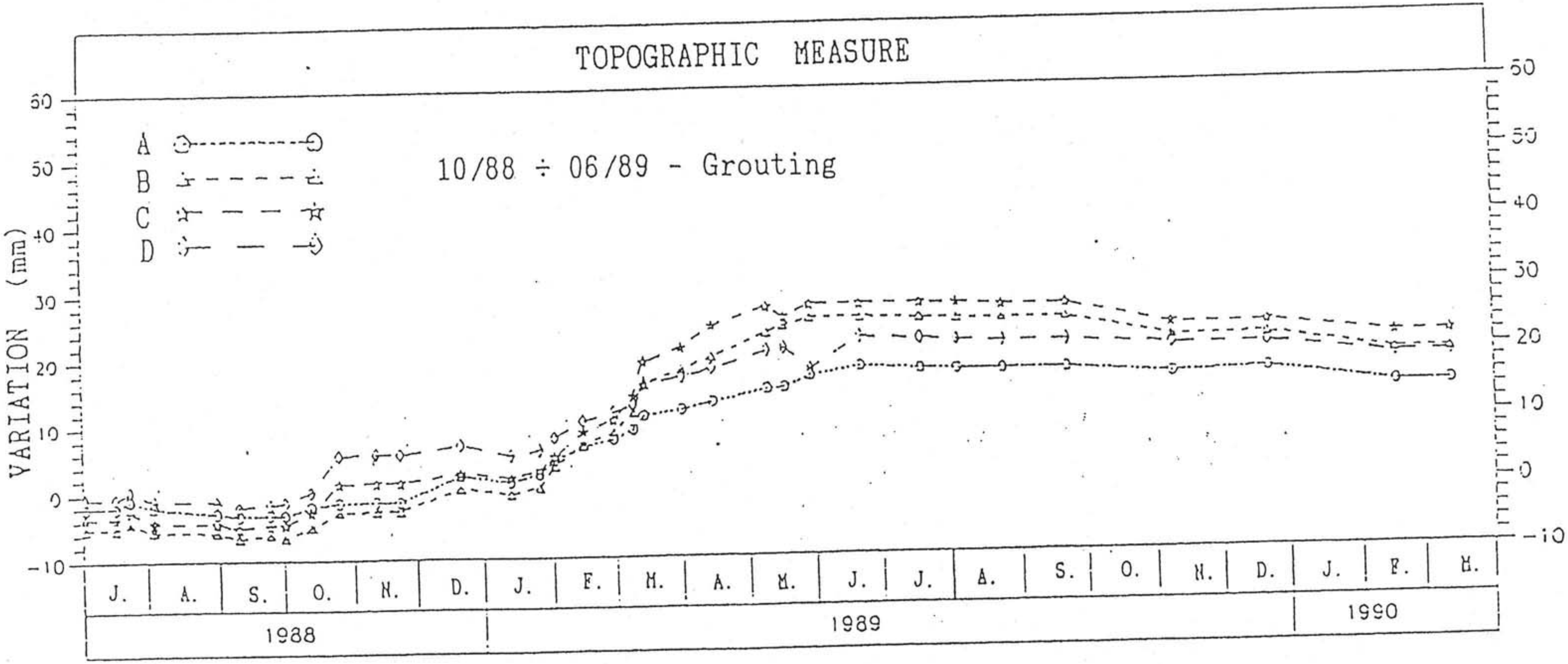


FIG.22



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

FIG.23



TOPOGRAPHIC MEASURE
10/88 ÷ 06/89 - Grouting
A
B
C
D
VARIATION (mm)
50
40
30
20
10
0
-10
J.
A.
S.
O.
N.
D.
J.
F.
M.
A.
M.
J.
J.
A.
S.
O.
N.
D.
J.
F.
M.
1988
1989
1990

FIG.24