Appendices

Introduction to the appendices

It is said that a good example is worth more than any long theoretical discourse. I have therefore brought together some of the most important examples of tunnelling performed in recent years in these appendices. They are in seven parts.
The first three parts (A, B and C) concern three particularly significant projects:

A. the design and construction of tunnels for the new Rome-Naples high speed/capacity railway line. The application of the ADECO-RS approach is illustrated at both the design and the construction stages in terms of the reliability of the design;
B. the design and construction of approximately 73 km of running tunnels for the new Bologna-Florence section of the new high speed/capacity Milan-Rome-Naples railway line, where the considerable length of the alignment that runs underground, the heterogeneity of the ground tunnelled and the extremely difficult stress-strain conditions to be overcome constituted a severe test of the effective capacity of the new ADECO-RS approach to meet expectations in terms of the industrialisation of tunnelling;
C. the construction of the "Tartaiguille" tunnel in France. It proved too difficult to drive this tunnel using conventional means, while the application of the new advance method in the presence of a rigid core enabled it to be completed on time and within the budgeted cost.

The next four parts of the appendices (D, E, F and G) are dedicated to the most successful of the tunnelling technologies designed by the author over the last twenty years:

D. the cellular arch with which it is possible to construct large underground cavities (up to a span of 60 m) using bored tunnel methods without causing surface settlement even in ground of poor geomechanical quality (reference project: Venezia Station on the Milan urban railway link line);
E. artificial ground overburdens (AGO), to drive underground tunnels even where the necessary overburden is lacking, with considerable savings on times and costs which would be required to construct the tunnel itself artificially (reference project: the new Rome-Naples high speed/capacity railway line, tunnels Piccilli 1, Piccilli 2, Castagne, Santuario, Caianello and Briccelle);
F. shells of improved ground by means of vertical jet-grouting and the same technique applied horizontally for the construction of tunnel portals in difficult grounds (reference project: portal on the Messina side of the S. Elia tunnel of the Messina-Palermo highway);
G. the widening of road, highway and railway tunnels without interrupting traffic, which solves the problem of modernising old tunnels on major transport routes (reference project: widening of the Nazzano tunnel on the Milan-Rome motorway from two to four lanes in both directions).

I believe that reading these seven examples should be very helpful both for acquiring an in-depth understanding of how the ADECO-RS approach is applied in different stress-strain contexts and for fully appreciating the progress that has been made in tunnelling in the last twenty years.

Rome-Naples high speed/capacity railway line, Piccilli 2 tunnel. Construction of the "artificial ground overburden (AGO)" in the section with no overburden, ground: pyroclastites

The design and construction of tunnels for the new Rome-Naples high speed/capacity railway line

The new Rome-Naples railway is part of the High Speed Train Milan-Naples line which in turn is a southern terminal of the European High Speed network.

The line has been divided into lots with differing costs. Construction contracts were awarded to five contractors belonging to the IRICAV UNO consortium (the general contractor for this part of the line), namely Pegaso, Icla, Italstrade, Vianini and Condotte.

The total length of the line to be constructed was 204 km and 28.3 km of it (13% of the total length) runs through bored tunnel.

The problems encountered in the design and construction of the parts of the tunnel designed by the author are discussed below. They amounted to 77% of the underground works for a total of 21.8 km divided into 22 tunnels.

Colle Pece Tunnel. North portal

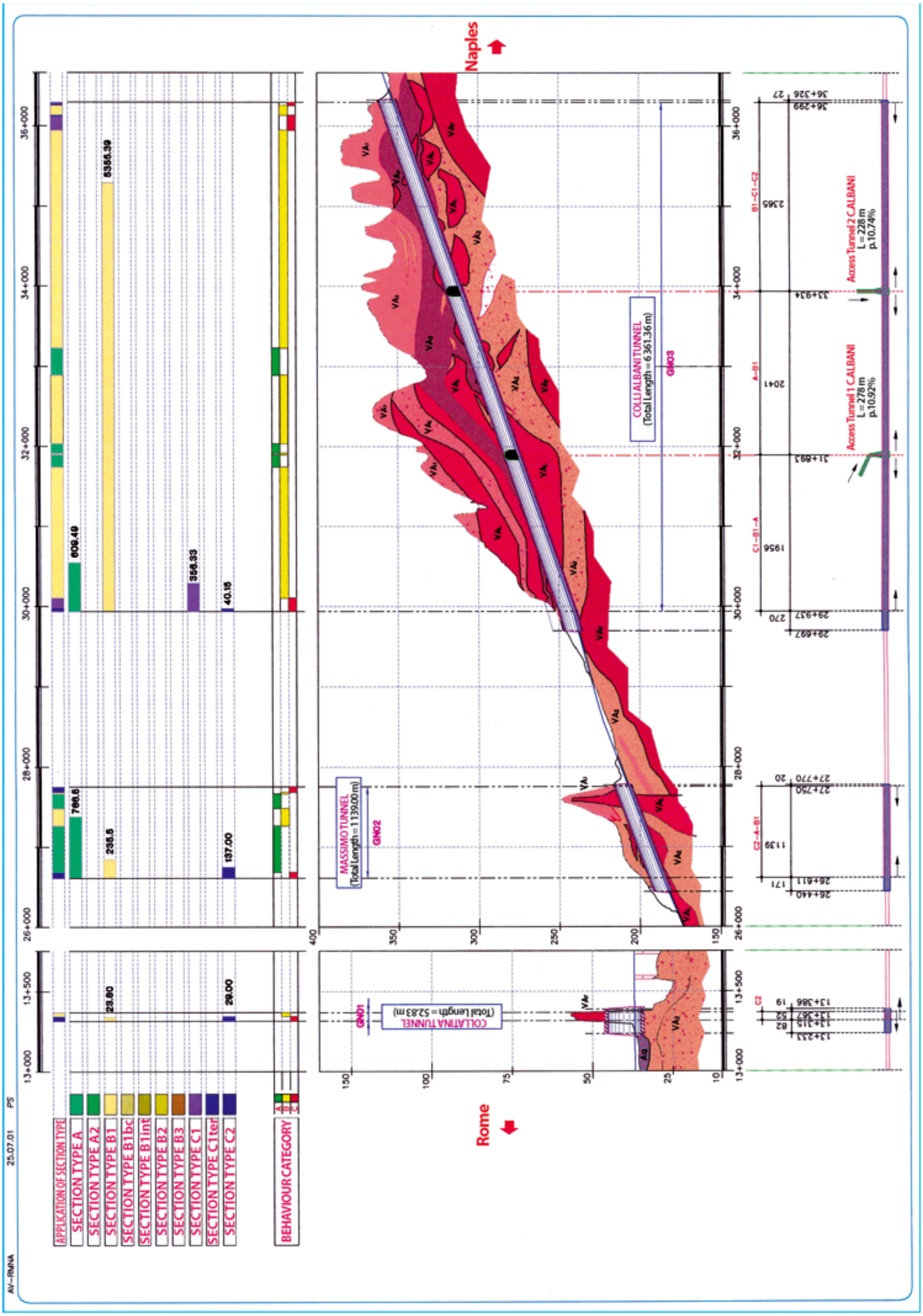


Fig. A.1 *Longitudinal profiles of the tunnels: Collatina, Massimo, Colli Albani*

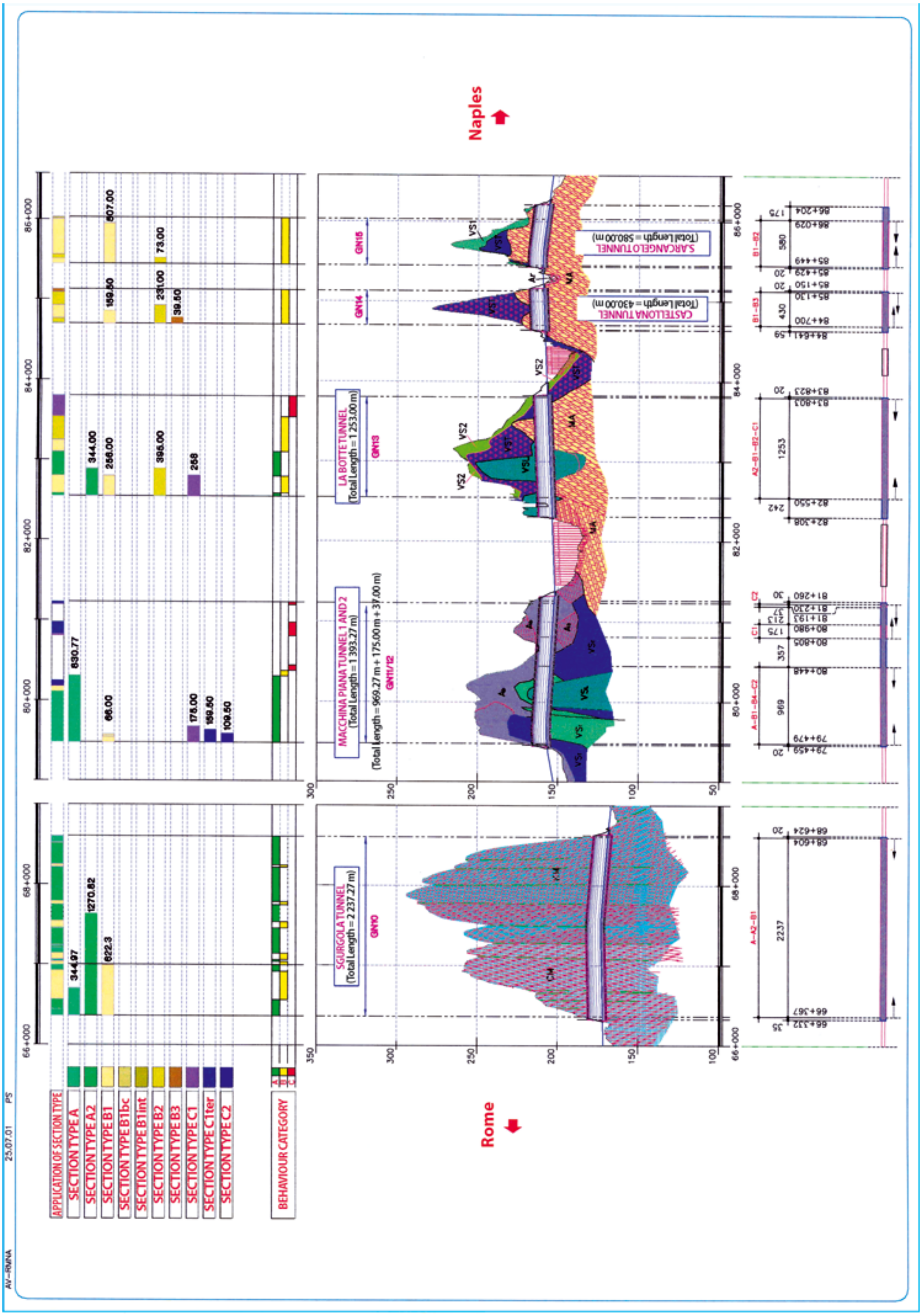


Fig. A.2 *Longitudinal profiles of the tunnels: Sgurgola, Macchia Piana 1 and 2, La Botte, Castellana, S. Arcangelo*

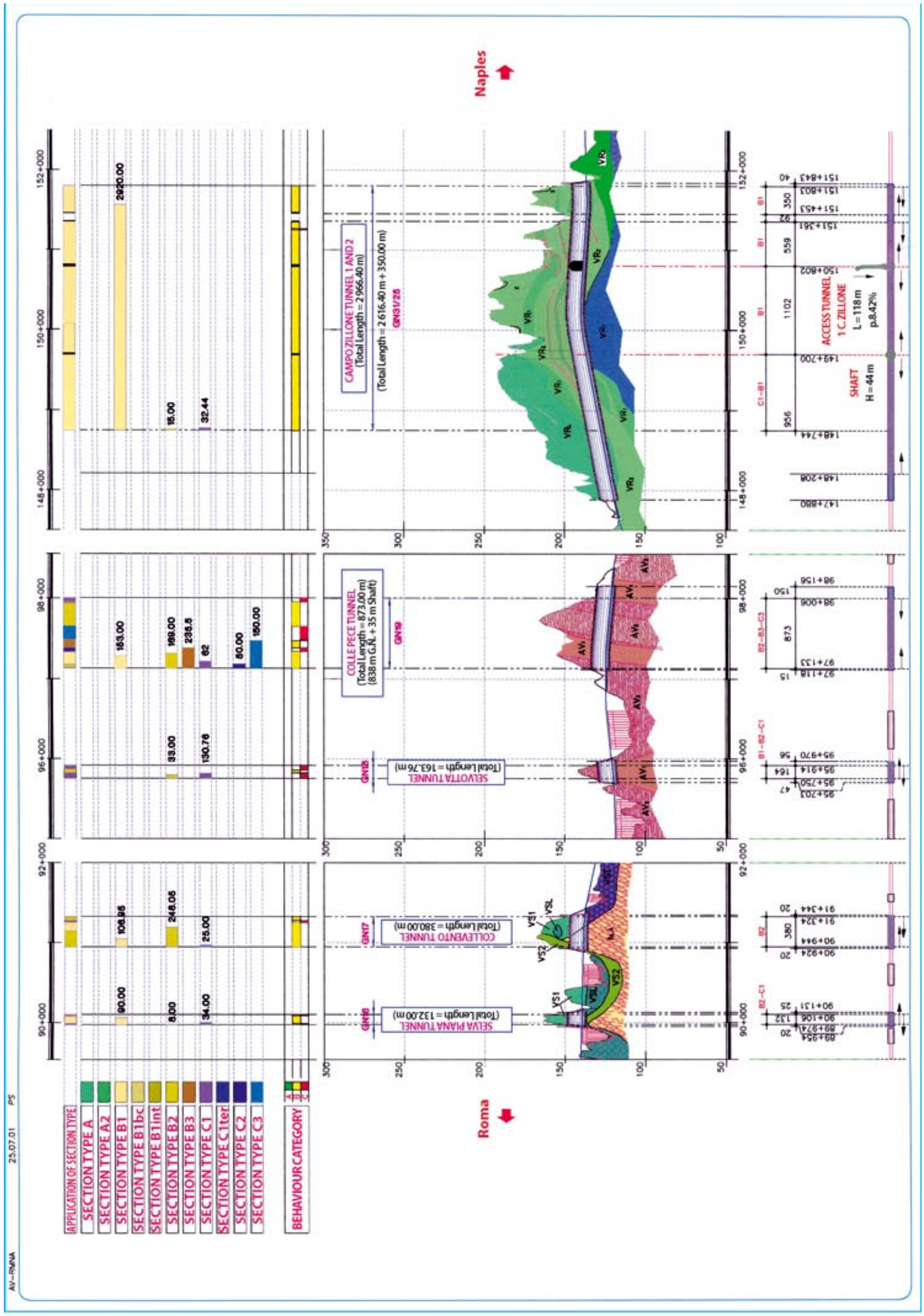


Fig. A.3 *Longitudinal profiles of the tunnels: Selva Piana, Collevento, Selvotta, Colle Pece, Campo Zillone 1 and 2*

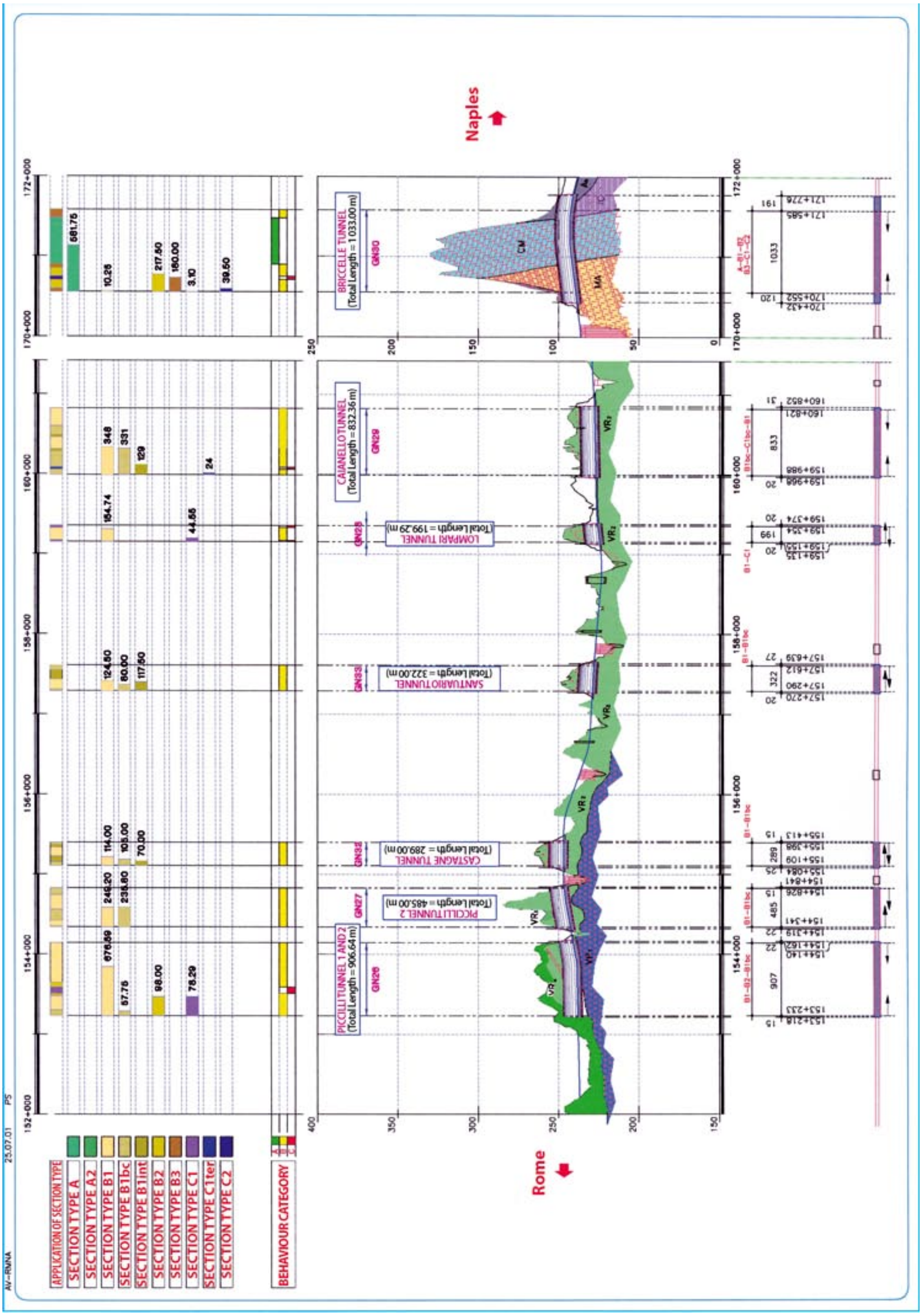


Fig. A.4 *Longitudinal profiles of the tunnels: Piccilli 1 and 2, Castagne, Santuario, Lompari, Caianello, Briccelle*

Colle Pece Tunnel. Face reinforced with fibre glass structural elements. Ground: scaly clays, overburden: ~ 25 m

Colle Pece Tunnel. Positioning a steel rib. Ground: scaly clays, overburden: 25 m

Figures A.1 to A.4 show the longitudinal geological profiles of the 22 tunnels: the *Colli Albani* tunnel (Pegaso lot, 6361 m) is the longest tunnel on the entire line, while the *Collatina* tunnel (Italstrade lot, 53 m) is the shortest.

The underground alignment runs through ground which can basically be classified as having two different types of origin:

- pyroclastic ground and lava flows, generated by eruptions of the volcanic complexes of *Latium*, *Valle del Sacco* and *Campania*;
- sedimentary rocks of the flyschoid and carbonatic type (marly and limy argillites) belonging to the Apennine system.

The overburdens vary greatly but never exceed 110 m, while they are often very shallow at the portals.

Geological and geotechnical background (survey phase)

As mentioned above, many of the tunnels pass through ground of volcanic origin. The activity of the volcanic bodies concerned began in the Pliocene period and developed mainly in the Pleistocene period from upper *Latium* down to the *Vesuvian* region dying out about 100,000 years ago. The volcanological development of the active centres passed through three different phases, which occurred in almost the same order in all the centres of activity: the phase of the volcano-stratum, the phase of great ignimbritic expansion and the phase of the construction of ash and lava cones.

Two important centres of eruption are identifiable along the alignment, one in the *Latium* area (*Colli Albani*) and the other (*Roccamonfina*) in the *Campania* area near the end of the route. There is a smaller volcanic body near the *Sacco* river (*Valle del Sacco*). Between the two major volcanic "domains", in the central part of the alignment, often interdigitating with the volcanic products of the *Valle del Sacco* volcanic body, there are outcrops of sedimentary rocks of the Apennine backbone of the Cretaceous and Miocene periods, in carbonatic, flyschoid and clayey-marly facies.

From a hydrogeological viewpoint, the route lies practically entirely above the regional water-table and consequently the tunnels are not subject to large heads of pressure; there are, however, some localised exceptions, for example in the *La Botte* tunnel of the Italstrade lot, and the *Castellona* tunnel of the Vianini lot, where the marly-arenaceous complex provides an impermeable bedrock to the overlying pyroclastites, favouring the formation of suspended water-tables with modest heads of pressure or at the northern portal of the *Colli Albani* tunnel where a water-table that supplies a fountain in the Vetrice area is intersected.

Massimo Tunnel. Passage under the crater of a volcano, overburden: ~ 15 m

Colli Albani Tunnel. A typical face running through vulcanites, overburden: ~ 55 m

	VOLCANIC COMPLEXES (Vulcaniti dei Colli Albani, Vulcaniti della Valle del Sacco, Vulcaniti di Rocca Monfina)			**CARBONATIC** COMPLEX (Calcari dei Monti Lepini)	**FLYSCHOID COMPLEX** (Argille Varicolori, Complesso Marnoso-Arenaceo)	
	Pyroclastites	**Tuffs**	**Lavas**	**Stratified limestones**	**Scaly clays**	**Clayey marls with arenaceous layers**
γ [t/m³]	1.4 – 1.6	1.6 – 2	2.6 – 2.7	2.5 – 2.7	2.0 – 2.1	2.2 – 2.4
C [Mpa]	0 – 0.1	0.1 – 0.5	0.5 – 5	0.5 – 1	0.01 – 0.05	0.2 – 0.4
φ [°]	25 – 35	20 – 25	30 – 35	35 – 40	18 – 23	28 – 33
σ_{gd} [Mpa]	1 – 5	–	–	1 – 4	–	–
E [Mpa]	1 – 5	300 – 600	2,000 – 5,000	10,000 – 12,000	50 – 100	200 – 400
υ	0.35	0.3	0.25	0.25 – 0.3	0.35	0.3

From a geotechnical viewpoint the lithotypes given in table below were identified in the geological complexes. The parameters for them with the variation in the geotechnical characteristics are given in the same table.

Stress-strain behaviour predictions (diagnosis phase)

It became immediately clear in the diagnosis phase that the tunnels to be driven, either because of the geotechnical characteristics of the ground or because of the varying overburdens, would be subject to extremely different stress-strain conditions.

The geological, geotechnical and hydrogeological information acquired and the results of calculations performed using analytical and/or numerical methods were therefore employed to divide the underground alignment into sections of uniform stress-strain behaviour as a function of core-face stability in the absence of intervention to stabilise the tunnel:

- stable core-face (behaviour category A);
- stable core-face in the short term (behaviour category B);
- core-face unstable (behaviour category C).

Category A comprised all those sections of tunnel in which calculations predicted that:

Colli Albani Tunnel. Detail of the ribs in the cross vault of the junction with the Finestra 1, access tunnel, ground vulcanites, overburden: ~ 50 m

- the stress state of the ground at the face and around the cavity would not have exceeded the natural strength of the medium;
- an "arch effect" would have been created close to the profile of the tunnel;
- deformation phenomena would have developed in the elastic range, having an immediate effect and a magnitude of a few millimetres;
- as a consequence, the face as a whole would have remained stable.

This category was found in lava, lithoid tuff and slightly fractured limestone sections, materials which generally present good strength properties in relation to the stresses mobilised by driving tunnels with the design overburdens.

Category B, however, included all those sections of tunnel where mathematical calculation predicted that:

- the stress state at the face and around the cavity during tunnel advance would have exceeded the natural strength characteristics of the medium, in the elastic range;
- an "arch effect" would not have been formed close to the profile of the tunnel, but at a distance equal to the size of the band of plasticised ground around the cavity;
- deformation phenomena would have developed into the elasto-plastic range with the effect deferred in time and measurable in centimetres;
- as a consequence, the core-face would have remained stable in the short term at normal tunnel advance rates, with contained extrusion of the core-face but not sufficient to affect the short term stability of the tunnel since the ground would still be able to generate sufficient residual strength.

This category was found in the flyschoid complexes (*Argille Varicolori* or marly-arenaceous ground) or in the stratified pyroclastites, as long as the overburdens were sufficient to make the natural formation of an arch effect possible.

Finally category C included all those sections of tunnel where numerical calculation predicted that:

- the state of stress in the ground would have exceeded the strength characteristics of the material considerably even in the zone around the face;
- an "arch effect" would have been formed naturally neither at the face nor around the tunnel since the ground would not have possessed sufficient residual strength;
- deformation phenomena would have developed into the failure range, with the effect deferred in time and be measurable in decimeters, giving rise to serious instability such as the failure of the face and the collapse of the cavity.

Colli Albani Tunnel. The portal of the Finestra 1 access tunnel

- as a consequence, in the absence of intervention to stabilise it, the core-face would have been completely unstable.

This category was found most frequently at portals and in general in sections with shallow overburdens, as well as in sections of clayey ground with a scaly structure belonging to the flysches of the *Argille Varicolori* with geotechnical characteristics close to the lower limits of the range identified (residual values). There is no way in which an arch effect can be formed in these cases except artificially.

Construction methodology (therapy phase)

After formulating reliable predictions of the stress-strain behaviour of the ground as a result of excavation, the most appropriate stabilisation techniques were chosen to control, contain or actually anticipate and eliminate deformation for each section of tunnel with uniform stress-strain behaviour.

The guiding principles on which the design of the tunnel section types was based are essentially as follows:

1. full face advance always, especially under difficult stress-strain conditions;
2. containment of the alteration and decompression of the ground around the tunnel by immediate application of effective preconfinement and/or confinement of the cavity (sub-horizontal jet-grouting, fibre glass structural elements in the core and/or in advance around the future tunnel and, if necessary, fitted with valves for cement injections, shotcrete, etc.) of sufficient magnitude, according to the case, to absorb a significant proportion of the deformation without collapsing or to anticipate and eliminate the onset of any movement in the ground whatsoever;
3. placing of a final concrete lining, reinforced if necessary, and completed, where necessary to halt deformation phenomena, with the casting of the tunnel invert in steps at a short distance from the face.

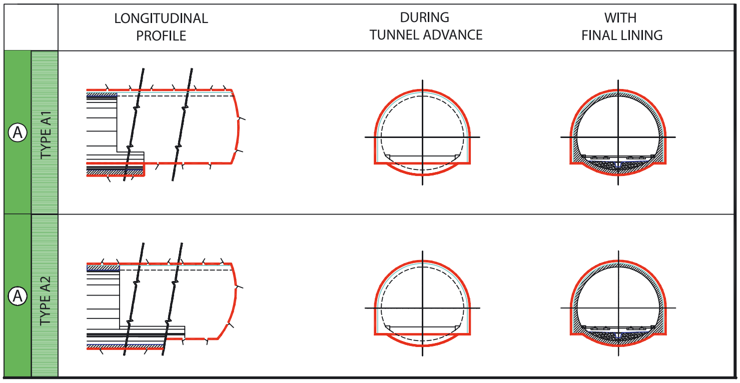
The following tunnel section types were actually designed (Fig. A.5):

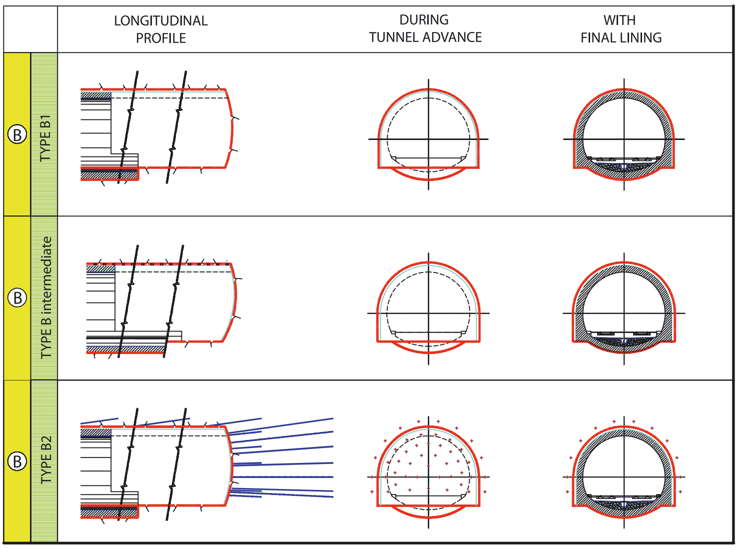
- **for sections of tunnel belonging to behaviour category A** (*stable core-face*) a type A section was designed consisting of a simple preliminary lining in sprayed concrete reinforced with simple steel ribs and a final lining in concrete 60 cm thick, closed with a tunnel invert also 60 cm thick;
- **for sections of tunnel belonging to behaviour category B** (*stable core-face in the short term*) three main tunnel section types were designed:

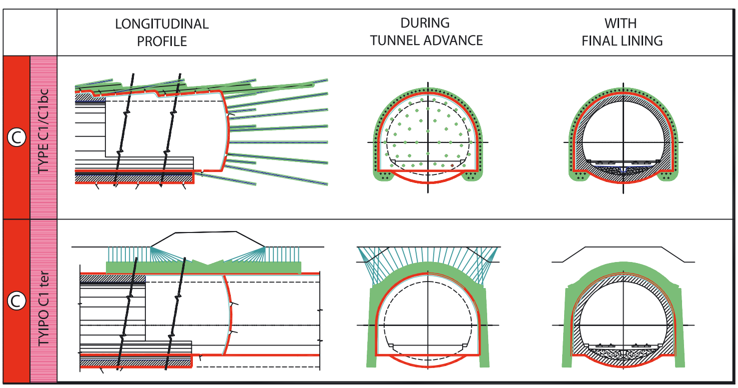
Colli Albani Tunnel. Work to reinforce the core-face, ground: vulcanites, overburden: ~30 m

Santuario Tunnel. Treatment of the ground with lime to create Artificial Ground Overburden (A.G.O.), type of ground: pyroclastites

Galleria Colli Albani. Detail of a rib next to the face


LONGITUDINAL PROFILE
DURING TUNNEL ADVANCE
WITH FINAL LINING
A
TYPE A1
A
TYPE A2


LONGITUDINAL PROFILE
DURING TUNNEL ADVANCE
WITH FINAL LINING
B
TYPE B1
B
TYPE B intermediate
B
TYPE B2


LONGITUDINAL PROFILE
DURING TUNNEL ADVANCE
WITH FINAL LINING
C
TYPE C1/C1bc
C
TYPO C1 ter

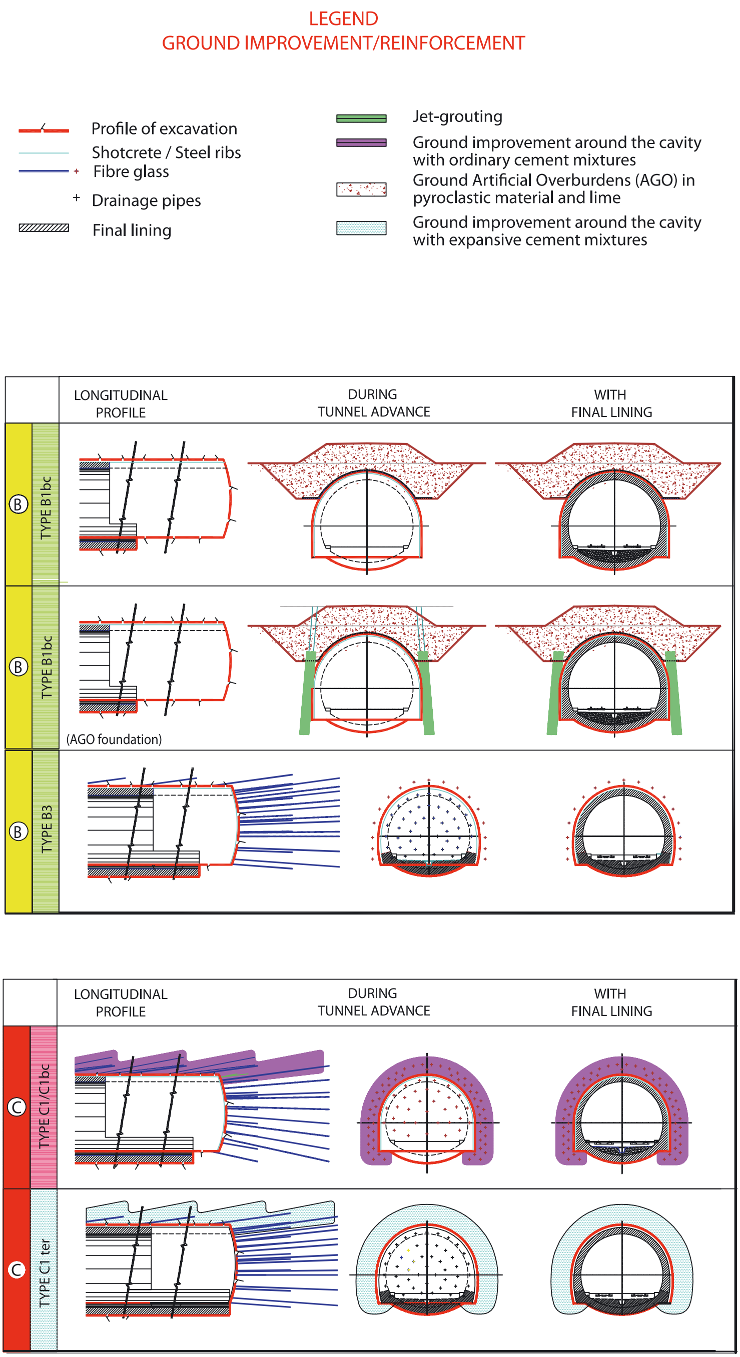


Fig. A.5. *Tunnel section types*

Macchia Piana Tunnel. Preparing the excavation scaffolding for the north portal

  - section type B1, consisting of a preliminary lining in shotcrete reinforced with double steel ribs + a final lining of 80 cm closed with a tunnel invert of 90 cm cast within 3 tunnel diameters from the face;
  - section type B2, for which reinforcement of the advance core is specified, performed using fibre glass structural elements + a preliminary lining in shotcrete reinforced with double steel ribs + a final lining of 90 cm closed with a tunnel invert cast within a distance of 1.5 tunnel diameters from the face;
  - section type B3, for which a geometry with curved tunnel walls is specified in order to withstand horizontal thrusts more effectively along with reinforcement of the advance core (more intensely than for B2 with more reinforcement and a longer overlap) again using fibre glass structural elements + a preliminary lining in shotcrete reinforced with double steel ribs + a final lining of 90 cm closed with a tunnel invert 100 cm thick cast in one piece with side kicker and floor slab in steps of 4–6 m from the face;
- **for sections of tunnel belonging to behaviour category C** (*unstable core-face*) two main types of tunnel section were designed:
  - section type C1, consisting of advance reinforcement of the ground around the tunnel using the technique of sub-horizontal jet-grouting + microcolumns of improved ground created using the same technique, but in the advance core and reinforced with fibre glass structural elements (15.5 m in length with overlap of 3 m) + a preliminary lining in shotcrete reinforced with double steel ribs + a final lining varying in thickness in the crown from 40 to 130 cm closed with a tunnel invert 100 cm thick, cast in steps 6–12.5 m in length, 1.5 tunnel diameters from the face;
  - section type C2, consisting of advance ground improvement around the future cavity using high pressure grout injections by means of valved fibre glass tubes + reinforcement of the advance core with grouted fibre glass structural elements (15 m in length, with an overlap of 5 m) + a preliminary lining in shotcrete reinforced with double steel ribs + a final lining of 90 cm closed with a tunnel invert 100 cm thick cast in steps 6–12.5 m in length, 1–1.5 tunnel diameters from the face.

Finally when the detailed design specifications were completed the percentages of different tunnel sections types with respect to the total length of the tunnels was as follows:

- section type A: 28.9%;
- section type B: 60%;
- section type C: 11.1%.

Piccilli 2 Tunnel. Construction of the "Artificial Ground Overburden (A.G.O.)" in the section with no overburden, ground: pyroclastites

In addition to the running tunnels, the detailed design also involved a few indispensible accessory works. The following are worthy of mention:

- three access tunnels for the passage of vehicles were driven to accelerate advance rates for the longest tunnels, working contemporaneously on several faces: two access tunnels on the *Colli Albani* tunnel (over 6 km in length) and a third access tunnel to the *Campo Zillone* tunnel (around 3 km in length);
- works for portals, which, depending on the morphology and the nature of the ground in question, were designed using the most appropriate technologies (shells of ground improved using jet-grouting methods, "berlin" walls, etc.).

Statics calculations

The statics and deformation behaviour of tunnels, both in the construction phases and the final service phase were analysed and verified by a series of calculations on three and two dimensional finite element models in the elasto-plastic range.

The models were developed to simulate the behaviour of the ground-tunnel system in the different construction phases, as realistically as possible. Particular attention was paid: to the effect of preconfinement treatments of the cavity and reinforcement of the advance core specified in the design; to deformation values to be expected at the constructions stage; and to stresses in the preliminary and final linings. The mathematical models were processed on computers using version 6.0 of the ADINA software package.

Contracts

Contracts for all the tunnels, just as for all the other surface works, required for construction of the line were of the "all-in, lump sum" type (2.844.644.600 euro, of which 324.231.600 for tunnels only) and awarded on the basis of detailed design specifications. With this type of contract the General Contractor IRICAV UNO accepted all risks including the geological risk.

Operational phase

Construction design began at the same time as excavation work (May 1995, *Colli Albani* tunnel) immediately after the contract was awarded.

Given the additional survey information available and direct in the field confirmation, the validity of the detailed design was substantially

Colli Albani Tunnel. Horizontal jet-grouting to improve the ground for the South portal

Macchia piana Tunnel. The scaffolding template for the North portal confirmed at the operational phase and only a few minor refinements were made during construction design:

- in order to specifically tackle the scaly clays, characteristic of the *Colle Pece* tunnel, a section type C3 was introduced that is different from C2, due above all to the use of expanding cement mixes for grouting of the fibre glass structural elements;
- a section type "$B1_{bc}$" for shallow overburdens and a section type "$C1_{ter}$" were designed for tunnel sections with particular design characteristics;
- section types A2 and an "$B1_{intermedia}$", very similar to A1 and B1 described above, were developed to optimise works in a few particular circumstances;
- admissible variations (e.g. intensity of reinforcement) were specified for each tunnel section type according to the actual deformation behaviour measured during construction as compared to that predicted by design calculations. This was done in order to apply quality assurance norms.

Finally, when the construction design was completed, the percentages of different tunnel section types were as follows:

- section type A: 22.5 %;
- section type B: 69.4 %;
- section type C: 8.2 %.

At the end of December 1999, after 1,100 working days (1,700 total days), approximately 21.6 km of tunnel had been completed, almost all fully lined, equal to about 99% of the underground sections.

Average advance rates were about 20 m/day, not counting excavation performed to open access tunnels, shafts and other accessory works.

Figure A.6 shows production graphs for the *Colli Albani* and *Sgurgola* tunnels from which it can be seen that production rates were not only high (around 100 m/month per face) but above all very constant, a sign that the construction design matched actual conditions excellently.

All the civil engineering works were completed in March 2001. Table A.1 gives a clear comparison of the differences in the distribution of tunnel section types between the detailed design, the construction design and "as built". These differences did not result in any significant change in the overall cost of the works, since the greater percentage of B section types was compensated for by a decrease in A and, above all, in C (by far the most costly) section types.

Monitoring phase

Monitoring during construction

The adequacy of the design hypotheses was verified by geostructural mapping of the face and monitoring stress-strain behaviour of the face and the cavity observed during construction for each stage and sequence specified in the design.

HIGH SPEED TRAIN - Milan to Naples Line - Rome to Naples Section - SECTION TYPES DISTRIBUTION

		DETAILED DESIGN								CONSTRUCTION DESIGN								AS BUILT (March 2001)								
WBS	TUNNEL	TUNNEL LENGTH [m]	SECTION TYPES (Lenght in metres)							TUNNEL LENGTH [m]	SECTION TYPES (Lenght in metres)							Builtsection		SECTION TYPES (Lenght in metres)						
			A	B 1	B 1 bc	B 2	B 3	C 1	C 2		A/A2	B 1	B 1 bc	B 2	B 3	C 1	C 2	(%)	[m]	A/A2	B 1	B 1 bc	B 2	B 3	C 1	C 2
GN01	COLLATINA	55							55	53		24					29	100.0	53	0	24					29
GN02	MASSIMO	1149	1092						57	1139	664	355					120	100.0	1139	766	236					137
GN03	COLLI ALBANI	6357	1462	3115				1780		6361	541	5425				355	40	100.0	6361	610	5355				356	40
GN10	SGURGOLA	2240	560	1680						2237	19887	250						100.0	2237	1615	622					
GN11	MACCHIA PIANA 1	1150	920	230						969	834	63					72	100.0	969	831	66					72
GN12	MACCHIA PIANA 2	540	540							212						191	21	100.0	212						175	37
GN13	LA BOTTE	1185	687			379			119	1253	300	348		342		263		100.0	1253	344	256		395		258	
GN14	CASTALLONA	470	249	221						430		178		173	79			100.0	430		159,5		231	39.5		
GN15	S.ARCANGELO	580				116	406		58	580		380		200				100.0	580		507		73			
GN16	SELVA PIANA	170	119	51						132		49		49		34		100.0	132		90		8		34	
GN17	COLLEVENTO	387				348			39	380		100		229,5		50,5		100.0	380		107		248		25	
GN18	SELVOTTA	173				173				164				31		133		100.0	164				33		131	
GN19	COLLE PECE	873				611	175		87	873		253		389	149	42	40	100.0	838		153		169	254	62	200
GN31	CAMPO ZILLONE 1	3163		1423		1740				2617		1565		801		251		100.0	2617		2586		15		15	
GN25	CAMPO ZILLONE 2	350				350				350		334				16		100.0	350		334				16	
GN26	PICILLI 1	845	127	591				127		907		553	58	296				100.0	907		677	58	98		74	
GN27	PICILLI 2	229		195				34		485		266	219					100.0	485		249	236				
GN28	LOMPARI	235		189			47			199		155				44		100.0	199		155				44	
GN29	CAIANELLO	210		168			21	21		832		474	334			24		100.0	832		477	331			24	
GN30	BRICCELLE	1033	424	609						1033	581	10		218	180	4	40	100.0	1033	582	10		218	180	4	39
GN32	CASTAGNE	0								289		189	100					100.0	289		184	105				
GN33	SANTUARIO	0								322		242	80					100.0	322		242	80				
TOTAL LENGHT [m]		21394	6180	8472		3717	649	1962	415	21817	4907	11212	791	2729	408	1408	362	100.0	21782	4748	12490	810	1488	474	1219	544
			A	B 1	B 1 bc	B 2	B 3	C 1	C 2		A/A2	B 1	B 1 bc	B 2	B 3	C 1	C 2			A/A2	B 1	B 1 bc	B 2	B 3	C 1	C 2
% SECTION TYPES USE			28.9	39.6	0.0	17.4	3.0	9.2	1.9		22.5	51.4	3.6	12.5	1.9	6.5	1.7			21.8	57.3	3.7	6.8	2.2	5.6	2.5
											TOTAL SECTION TO BE TUNNELED (%/m)							0.0	0							

Table A.I *Percentage use of different tunnel section types: a comparison between detailed design, construction design and "as built"*

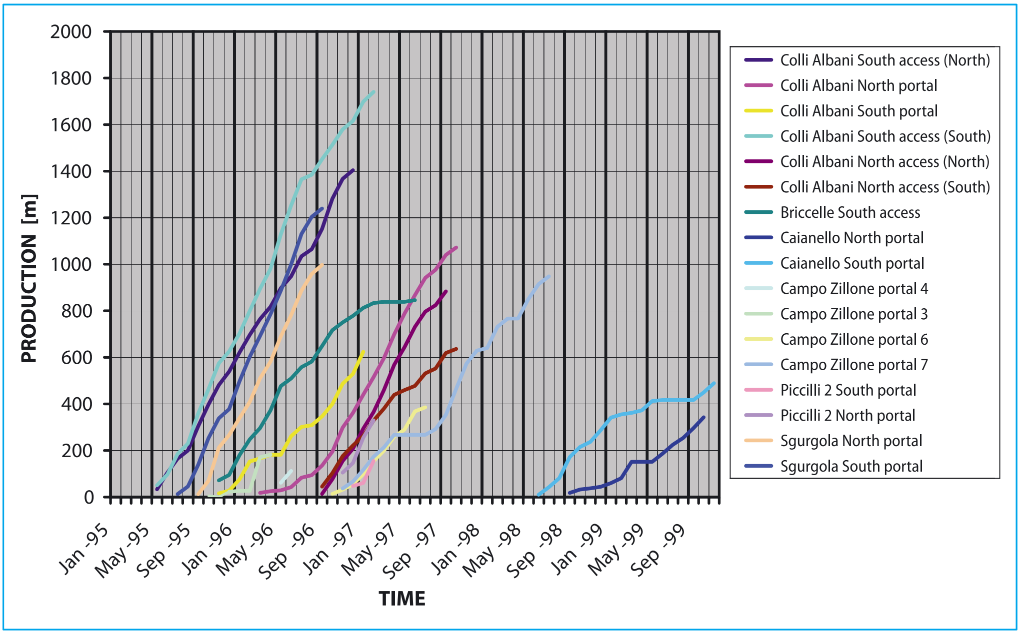


Fig. A.6 *The very distinct linear nature of the curves is a result of the high level of industrialisation achieved*

To achieve this a complete monitoring system was devised that included:

1. geological and structural mapping of the face;
2. face extrusion measurements;
3. convergence measurements;
4. extensometer measurements from the surface;
5. surface levelling measurements (on sections with shallow overburdens);
6. inclinometer measurements (at portals);
7. laboratory tests on samples taken from the face.

Final monitoring

Twenty two completely automated permanent monitoring stations for digital recording of data were located along the rail tunnels.

Their purpose was to:

1. verify design hypotheses;
2. monitor the behaviour of the tunnels over time;
3. provide information for maintenance purposes.

The following was performed as part of monitoring during construction:

1. convergence measurements to determine radial deformation;

Macchia Piana Tunnel. Demolition of the core-face using a demolition hammer (ground: tuff, overburden: ~3 m)

Macchia Piana Tunnel. Work on the north portal (ground: rubble)

Colli Albani Tunnel. The junction with the south access tunnel, ground: vulcanites, overburden: ~ 40 m

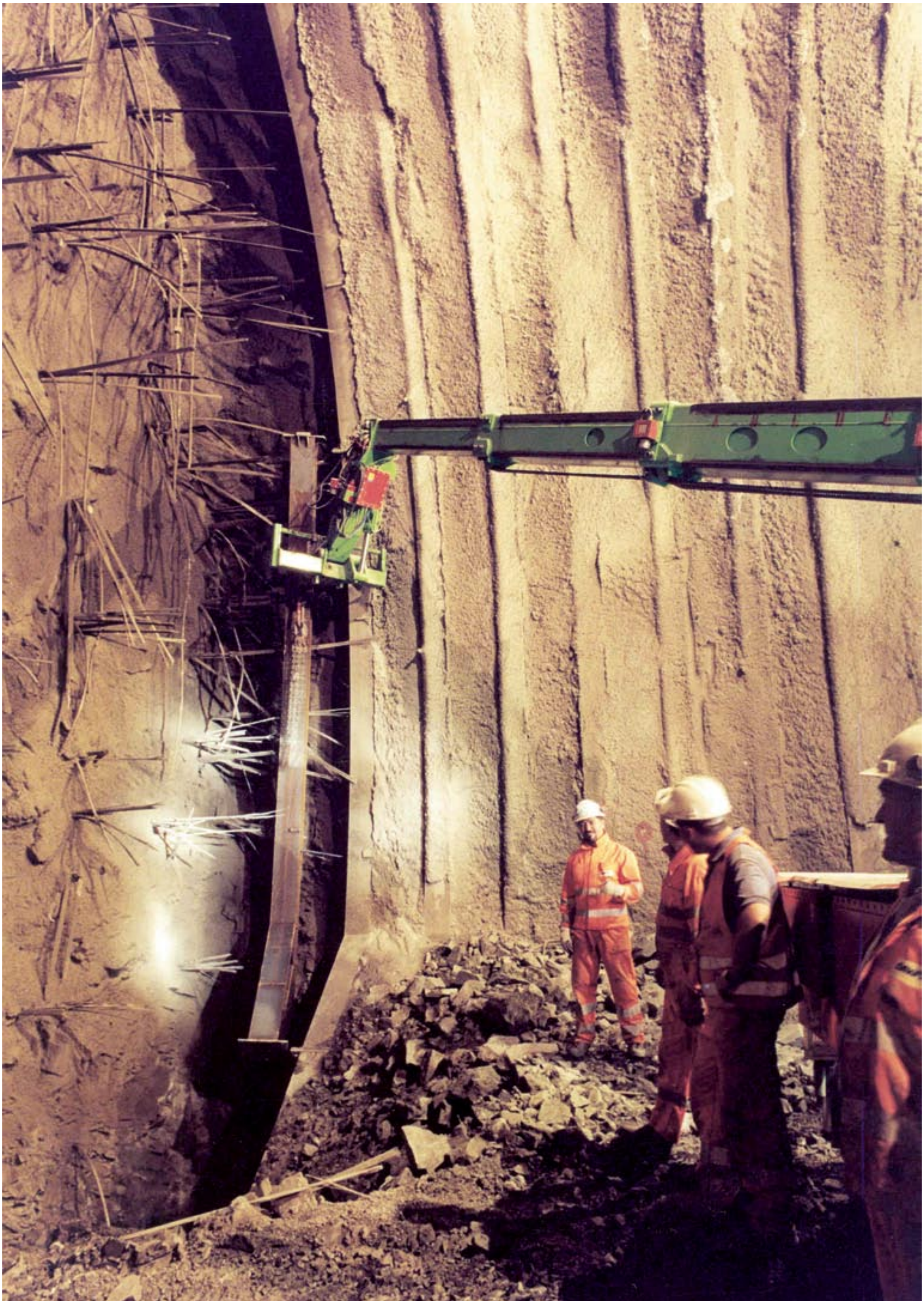
2. extensometer measurements to assess the development of deformation inside the ground;
3. measurements of total pressure of the ground on the lining;
4. measurements of pore pressure in the ground;
5. measurements of stress in structural members;
6. measurements of the temperature of the concrete in the final lining;
7. measurements of vibrations induced in structures by the passage of trains.

Conclusions

On the basis of the final data reported above it can be stated that the design predictions made for the tunnels on the High Speed Rome-Naples line using the ADECO-RS approach were found to match the reality very closely despite the geological difficulty of the sites tunnelled.

When account is taken of the ground involved and of some objectively difficult conditions that had to be tackled, the fast average advance rates achieved constitute a good indicator of both the high standard and the reliability of the design using ADECO-RS principles and the high degree of industrialisation of the tunnel advance operations achieved on construction sites, as a consequence.

The underground works were completed on schedule in 2001. Construction costs too differ by only a few percentage points from forecasts and there were never any serious disputes between the contractors and the client.



High speed/capacity Bologna-Florence railway line, Pianoro tunnel. Placing a steel rib next to the face

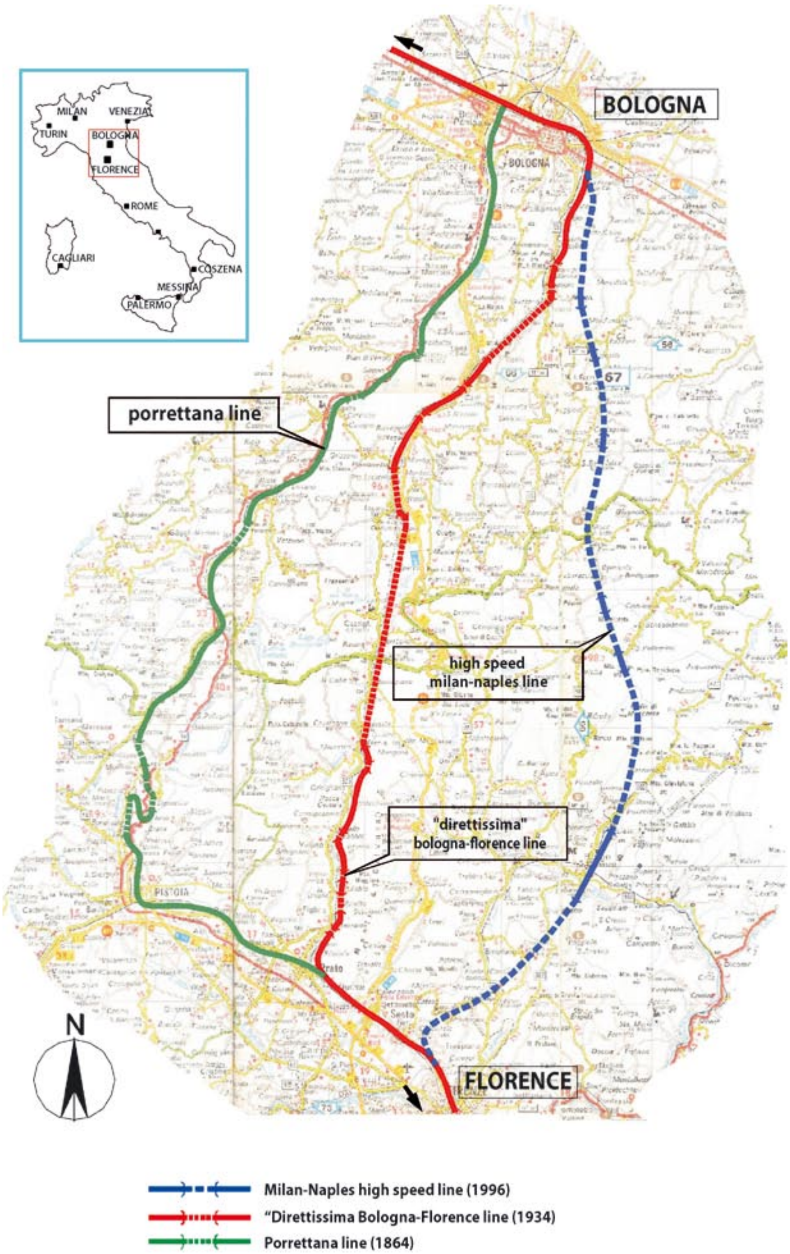
The design and construction of tunnels for the new Bologna-Florence high speed/capacity railway line

A look at the past

It is interesting to consider how the history of the railway connection between Bologna and Florence across the Tuscan Emilian Apennines has gone hand in hand with the emergence of railways as the main system of land transport for goods and passengers in a context of socio-political development which saw exponential growth in trade between southern and central Italy in the period between the unification of the country and its recent entrance into the European Union.

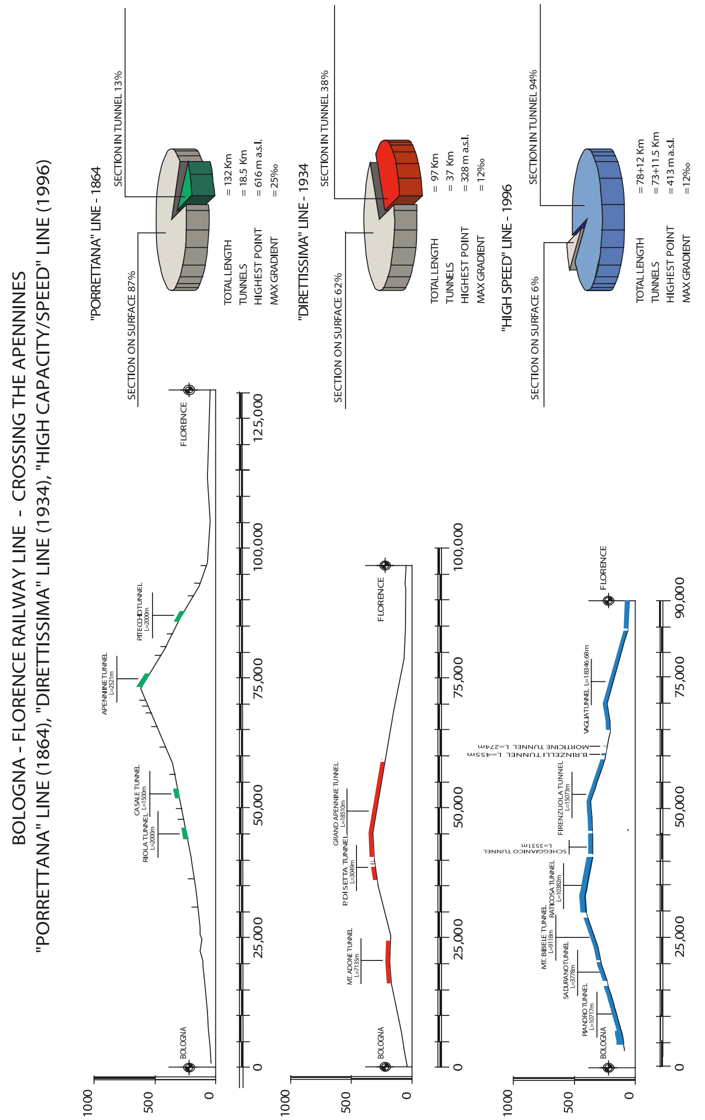
Work started on the first connection between Bologna and Florence in 1856 and it went into service in the November of the 1864. The alignment which follows the design

"Porrettana" Line: Construction of the South portal of the Signorino tunnel (1862 ca.)



BOLOGNA
FLORENCE
porrettana line
high speed
milan-naples line
"direttissima"
bologna-florence line
MILAN
VENEZIA
TURIN
BOLOGNA
FLORENCE
ROME
CAGLIARI
COSZENA
MESSINA
PALERMO
N
Milan-Naples high speed line (1996)
"Direttissima Bologna-Florence line (1934)
Porrettana line (1864)

Fig. B.1

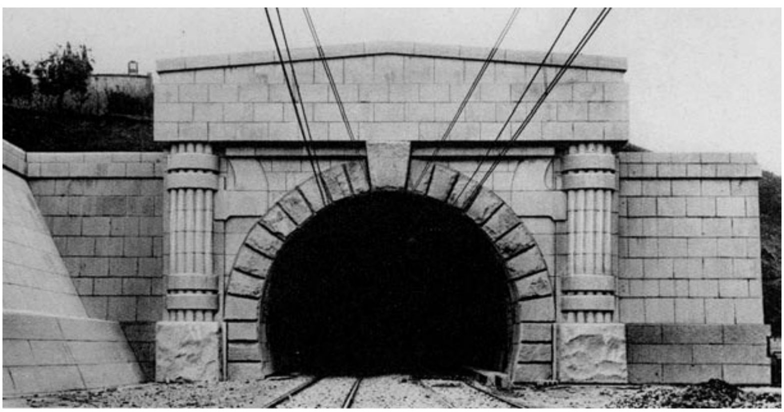


BOLOGNA - FLORENCE RAILWAY LINE - CROSSING THE APENNINES
"PORRETTANA" LINE (1864), "DIRETTISSIMA" LINE (1934), "HIGH CAPACITY/SPEED" LINE (1996)
"PORRETTANA" LINE - 1864
SECTION ON SURFACE 87%
SECTION IN TUNNEL 13%
TOTAL LENGTH = 132 Km
TUNNELS = 18,5 Km
HIGHEST POINT = 616 m a.s.l.
MAX GRADIENT = 25‰
"DIRETTISSIMA" LINE - 1934
SECTION ON SURFACE 62%
SECTION IN TUNNEL 38%
TOTAL LENGTH = 97 Km
TUNNELS = 37 Km
HIGHEST POINT = 328 m a.s.l.
MAX GRADIENT = 12‰
"HIGH SPEED" LINE - 1996
SECTION ON SURFACE 6%
SECTION IN TUNNEL 94%
TOTAL LENGTH = 78+12 Km
TUNNELS = 73+11.5 Km
HIGHEST POINT = 413 m a.s.l.
MAX GRADIENT = 12‰
BOLOGNA
FLORENCE
MT. ADONE TUNNEL
GRAND APENNINE TUNNEL
B.RINZELLI TUNNEL L=455m
MORTICINE TUNNEL L=274m
SCHEGGIANICO TUNNEL L=3531m
FIRENZUOLA TUNNEL
RATICOSA TUNNEL
MT. BIBELE TUNNEL
SADURANO TUNNEL
PIANORO TUNNEL
VAGLIA TUNNEL L=18346.68m
0
500
1000
25,000
50,000
75,000
90,000
100,000
125,000

Fig. B.2

Bologna-Florence Direttissima Line, excavation of the Grand Apennine Tunnel: reinforcement in the crown with steel ribs

Bologna-Florence Direttissima Line, excavation of the Station chamber from the previous Grand Apennine Tunnel: steel ribs in the crown and side drifts



Bologna-Florence Direttissima Line, the Grand Apennine Tunnel: the portal on the Florence side by *ing.* Protche of Bologna is that of the Porrettana line (Fig. B.1), still in existence today, which winds along the Reno valley to pass over the Apennines at Pracchia (altitude 616 m a.s.l.) and reach Florence passing through Pistoia.

It was realised before work on the construction of this railway had even finished that because of its high gradients it would soon be insufficient to handle all the vigorously growing traffic between the Po river valley and the capital. A new more direct connection, and above all one with greater capacity, had therefore to be established between Bologna and Florence as soon as possible. The problem, examined by the top civil engineers of the period, gave rise to a series of proposals with different vertical and horizontal alignments. What they had in common was that in their desire to limit the length of the main tunnel, they were all induced to maintain the maximum altitude at around 500 m a.s.l. accepting gradients only a little lower than those of the existing line and hardly improving capacity.

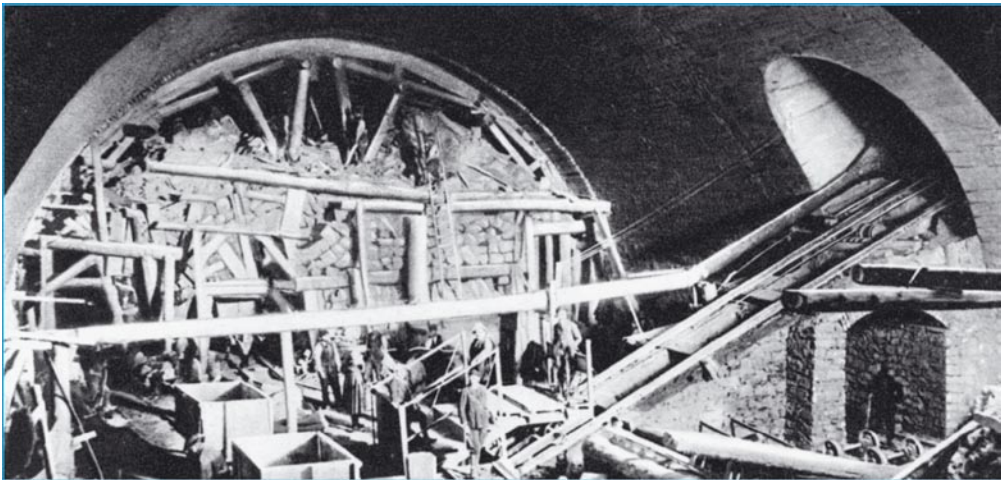
In 1882 the authorities concerned appointed *ing.* Protche to examine the design drawn up by *ing.* Zannoni in 1871 and to propose any improvements he might see necessary. Protche's proposals were for a line that followed the Setta and Bisenzio valleys to connect with the existing line at Sasso Marconi and Prato; the Apennines would be crossed by a large 18,032 m tunnel with the highest point at 328 m a.s.l., which would give a maximum gradient of 12‰.

In 1902 the Upper Council of Public Works set up a special commission to examine the studies that had been presented at that time. The commission chose the Protche design because the gradients and the greater stability of the terrain along the alignment came closest to meeting requirements for the line. The ministerial commission chaired by the senator *ing.* G. Colombo, did not limit itself to selecting the Protche design but also studied detailed alternatives and chose the best.

In 1908 the government enacted Law No. 444/1908 on the basis of the Colombo commission report and the final design studies were to commence, with 150 million lire authorised for the construction of the Bologna-Florence *Direttissima*.

In reality construction work didn't start until 1913 and then it proceeded very slowly and intermittently because of the First World War and subsequent events which greatly upset the life of the country in that period. Continuous work didn't start until 1921 and the "Direttissima" was finally inaugurated in 1934 with total expenditure amounting to £ 1,122 million, 460 of which was spent on the Grand Apennine Tunnel alone.

Fifty years after the Bologna-Florence Direttissima was commissioned new demands for greater capacity arose. It was above all the demand to bring the national railway network up to the same standards as the European model to make it an integral part of the continental high speed network that led to the development of the high speed/capacity train project.



Bologna-Florence Direttissima Line, excavation of the Station chamber from the previous Grand Apennine Tunnel: access stairways on the right

This project redesigns the Italian railway network by quadrupling lines with new high speed/capacity routes that run East-West across the Po valley and North-South down the peninsula. It is because of this latter development that the Bologna-Florence section constitutes the greatest design and construction commitment.

Without going into the details of the TAV (*Treno ad Alta Velocità* – High Speed Train) project (described later) in this brief history, let it suffice to say that the State Railways granted the concession for the design, construction and management of the new lines for a period of 50 years to the company, TAV S.p.A., formed in 1991.

TAV then selected a number of large groups of companies as general contractors responsible for the design and construction of these lines. The group selected for the Bologna-Florence section was the FIAT group which sub contracted the design and construction to FIAT Engineering S.p.A. and the CAVET consortium. Rocksoil S.p.A of Milan was selected for the design of the underground works.

Study of the environmental impact and of the executive design of the works began at the beginning of 1992 when TAV delivered the general design for the alignment to FIAT. An alignment to the east of the existing route was decided on the basis of those studies with a total length of 78 km, of which 73 km underground and 5 km on the surface; the maximum altitude was 413 m a.s.l..

To conclude this brief history, Fig. B.2 illustrates the development of the routes from the 1864 "Porrettana" line to the future high speed/capacity line. It perhaps summarises the technological progress that has been made and the quality of the infrastructure of the latest line better than other descriptions. While the progress made by the Porrettana compared to the *Direttissima* line consisted of improving the vertical and horizontal geometry of the alignment (decreasing the maximum gradient, increasing the radii of curvature and shortening the length of the alignment, with the final result that it allowed greater speed), what changed with the new high speed/capacity line was not only the much broader geometrical parameters, but above all the design mentality of giving due importance to environmental factors and therefore resorting to the underground option as much as possible, thanks to the development of new construction technologies.

The new Bologna-Florence high speed/capacity railway line

The new Bologna-Florence high speed/capacity railway line across the Apennines consists of an alignment 78.5 km in length of which a good 70.6 km (90% approx.) passes through twin track underground tunnels driven through the insidious and complex terrain of the hills and mountains in those places.

The project involves the construction of:

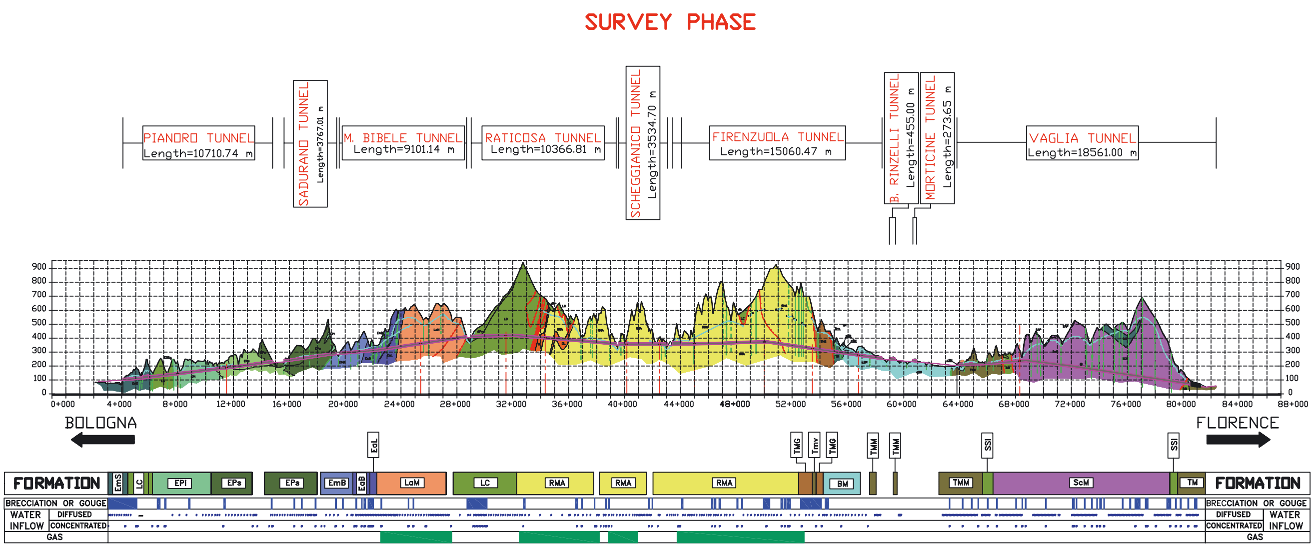
- 9 nine running tunnels with a cross section of 140 sq.m and lengths of between 528 m and 16,775 m;
- 14 access tunnels for a total of 9,255 m;
- 1 service tunnels for a total of 10,647 m;
- 2 connecting tunnels passage way tunnels for a total of 2,160 m

The works are at an advanced state of completion with excavation of all the running tunnels completed in October 2005.

Design

The geological-geotechnical context (survey phase)

The exceptional complexity of the ground involved was well known. It had already been tackled with great difficulty for the construction of the "*Direttissima*" railway line inaugurated in 1934 still in service today. A sum of € 84 million, 2% of the total cost of the project, was therefore invested in the geological survey campaigns required for final design. This provided a geological-geomechanical characterisation of the ground to be tunnelled that was very detailed and above all accurate. As is shown in Fig. B.3, it consisted primarily of flyschioid formations, clays, argillites and loose soils, at times with extensive water tables, which covered more than 70% of the underground alignment, with overburdens varying between 0 and 600 m. Some of the formations also presented the problem of gas, always insidious and delicate to deal with. During the survey phase, the route was divided into sections with similar geological and geomechanical characteristics on the basis of the information acquired. Strength and deformation parameters most representative of the sections were attributed to these for the subsequent diagnosis and therapy phases.



FORMATION		LENGTH [m]	OVERBURDEN [m]	Cohesion [Mpa] Angle of friction [∞]		Modulus of elasticity [Gpa]
Marne in facies di Schlier (EmS)		2012	0–120 (Pianoro Tunnel)	c = 0.25–0.3	φ = 28–30	E = 2.9–3.4
Pliocene Intrappenninico Superiore (EpS)		11445	0–245 (Sadurano Tunnel)	c = 0.5–0.7	φ = 28–30	E = 3.0–6.0
Marne di Bismantova		7055	0–140 (Monte Bibele Tunnel)	c = 0.8–1.35	φ = 29–33	E = 4.0–6.0
Flysch di Monghidoro (LaM)		9115	0–290 (Monte Bibele Tunnel)	c = 0.12–0.5	φ = 23–35	E = 3.0–8.5
Complesso Caotico (LC)		11187	0–520 (Raticosa Tunnel)	c = 0.03–0.7	φ = 10–25	E = 0.1–1.8
Marly-Sandstone Formation (RMA)		39961	0–550 (Firenzuola Tunnel)	c = 2.0–2.2	φ = 24–38	E = 15–20
Clays of Mugello Basin (aBM)		7404	0–80 (Firenzuola Tunnel)	c'_p = 0.03	φ'_p = 26–28	E = 0.01–0.3
Monte Morello Formation (ScM)	limestones	13861	25–560 (Vaglia Tunnel)	c = 1.0–1.4	φ = 42–45	E = 7–15
	clays and marls			c = 0.18–0.24	φ = 28–33	E = 2–8

Fig. B.3. *Survey phase: variability of strength and deformation parameters for the main formations*

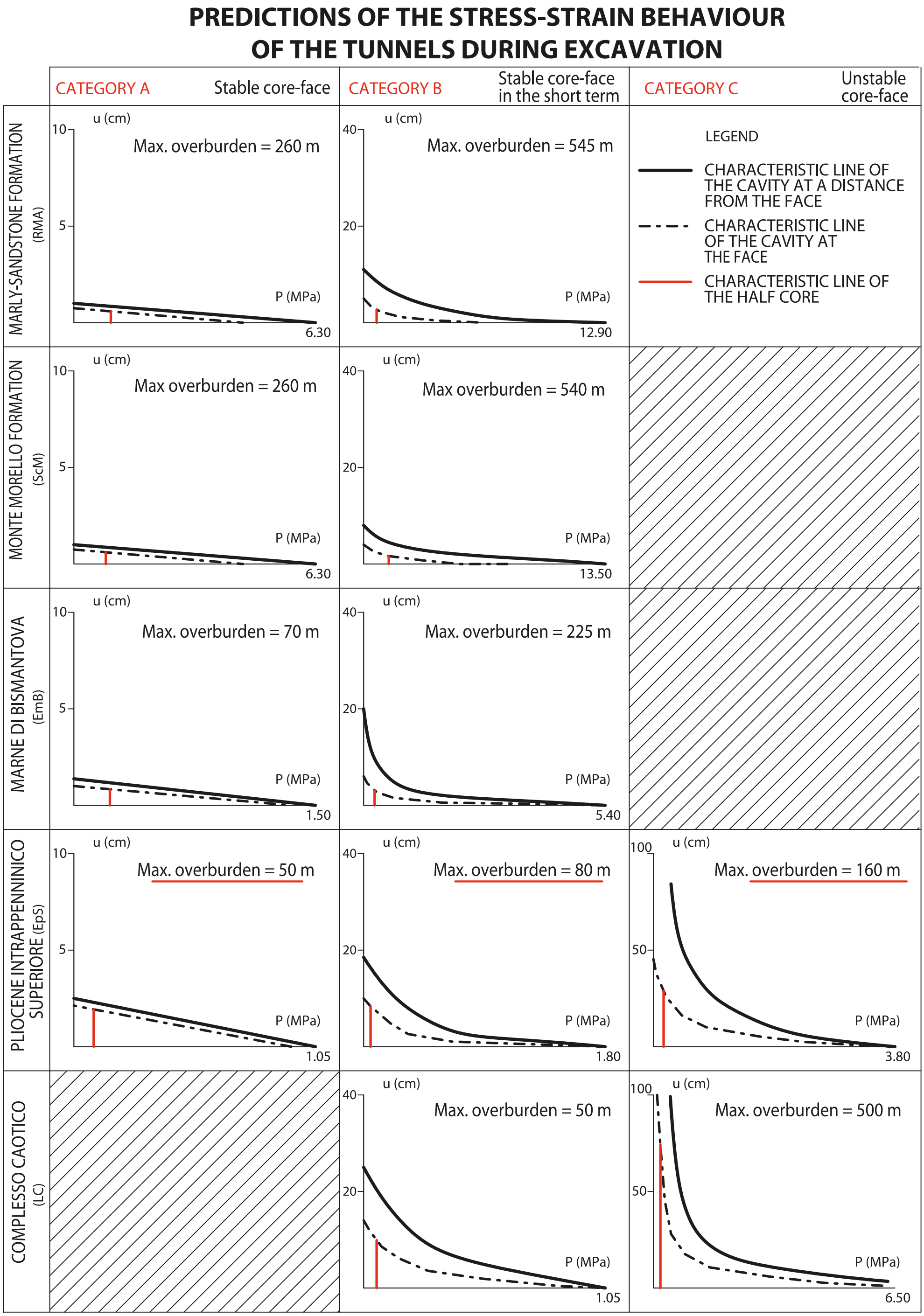

PREDICTIONS OF THE STRESS-STRAIN BEHAVIOUR OF THE TUNNELS DURING EXCAVATION
CATEGORY A
Stable core-face
CATEGORY B
Stable core-face in the short term
CATEGORY C
Unstable core-face
MARLY-SANDSTONE FORMATION (RMA)
MONTE MORELLO FORMATION (ScM)
MARNE DI BISMANTOVA (EmB)
PLIOCENE INTRAPPENNINICO SUPERIORE (EpS)
COMPLESSO CAOTICO (LC)
u (cm)
P (MPa)
Max. overburden = 260 m
Max. overburden = 545 m
Max overburden = 260 m
Max overburden = 540 m
Max. overburden = 70 m
Max. overburden = 225 m
Max. overburden = 50 m
Max. overburden = 80 m
Max. overburden = 160 m
Max. overburden = 50 m
Max. overburden = 500 m
6.30
12.90
6.30
13.50
1.50
5.40
1.05
1.80
3.80
1.05
6.50
LEGEND
CHARACTERISTIC LINE OF THE CAVITY AT A DISTANCE FROM THE FACE
CHARACTERISTIC LINE OF THE CAVITY AT THE FACE
CHARACTERISTIC LINE OF THE HALF CORE

Fig. B.4

Monte Bibele Tunnel. View of the face while tunnelling through the Bismantova Marls, overburden: ~ 40 m)

Monte Bibele Tunnel. View of the face while tunnelling through the Monghidoro Flysch, overburden: ~ 200 m)

Predicting the behaviour of the rock and soil masses in response to excavation (diagnosis phase)

Whether because of the different geotechnical and geomechanical characteristics of the ground or the different overburdens, the tunnels to be constructed would very definitely be driven under extremely different stress-strain conditions. The underground route was therefore divided, in the diagnosis phase, into sections with uniform stress-strain behaviour. This was performed using the stability of the core-face in the absence of stabilisation measures as the only and universal parameter, which can be predicted according to the criteria illustrated in chapter 7, and also on the basis of the geological, geotechnical, geomechanical and hydrogeological information acquired in the survey phase. The sections were as follows:

- the core-face would in all probability be stable (behaviour category A; deformation phenomena in the elastic range, prevailing manifestations of instability: rock fall at the face and around the cavity);
- the core-face would in all probability be stable in the short term (behaviour category B; deformation in the elasto-plastic range; prevailing manifestations of instability: spalling at the face and around the cavity);
- the core-face would in all probability be unstable (behaviour category C; deformation phenomena in the failure range: consequent manifestations of instability: failure of the face and collapse of the cavity).

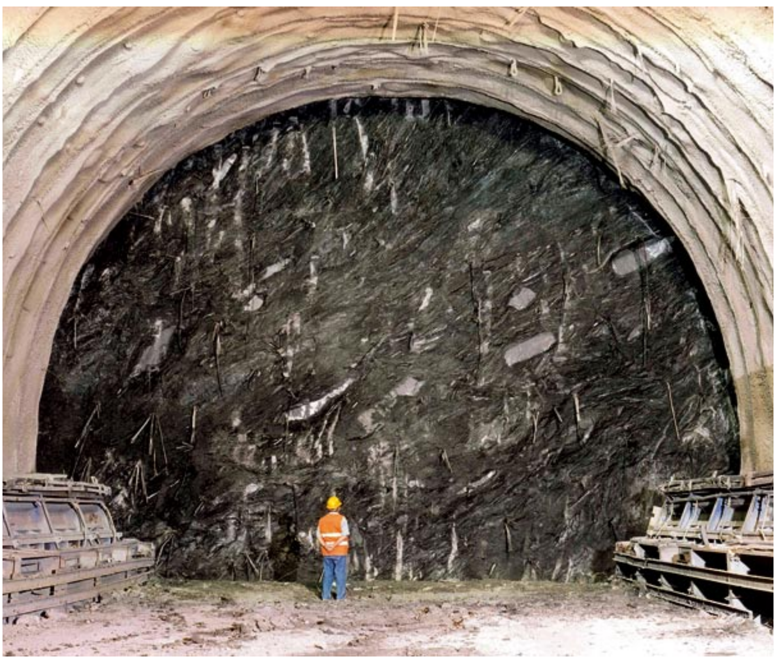
It was found from this analysis that 17% of the route would pass through ground which would have reacted to excavation with behaviour in category A, while it was predicted that 57% would be affected by deformation phenomena in the elasto-plastic range of behaviour category B and finally approximately 26% would have been characterised, in the absence of appropriate intervention, by serious instability of the core-face typical of behaviour category C.

Definition of excavation methods and stabilisation measures (therapy phase)

Once reliable predictions of the stress-strain response of the ground to excavation had been formulated, the action required (preconfinement and/or ordinary confinement) to guarantee the formation of an arch effect as close as possible to the profile of the excavation, in each situation hypothesised, was identified for each section of tunnel with uniform stress-strain behaviour. The advance methods (method of actual excavation, length of tunnel advances) and the most appropriate techniques for producing the required action and to guarantee, as a consequence, the long and short term stability of the excavation were then designed.

The variable character of the ground, present to a greater or lesser extent in all the tunnels, meant that totally mechanised technologies were not advisable with the exception of the service tunnel for the Vaglia tunnel. The main principles on which the design of the tunnel section types was based were therefore as follows:

Raticosa Tunnel. Two moments in the reinforcement of the advance core using fibre glass structural elements. Drilling and insertion (left); cementation (right)



Raticosa Tunnel. Reinforcement of the core-face by means of fibre glass structural elements in the scaly clays of the Complesso Caotico (overburden: ~ 550 m)

1. full face tunnel advance always, especially under difficult stress-strain conditions: due to its peculiar static advantages and because large and powerful machines can be profitably used in the wide spaces available, it is in fact possible to advance in safety with excellent and above all constant advance rates even through the most complex ground by using full face advance after core-face reinforcement, when necessary;
2. confinement, where necessary, of the alteration and decompression of the ground caused by excavation, with the immediate application of effective cavity pre-confinement and/or confinement (sub-horizontal jet-grouting, fibre glass structural elements in the core and/or in advance around the cavity, fitted, if necessary, with valves for pressure cement injections, shotcrete, etc.) of dimensions sufficient, according to the case, either to absorb a significant proportion of the deformation without collapsing or to anticipate and neutralise all movement of the ground from the outset;
3. placing a final lining in concrete, reinforced if necessary, complete with the casting of a tunnel invert at short intervals immediately behind the face when the need to halt deformation phenomena promptly was recognised.

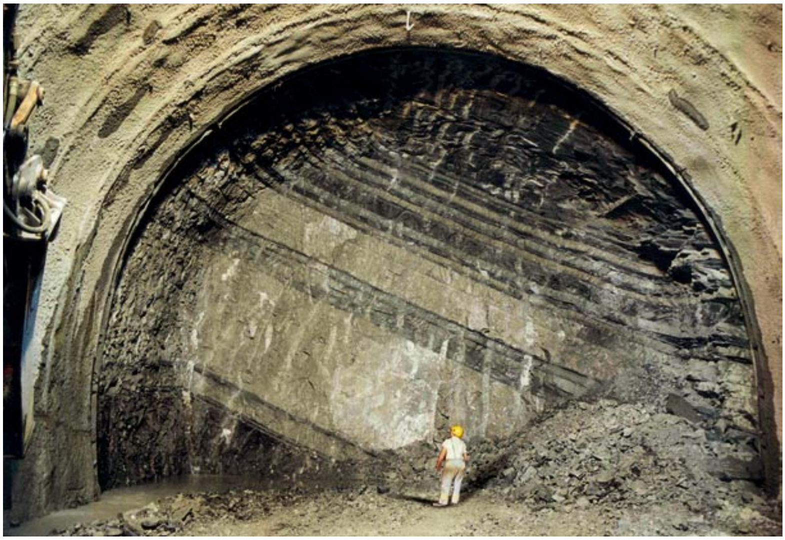
   The follow design decisions were then made.

- **Pliocene Intrappenninico Superiore** (Pianoro tunnel): the just reasonable geomechanical characteristics of the ground in this formation and the stress states found resulted in the prediction of behaviour varying from stable core-face in the short term to unstable core-face (categories B and C). No intervention to improve the ground seemed necessary in the first case (which was prevalent), as long as a fast advance rate was maintained (higher than 2 m/day). The section types B0 and B2, considered appropriate for the expected operating conditions, were specified. Given the average strength of the matrix, an excavator hammer was specified as the means for excavating the ground. It was therefore considered that an advance step of 1.5 m, with the face profiled to give it a concave shape, would allow a steel rib to be placed on each eight hour shift and ensure the necessary speed of tunnel advance required.
- **Marne di Bismantova** (M. Bibele North tunnel): the consistency of the ground is that of rock with little fracturing, consisting of approximately two kilometres of marls followed by sandstones interbedded with marly strata. The diagnosis study found mainly stable core-face behaviour (category A), generally without the need for improvement of the rock mass. The section types Ab, Ac, B0, were therefore specified. Given the characteristics of the rock (σ_c >30 MPa), conventional excavation was specified by blasting with one round of shots per day to give tunnel advances of 4.5–5 m.

Firenzuola Tunnel. View of the face while tunnelling through the Marnoso-Arenacea formation (overburden: ~ 500 m)

Vaglia Tunnel. View of the face reinforced using fibre glass structural elements while tunnelling through the Clays of the Mugello basin (overburden: ~ 15 m)

- **Flysch di Monghidoro** (M. Bibele South tunnel): consists of densely alternating, heavily tectonised and intensely fractured argillites, clayey marls, limestones and calcarenites. Since the prevalent behaviour type found in the diagnosis phase was type B (stable core-face in the short term), with possible transformations to C (unstable core-face), the section types B0 and B2 were designed. Consequently, excavation would take place with and without intervention to improve and reinforce the ground. Given the marked fracturing of the ground, excavation using an excavator hammer was opted for, despite the rock consistency of the matrix, with maximum tunnel advances of 2,5 m in the better quality rock and up to 1 m in the ground consisting mainly of argillites.
- **Complesso Caotico** (Raticosa Tunnel): this consists of intensely fractured and tectonised scaly clays of poor geomechanical quality, to be excavated under high stress states (behaviour category C: unstable core-face). Full face advance after first reinforcing the advance core and the ground around the future tunnel with fibre glass structural elements was specified given the difficult conditions (section types C4R, C4V). The walls of the tunnel were then to be immediately lined with a ring of steel ribs closed in the tunnel invert with a strut to brace it and with shotcrete. The sidewalls and the tunnel invert were then to be cast within one tunnel diameter from the face. A ripper, with which it was easier to profile the face with a concave shape, was specified to excavate this soft and rather inhomogeneous ground. Finally, a tunnel advance of approximately 1.2 m was considered adequate.
- **Marly-Sandstone Formation** (Scheggianico and Firenzuola tunnels): this is an aquifer consisting of alternating marls and sandstones in sub-horizontal banks ranging from a few decimetres to some metres. Given the good strength and deformability characteristics of the material, it responds to excavation in the plastic range even under the largest overburdens (category A: stable core-face). Consequently section types Ab and Ac were specified, while the good mechanical quality of the rock made blasting the best method of excavation even if this would cause difficulties in profiling the tunnel cross section.
- **Clays of the Mugello Basin** (Firenzuola Tunnel): these consist of clayey silts with fine sandy and saturated interbedding. It was predicted that tunnel advance would take place under behaviour category C (unstable core-face). Appropriate intervention was therefore taken to improve and reinforce the ground (section types C1, C4R, B2). A ripper excavator was specified for excavation with immediate placement of the lining (sidewalls and tunnel invert cast within one tunnel diameter of the face).
- **Monte Morello Formation** (Vaglia Tunnel): this is a flysch consisting of limestones, marly limestones and poorly fractured



Vaglia Tunnel. View of the face while tunnelling through the Monte Morello formation (overburden: ~ 600 m)

S. Giorgio access tunnel. Construction of the junction with the Firenzuola running tunnel (ground: Clays of the Mugello Basin, overburden: ~ 30 m) marls in compacted banks of a few decimetres. Given the good geomechanical quality, the main behaviour found was category A (stable core-face). Section types Ac or B0 were therefore specified. Blasting was the only choice for excavation considering the characteristics of the material.

Table B.I summarises the design specifications described.

The range of geological and geomechanical and stress-strain (extrusion and convergence) conditions within which they were to be applied was clearly defined for each type of longitudinal and cross section types as well as the position in relation to the face, the intensity and the phases and intervals for placing the various types of intervention (advance ground improvement, preliminary lining, tunnel invert etc). Very reliable work cycles based on a considerable number of previous experiences was drawn up from which precise predictions of daily advance rates could be made. Figure B.5 shows the main section types adopted (there were 14 in all), grouped according to the type of behaviour category, A, B and C. The 'variabilities' to be applied were designed for each section type for statistically probable conditions where, however, the precise location could not be predicted on the basis of the available data (see section 8.9).

It is essential to identify the variabilities for each tunnel section type that are admissible in relation to the actual response of the ground to excavation, which will in any case always be within the range of deformation predicted by the ADECO-RS approach. This is because it allows a high level of definition to be achieved in the design and also at the same time the flexibility needed to be able to adopt ISO 9002 quality assurance systems during construction to advantage. By employing this method, non

FORMATION	TUNNEL	σ_C [MPa]	BEHAVIOUR CATEGORIES	H_2O	SECTIONS TYPE	FACTORS AFFECTING EXCAVATION DESIGN			
						Ground improvement or reinforcement intervention	Excavation system	Excav. step [m]	Production predicted
Pliocene Intrappenninico (EPs -EPi)	Pianoro South	12	B, (C)		B0, B2 (C1, C2)	sporadic	hammer	1.5	2
Marne di Bismantova (EmB -EaB -EaL)	Monte Bibele North	30	A, (B)		Ab, Ac (B0)	sporadic	blasting	4.5–5	5
Flysh di Monghidoro [LaM]	Monte Bibele South	8	B, (C)		B0, B2 (C4)	sporadic	hammer	1–2.5	2
Complesso Caotico [LC]	Raticosa North	2	C		C4R, C4V	systematic	ripping	1.2	1
Marly-Sandstone [RMA]	Firenzuola from Rovigo	40	A, (B)	yes	Ab, Ac (B0)	systematic drainage	blasting	2.5–5	5
Clays of Bacino del Mugello [aBM]	Firenzuola from S. Giorgio	2	C, (B)	yes	C1, C4R (B2)	systematic	ripping	1–1.2	1,3
Monte Morello [ScM]	Vaglia	30	A, (B, C)		Ac (B0)	sporadic	blasting	2.5–5	5

Table B.1

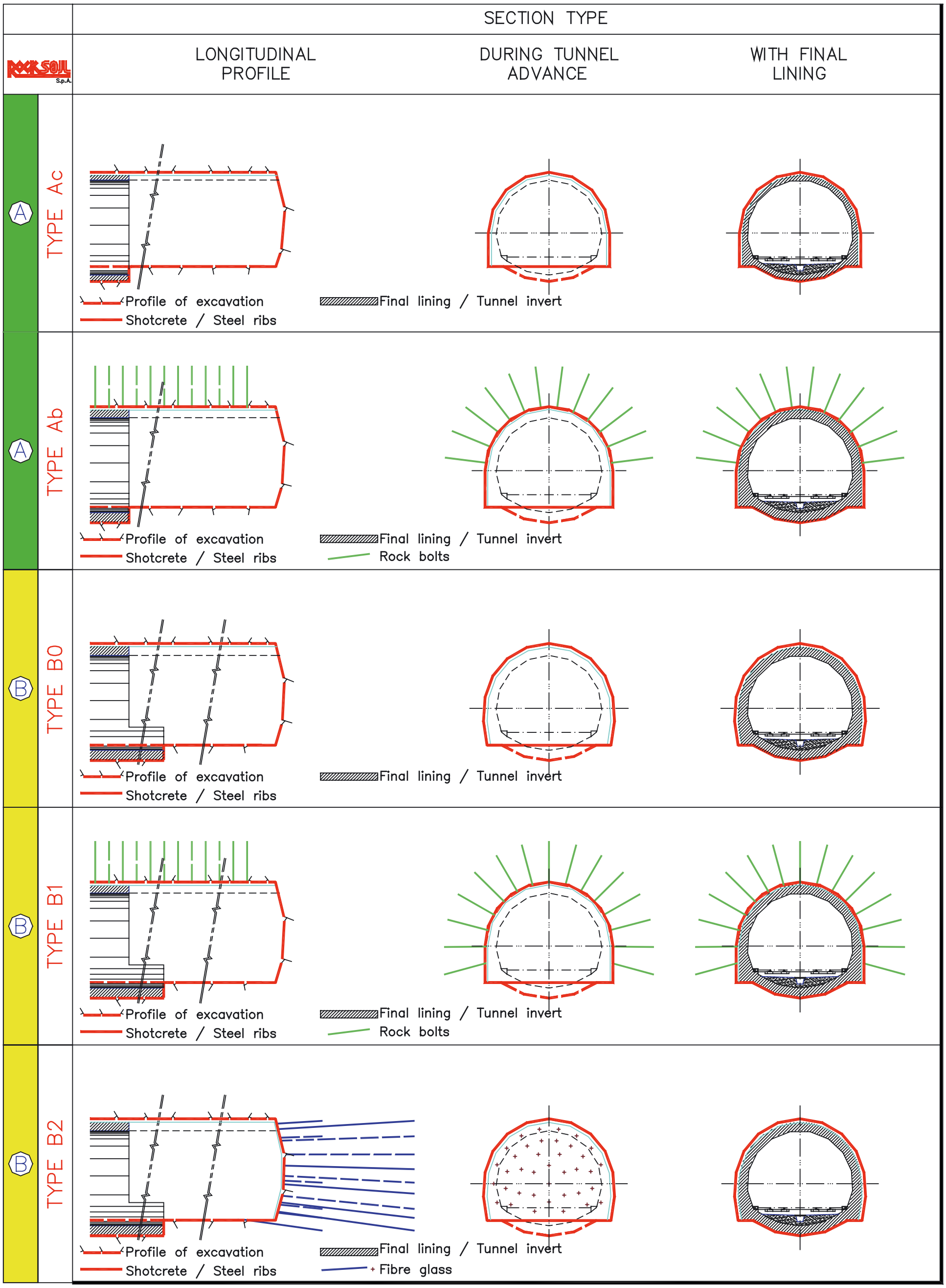
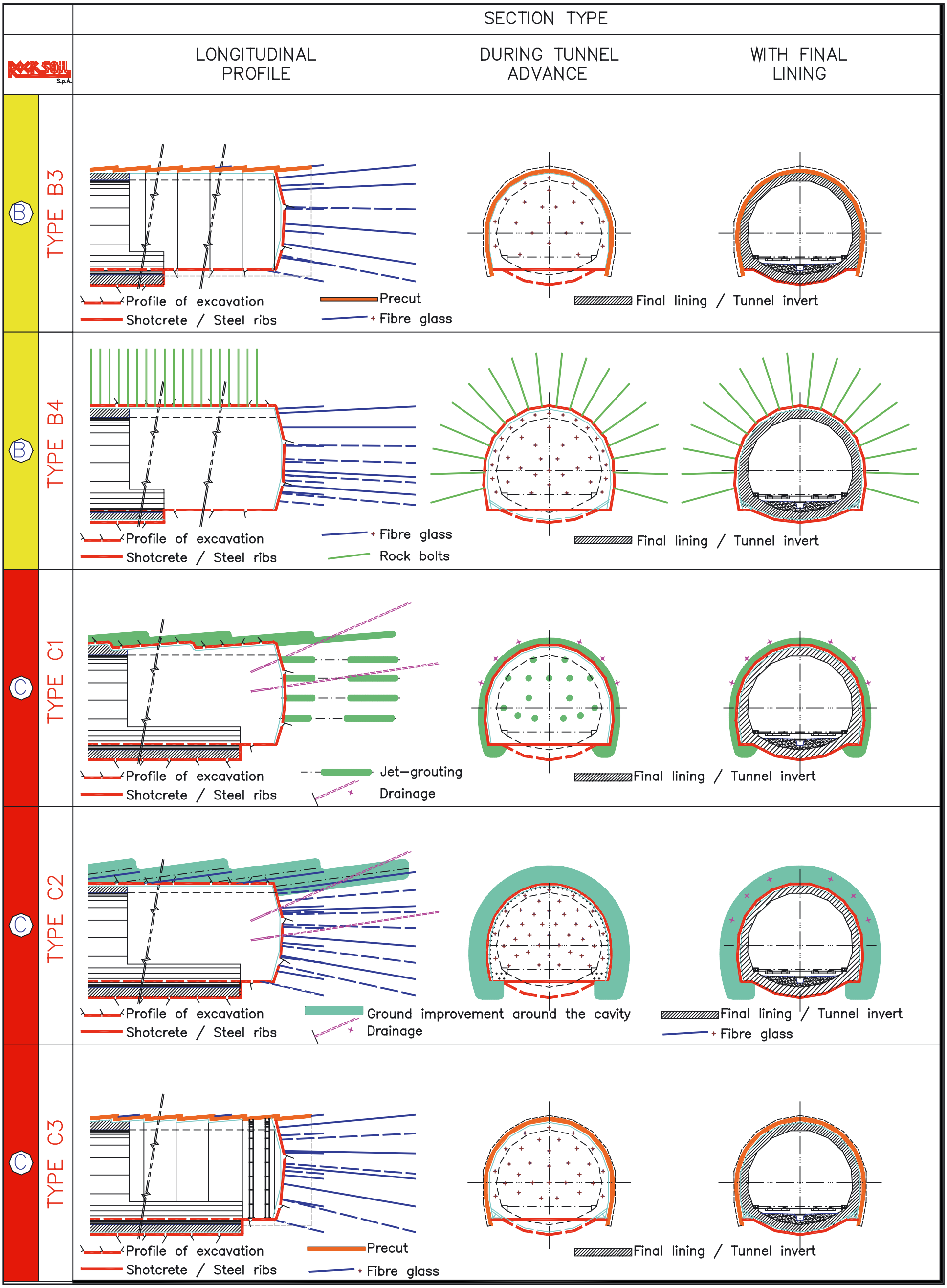
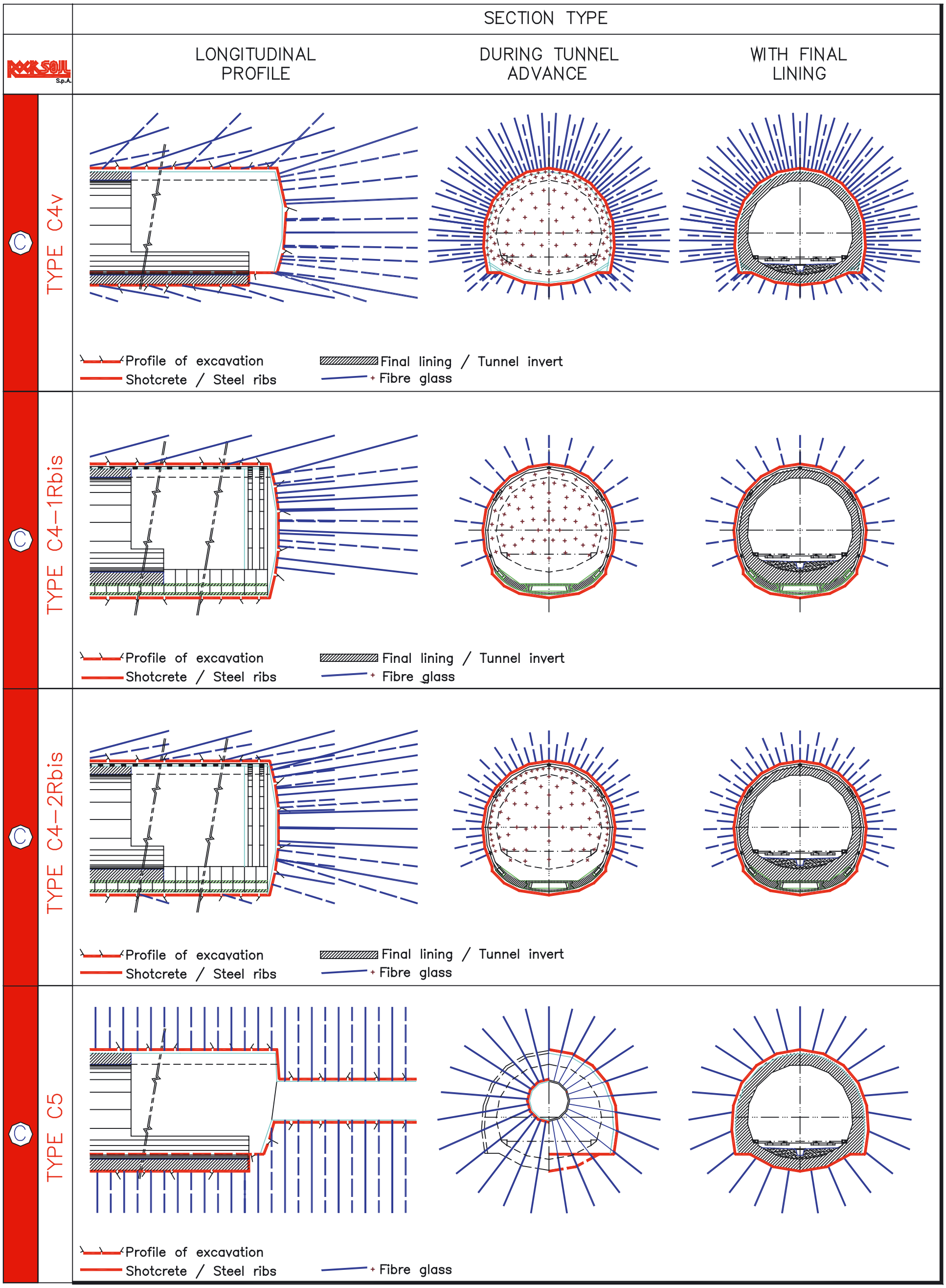


Fig. B.5 *Main tunnel section types*


SECTION TYPE
LONGITUDINAL PROFILE
DURING TUNNEL ADVANCE
WITH FINAL LINING
B TYPE B3
Profile of excavation
Shotcrete / Steel ribs
Precut
Fibre glass
Final lining / Tunnel invert
B TYPE B4
Profile of excavation
Shotcrete / Steel ribs
Fibre glass
Rock bolts
Final lining / Tunnel invert
C TYPE C1
Profile of excavation
Shotcrete / Steel ribs
Jet–grouting
Drainage
Final lining / Tunnel invert
C TYPE C2
Profile of excavation
Shotcrete / Steel ribs
Ground improvement around the cavity
Drainage
Final lining / Tunnel invert
Fibre glass
C TYPE C3
Profile of excavation
Shotcrete / Steel ribs
Precut
Fibre glass
Final lining / Tunnel invert


SECTION TYPE
LONGITUDINAL PROFILE
DURING TUNNEL ADVANCE
WITH FINAL LINING
TYPE C4v
Profile of excavation
Shotcrete / Steel ribs
Final lining / Tunnel invert
Fibre glass
TYPE C4–1Rbis
Profile of excavation
Shotcrete / Steel ribs
Final lining / Tunnel invert
Fibre glass
TYPE C4–2Rbis
Profile of excavation
Shotcrete / Steel ribs
Final lining / Tunnel invert
Fibre glass
TYPE C5
Profile of excavation
Shotcrete / Steel ribs
Fibre glass

Scheggianico Tunnel. South tunnel under the state road (ground: Marly-Sandy formation)

Castelvecchio access tunnel. The construction of 9 tunnels on the new high speed/capacity Bologna-Florence railway line required the construction of 14 access tunnels for a total length of 9,255 m

Firenzuola Tunnel. View of the works for the construction of the S. Pellegrino chamber at the North portal (ground: Marly-Sandstone formation)

Firenzuola Tunnel. Construction of the artificial tunnel of the S. Pellegrino chamber at the North portal (ground: Marly-Sandstone formation) conformities (i.e. differences between actual construction and design), which oblige partial redesign each time a change in conditions is encountered even if it involves only a minor change to the design, are avoided.

Each section type was analysed in relation to the loads mobilised by excavation as determined in the diagnosis phase, with regard to both the various construction phases and the final service phase by employing a series of calculations on plane and three dimensional finite element models in the elastic-plastic range.

Finally, precise specifications were formulated for the implementation of a proper monitoring campaign, which, according to the different types of ground tunnelled, would both guarantee the safety of tunnel advance, test the appropriateness of the design and allow it to be optimized in relation to the actual conditions encountered.

The philosophy which guided the design and subsequently the implementation of the monitoring system proposed can be summarised by the following basic concepts:

- *attention to detail*: each instrument must be selected and positioned to provide answers to a specific question;
- *monitoring "cause and effect"*: for the design engineer a change in an important parameter is governed by a law of cause and effect. It is only by measuring both phenomena that a relationship can be established between them and the necessary corrective action taken to remove the causes which resulted in undesired effect;
- *the principle of redundancy in measurements*: most measurements are of the highly localised type and as such can be influenced by the particular characteristics of the portion of the structure or rock in the immediate vicinity of the instrument.
  Single measurements may not therefore be representative of the phenomena on a larger scale. This problem can be solved by setting up measurement stations with a large number of points of measurements;
- *the principal of the plurality and the modularity of measurement systems and control parameters:* one single type of instrument may furnish information which is difficult to interpret or partial in determined contexts.

The use of instruments which measure different physical magnitudes which are correlated with each other may validate and/or complete the information on what is measured. Similarly, if the quantity and type of sensors can be adjusted in sections that are monitored according to how critical the situation encountered or predicted is, this guarantees appropriate dimensioning of the monitoring system and consequently also of the cost benefit ratio of the system itself. This last concept, together with those that precede it guarantees, amongst other things, a significant reduction in the subjectivity involved in the interpretation of monitoring phenomena and, secondly, allows

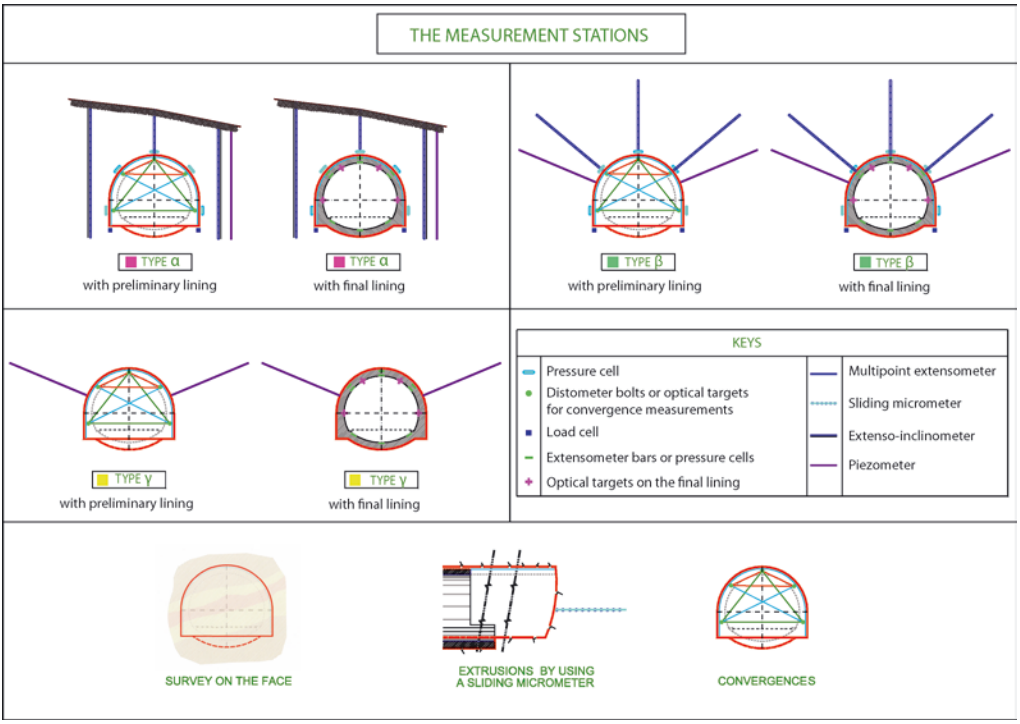


Fig. B.6

sufficient information to be acquired for statistical analysis to be performed. Geotechnical monitoring stations have been designed (Fig. B.6), which are based on these principles and on the parameters identified as the references for optimising the design during construction. They have been employed individually or in combination according to different criteria and frequency as a function of the three basic stress-strain behaviour categories (A, B and C).

Information on the implementation and use of monitoring is given later in this appendix in the section on monitoring during construction. However, before concluding the design stage and starting to discuss what happened in the construction phase, it should perhaps be stated that the detailed design illustrated here obviously also included the design of the necessary accessory works, such as portals, chambers, access and service tunnels which are not described here in order to avoid excessive detail.

Tunnel construction

Type of contract

The contract for the entire section of the railway between Bologna and Florence was awarded on a rigorous lump sum basis (€ 4.209 billion) by FIAT S.p.A., the general contractor, which accepted responsibility for all unforeseen events, including geological risks on the basis of the

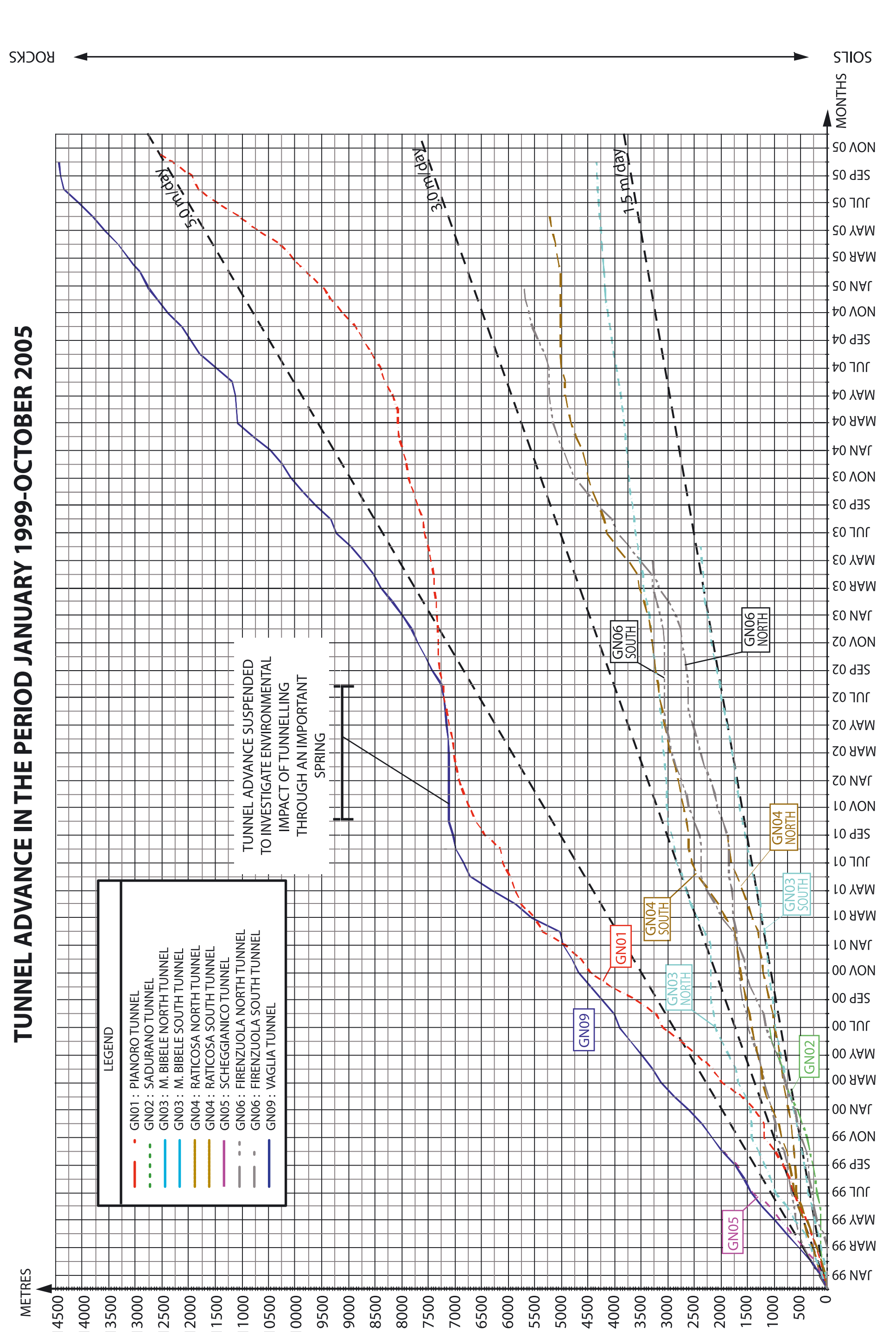


Fig. B.7 *Advance rates achieved under the different stress-strain conditions encountered*

Raticosa Tunnel. Reinforcement of the tunnel invert very close to the face (ground: scaly clays of the Complesso Caotico, overburden: ~550 m)

Raticosa Tunnel. Mobile formwork for casting the final lining in reinforced concrete detailed design as illustrated above. It subcontracted all the various activities out to the CAVET consortium (land expropriation, design, construction, testing, etc.).

Operational phase

Immediately after the contract was awarded, the construction design of works began at the same time as excavation work (July 1996).

Additional survey data and direct observation in the field generally confirmed the validity of the detailed design specifications, while the following minor refinements were made in the construction design phase:

- to deal specifically with particularly delicate stress-strain conditions, a steel strut was introduced as a variation to section types B2 and C4 in the tunnel invert to produce much more rapid containment of deformation. This modification of the B2 section type was found to be much more versatile and appropriate even for many situations where the core-face was unstable. Use of the heavier C section types was thus limited to the more extreme stress-strain conditions;
- the effectiveness of core-face ground improvement using fibre glass structural elements was increased considerably by introducing an expansive cement mix to grout the fibre glass nails;
- as a consequence of positive results acquired during the construction of tunnels on the Rome-Naples section of the same railway line, a section type B2pr was designed for underground construction of the Sadurano, Borgo Rinzelli and Morticine tunnels originally designed as artificial tunnels;
- finally it was decided to replace section types B3 and C3 which involved the use of mechanical pre-cutting with section C2 (ground improvement in the core-face and around it with fibre-glass structural elements) better suited to the ground to be tunnelled.

The final result of construction design was the definition of the following percentages of tunnel section type:

- section type A: 20.5%;
- section type B: 57.5%;
- section type C: 22.1%.

Section type	Forecast advance rates [m per day of finished tunnel]	Actual advance rates [m per day of finished tunnel]
A	5.40	5 – 6
B0	4.30	5 – 5,5
B2	2.25	2.10 – 2.2
C1	1.40	1.40
C2	1.25	0.85
C4V	1.25	1.63

Table B.II

Approximately 73 km of running tunnel, 100% of the total, have been driven and almost all of it has been lined. Average monthly advance rates of finished tunnel were around 1,000 m of finished tunnel with peaks of 2,000 m reached in March 2001, working simultaneously on 30 faces.

Figure B.7 gives the advance rates for the different tunnels constructed. It can be seen that not only were the advance rates very high in relation to the type of ground tunnelled, but above all they were very constant, an indicator of the excellent match between the design specifications and the actual reality. Even for the Raticosa tunnel, driven under extremely difficult stress-strain conditions in the *Complesso Caotico* formation consisting of the much feared scaley clays, average advance rates were never less than 1.5 m per day. Table B.II gives a comparison of daily advance rates forecast by the detailed design specifications for some section types and the actual advance rates achieved.

Table B.III on the other hand gives the distribution of differences in section types between the detailed design specifications and the tunnel as built. Even if account is taken of the increased rigidity introduced to the B2 section type, which made it possible to employ the type in many situations typical of behaviour category C, there was nevertheless a significant reduction in the use of the more costly section types in favour of more economical solutions. This result was to a large extent due to the exceptional effectiveness of the preconfinement methods employed even under large overburdens. It was the first time they had been used with overburdens greater than 500 m.

Some consider making the core-face more rigid before tunnel advance as counter productive under large overburdens. However, if it is properly performed and the continuity of the action from ahead of the face back down into the tunnel is ensured with the placing of a steel strut in the tunnel invert, it is, as was demonstrated, extremely effective even with large overburdens and resort to the heavier section types was only necessary in the most extreme situations.

Another reason for the greater use of section type A was that there are only minor differences between it and the B0 sections types such as the thickness of the final lining or the distance from the face at which the tunnel invert is cast.

As a consequence, section type A was adopted in place of section types B0 or B0V, whenever the ground conditions allowed this to be done without taking greater risks (e.g. in long sections of the Vaglia tunnel where the presence of the adjacent service tunnel already built made the situation particularly clear). The differences found between the detailed design specifications and the tunnel as built did not reveal any sensational discrepancies, neither in terms of the overall cost of the works, which was a little less than budgeted under the detailed design specifications (~−5%), nor with regard to construction times. While the contractor will benefit from the lower cost, a reward for the greater risk run by agreeing to sign a rigorously lump sum, all-in contract, the client and citizens will benefit from the punctual observance of time schedules because they will be able to use the new transport services without intolerable delays.

The monitoring and calibration of the design during construction (monitoring phase)

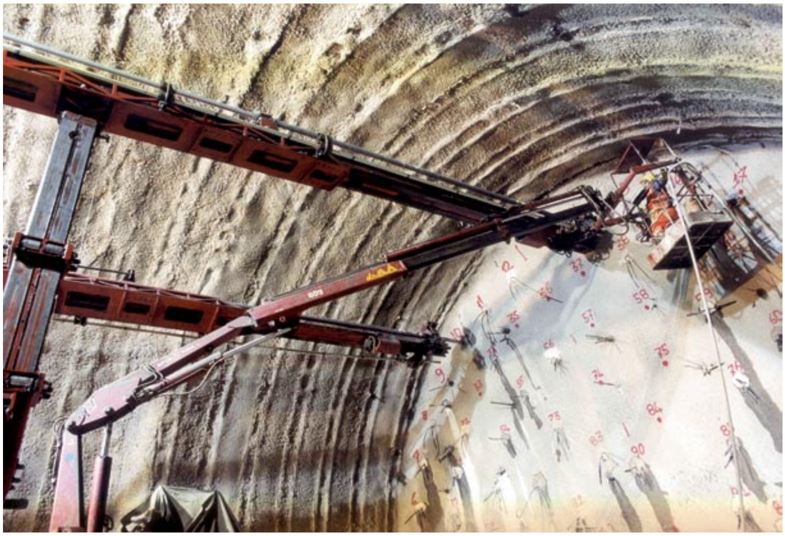
The particular nature and the importance of the project required a thorough monitoring programme both during construction and for the completed tunnel in service.

HIGH SPEED/CAPACITY TRAIN - Milan to Naples Line - SECTION TYPES DISTRIBUTION																												
	DETAILED DESIGN														AS BUILT													
TUNNEL	Length [m]	SECTION TYPES DISTRIBUTION [m]													Length [m]	Tunnelled length		SECTION TYPES DISTRIBUTION [m]										
		A	B0	B0V	B1	B2	B3	B4	C1	C2	C3	C4	C4V	C5		[%]	[m]	A	B0	B0V	B2	B2pr	B2V	C1	C2	C4	C4V	C6
Pianoro	10293.4		951.8			3886.4	3036		62.0	948.8	1083	310		15.5	10710	100.0	10710.0		8167.0		682.5		63,0	3,5			1794.3	
Sadurano	3778.0	64.0	2580.8			875.0			68.0	190.3					3767.0	100.0	3767.0	2213.1	1408.0		85	53			8			
M. Bibele	9118.5	978.2	1094.6		4529.1	1212.2			76.0	1112.8		115.6			9101.0	100.0	9101.1	2935.5	2015.7		3965.8		97,9	41	45			
Raticosa	10381.0	3043.0			972.2	758.4			40.0	786.7		4465.1		315.67	10367.0	100.0	10367.2	3680.0	786.0		857		25	85		1468	673.4	2792.6
Scheggianico	3530.6	2089.9			1404.7					36.0					3535.0	100.0	3535.0	3517.0	18.0									
Firenzuola	14311.5	3528.7			5950.4	716	412.2		227.5	511.9	2226.8	738.1			15211.0	100.0	15211.0	6833.1	3556.3		3081.8		263.5	577.1	9		125.83	43.46
B. Rinzelli	455.0								160		295				528.5	100.0	528.5					303.5			225.1			
Morticine	273.7								80	193.7					565.5	100.0	565.5					537.0			28.5			
Vaglia	16757.0	2017.2	3104.3	1129.8	5629.0			1151.2	692	708.5			2325.2		16757.0	100.0	16757.7	5287.60	9299.50	96.60	1547.0		296.70	128.25	101.4			
TOTAL LENGTH [m]	68898.6	11721.0	7731.5	1129.8	18485.2	7447.9	3448.2	1151.2	1405.4	4488.6	3604.8	5628.8	2325.2	331.2	70542.3	100.0	70542.4	24466.2	25250.5	96.6	10219.1	893.5	746.1	834.8	417.0	1468.0	2593.5	2836.1

	A	B0	B0V	B1	B2	B3	B4	C1	C2	C3	C4	C4V	C5
SECTION TYPES DISTRIBUTION [%]	17.0	11.2	1.6	26.8	10.8	5.0	1.7	2.0	6.5	5.2	8.2	3.4	0.5

A	B0	B0V	B2	B2pr	B2V	C1	C2	C4	C4V	C6
34.7	35.7	0.1	14.5	1.3	1.1	1.2	0.6	2.1	3.7	4.0

Table B.III *Percentage of section types used: comparison between the detailed design and as built*



Firenzuola Tunnel: cementation of the fibre glass reinforcement in the core-face

Borgo Rinzelli Tunnel: effect of stabilisation using artificial ground overburden (ground: Clays of the Mugello Basin, overburden: zero)

The following was monitored during construction:

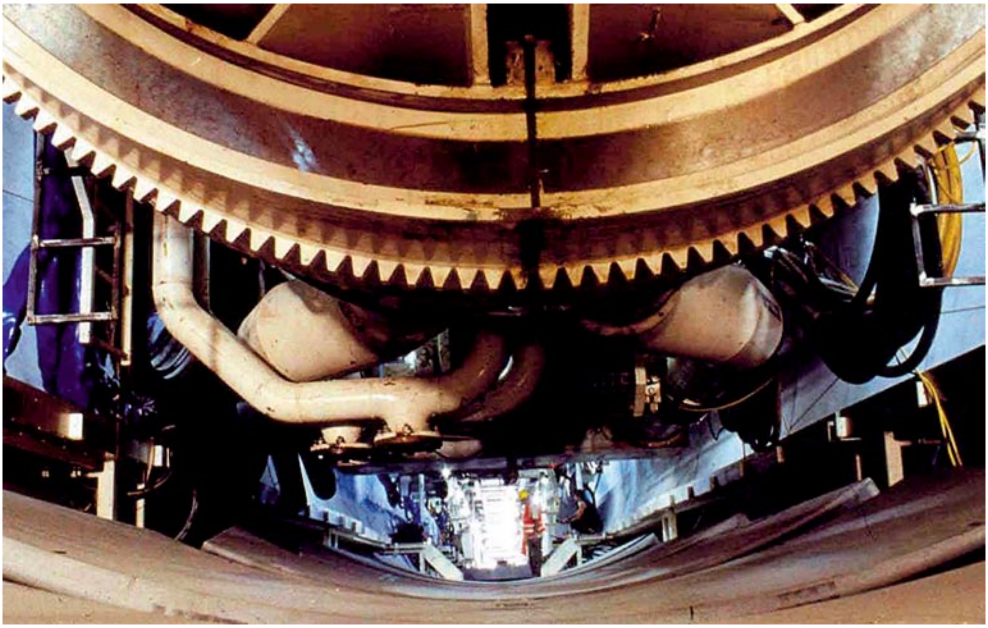
- the tunnel faces by means of systematic geomechanical surveying of the ground at the face. The surveys were conducted to I.S.R.M. (International Society of Rock Mechanic) standards and gave an initial indication of the characteristics of the ground to be compared with design predictions;
- the deformation behaviour of the core-face by measuring both surface and deep core-face extrusion, with measurements taken as a function of the different behaviour categories. Systematic measurements of this kind under difficult stress-strain conditions are crucial: in fact as already stated in chapter 10 on the monitoring phase, monitoring convergence alone, which is the last stage of the deformation process, is not sufficient to prevent tunnel collapse under these conditions. Extrusion, on the other hand, is the first stage in the deformation process and if it is kept properly under control will allow time for effective action to be taken;
- the deformation behaviour of the cavity by means of systematic convergence measurements;
- the stress behaviour of the ground-lining system by placing pressure cells at the ground-lining interface and inside the linings themselves, both preliminary and final.

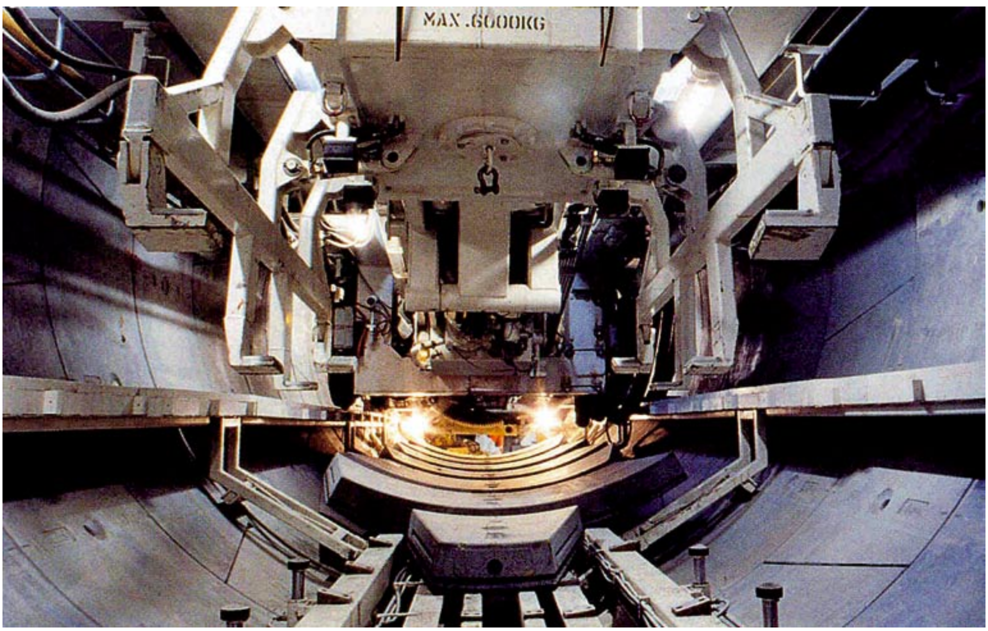
The results of monitoring activity guided the design engineer and the project management in deciding whether to continue with the specified section type or to modify it according to the criteria already indicated in the design, by adopting the 'variabilities' contained in it. Obviously, in the presence of particular conditions that were not detected at the survey phase and therefore not provided for by the design, it is always possible to design a new section type. This method of proceeding allowed the uncertainty connected with underground works to be managed satisfactorily even with a rigorously lump sum contract like that between FIAT and TAV.

Most of the instrumentation already used during construction, which is connected to automatic data acquisition systems, will continue to be employed for monitoring when the tunnel is in service. The automatic data acquisition system can be interrogated at any moment in the life of the works to obtain data and verify the real behaviour of the tunnels and compare it with design predictions.

The construction of the Ginori service tunnel for the Vaglia running tunnel

The construction of the Ginori service tunnel for the Vaglia running tunnel deserves particular attention.This tunnel is 9,259 km in length with an internal diameter of 5.6 m and runs parallel to the "Vaglia" running tunnel for 6,501 metres to which it is connected by small pedestrian tunnels every 250 m (Fig. B.8). The first section of 1,588 m and the final section of 1,170 m run down and up respectively with a gradient of 1.8%. As already mentioned, fully mechanised technology was employed to drive this tunnel. This decision was made on the basis of in-depth study of the geological, geomechanical and hydrogeological conditions of the ground and it fully satisfied a series of contingent needs and requirements:





Ginori service tunnel: two views from inside the TBM. Note the conveyor line for the prefabricated concrete segments of the final lining below

- the environmental need to drive the tunnel from one single portal near Ginori;
- the need to control interference with ground water as much as possible;
- finally the requirement to complete the tunnel rapidly.

Given the success already achieved with the design and construction of the running tunnels across the Apennines, important innovations were also introduced to the design for mechanised tunnel construction. A series of criteria were developed and introduced in the design (and as a consequence in the contract) to follow during construction consisting of design and/or operational measures to be applied if 'variabilities' (differences between predicted and actual conditions) were encountered during tunnel advance.

The design of tunnel advance by TBM

The ground through which the "Ginori" tunnel passes all lies within the *Monte Morello* formation. It is certainly a complex formation with mechanical characteristics which vary greatly, ranging from argillites to compact limestones. Consequently the designer of the

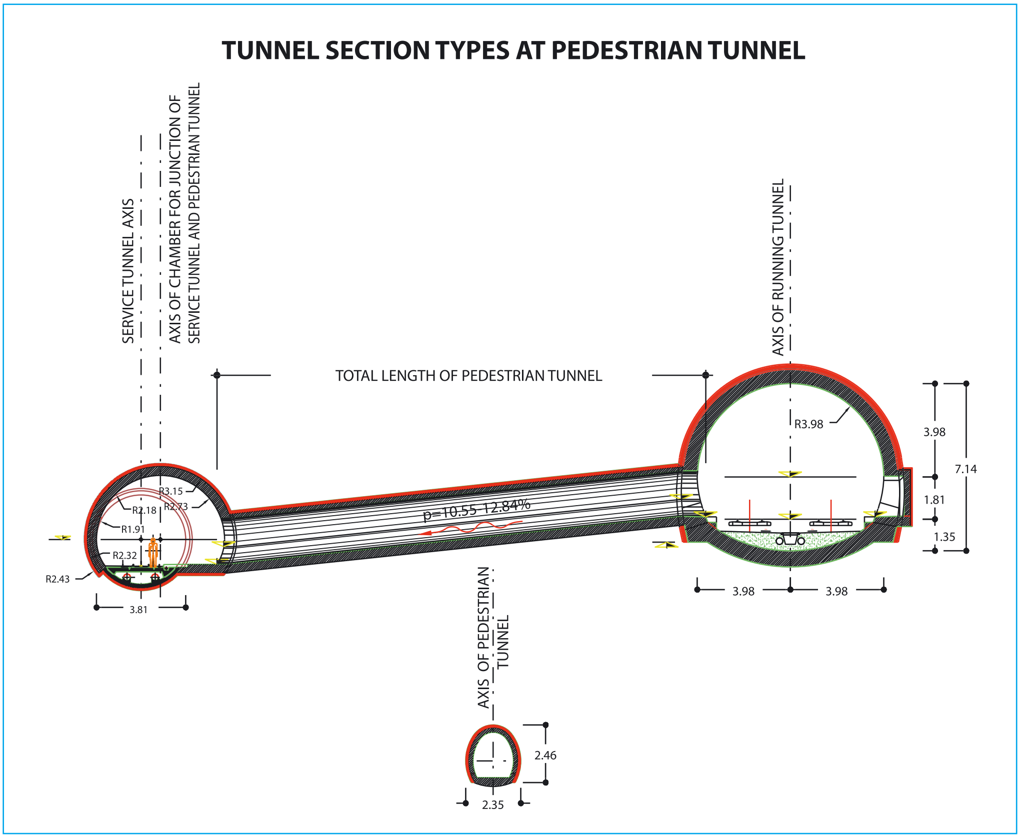
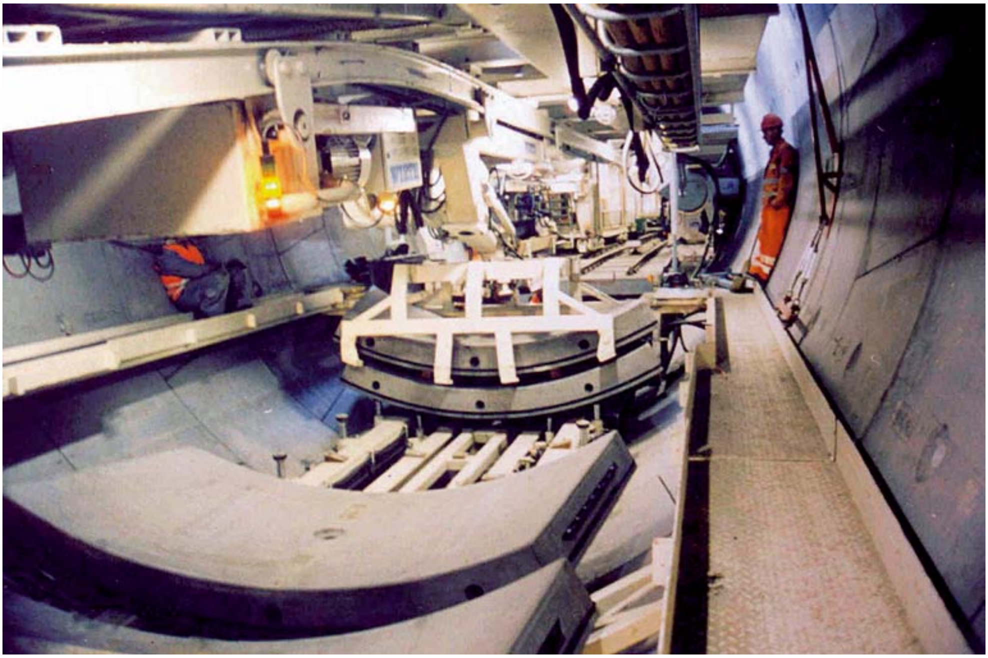


Fig. B.8 *Typical cross section of the Ginori service tunnel at the junction with a pedestrian tunnel connecting with the Vaglia tunnel*



Ginori service tunnel: The mechanism for lifting prefabricated concrete segments for the final lining

Ginori service tunnel: the “tail” of the TBM inside the tunnel already lined

TBM received considerable input from the consulting civil engineer and the result was an innovative machine fitted with advanced geological surveying systems, capable of adjusting the method of advance according to the different geomechanical conditions it encounters and, where necessary, of performing advance reinforcement both in the core and around it (Fig. B.9). The ways in which it acquires cutter operating parameters and the criteria it employs to change the type of tunnel advance and to prepare the ground ahead are explained in a document appended to the design specifications entitled *Guide Lines for performing surveys in advance, for selecting the appropriate operating methods and performing ground reinforcement operations*. This document clearly defines, right at the design stage, the criteria that the TBM operator and/or design engineer must adopt to:

- confirm the current operating method;
- perform extra surveys to obtain a better understanding of the nature of the rock ahead of the face;
- change the operating method when different ground conditions are encountered;
- perform ground reinforcement operations in the face and around the tunnel that the machine is capable of doing.

To achieve it was important for the *guide lines* to give the main parameters to be followed systematically during construction with precise descriptions with quantities. These included the following:

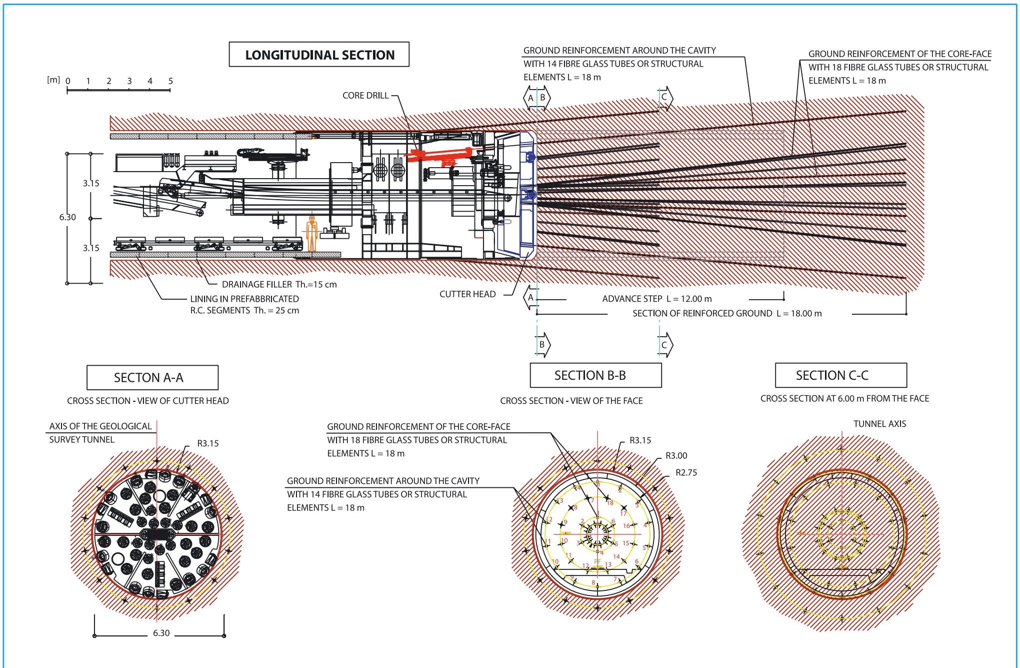


Figura B.9 *Intervention to improve the ground that can be performed from the TBM*

Ginori service tunnel: view of the head of the TBM during insertion in the tunnel

Depot for the prefabricated concrete segments for the final lining on the construction site of the Ginori service tunnel driven by TBM

- the TBM advance parameters (advance velocity, thrust, specific energy);
- the geological and geomechanical parameters acquired from systematic seismic surveying performed continuously in advance. This information may be added to by georadar tests in probe holes drilled into the face (Fig. B.10).

These parameters are useful for continuous and systematic monitoring of the characteristics of the ground and its response to excavation. They are associated with value ranges for normal functioning of the TBM. If the values actually measured differ substantially from those predicted as a result of particular unexpected local geomechanical conditions (tectonised zones, sudden lithological and structural changes, etc.), the TBM operator and/or the design engineer must take appropriate corrective or additional action as specified in the *guide lines*. Specified action may include operational changes (e.g. overbreak by the cutter head, locking the telescope mechanism, activating the tail pistons, etc.) extra surveys (e.g. core drilling) or ground reinforcement design operations. If the conditions encountered do not correspond to any of those specified in the design, then other solutions must be considered (Fig. B.11).

With this method of proceeding, action taken in a completely arbitrary manner based on no particular rules, which might have a marked effect on the stress-strain response of the rock mass, is avoided. At the same time a guide is provided for adapting to ordinary discrepancies between design predictions and the reality not of an emergency nature, with action taken based on standard design procedures.

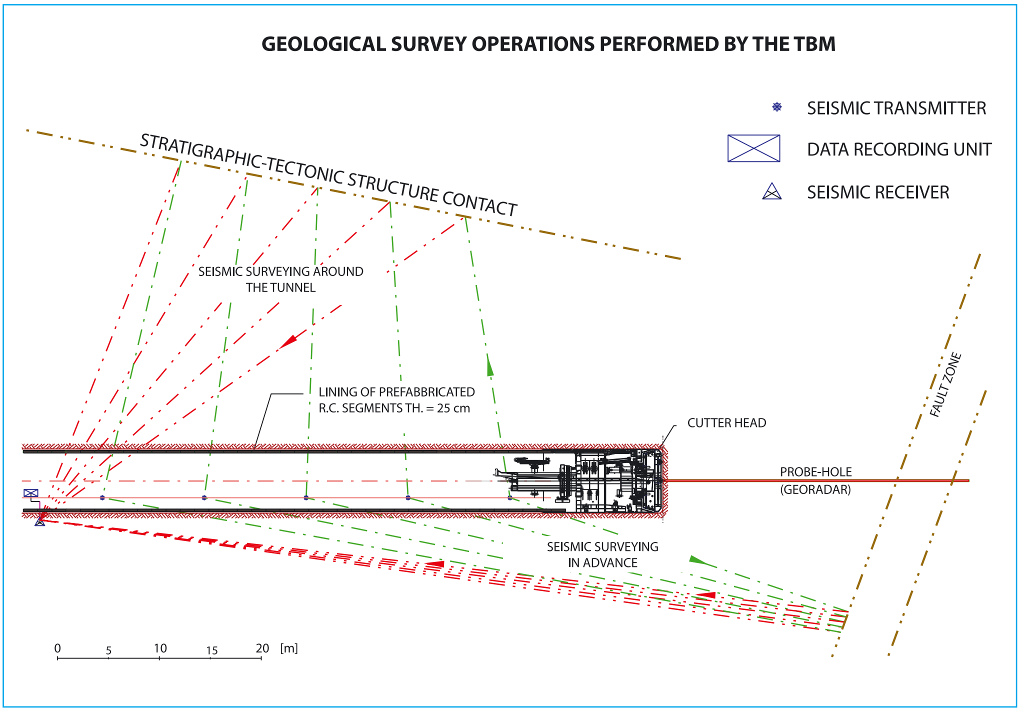


Fig. B.10 *Geological surveys performed by the TBM*

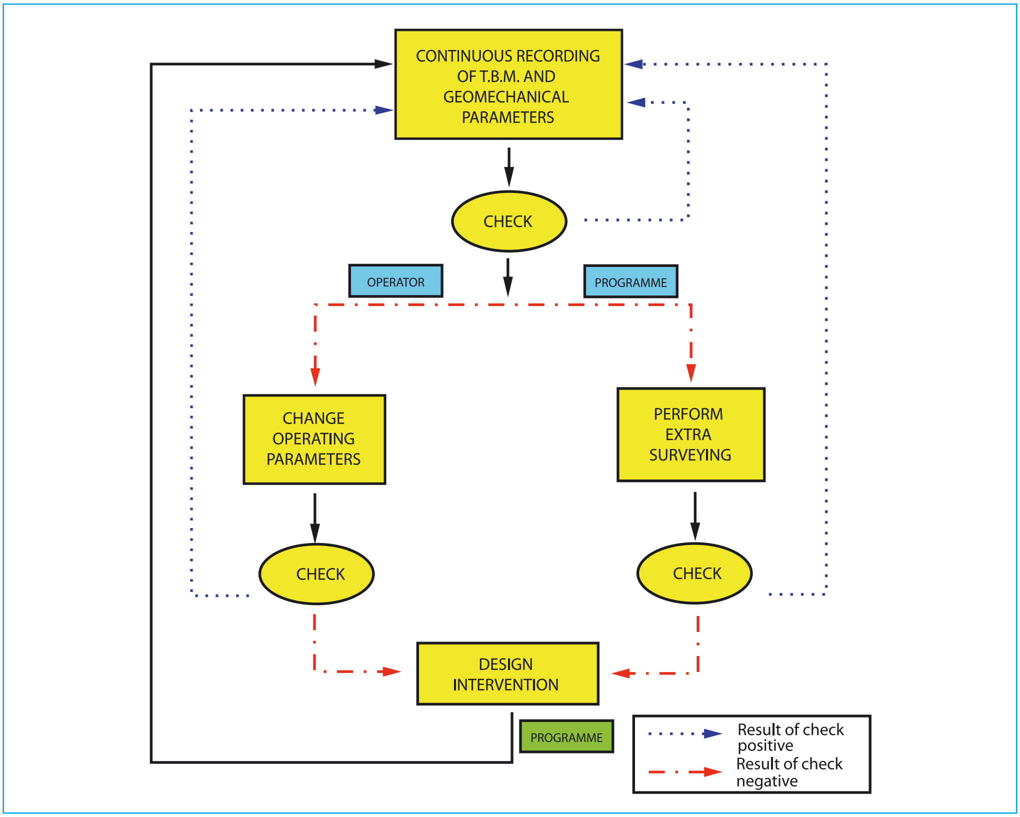


Fig. B.11 *Flow diagram for design control by TBM*

Tunnel construction

The TBM used for the excavation of the tunnel and its lining is a Wirth TB 630 E/TS, double shielded telescopic rock cutter with a diameter of 6.3 m capable of generating a maximum thrust of 30,000 kN and an excavation torque of 5 kNm. As has been said, the distinguishing feature of the machine is that it was fitted with drilling equipment inside the shield capable of drilling holes in advance through special apertures in the cutter head and on the steel shell for the purpose of surveying or improving and reinforcing the ground (in accordance with the principles of the ADECO-RS approach). The spoil removal system consisted of conveyor belts. A special rack and pinion traction system supplied the TBM with the material required for excavation and tunnel lining activity.

The lining itself consisted of rings of prefabricated r.c. segments 140 cm in length and 25 cm thick, designed to withstand a hydraulic pressure of up to 5 bar. The rings, composed of six segments plus a key, had a special diagonally truncated cone shape, which made it easy to follow the theoretical alignment of the tunnel with excellent precision by simple fitting the segments for each ring in the sequence appropriate for each occasion (Fig. B.12). Mechanised excavation of the Ginori tunnel was completed without any serious problems at an average advance rate of approximately 20 m/day of finished tunnel on schedule and to budget.

Conclusions

The experience acquired in the construction on time and to budget of this project to cross the Apennines with a new high speed/capacity railway line, which was exceptional in terms of its size and the heterogeneity and difficulty of the ground, demonstrates beyond any doubt that, by using new technologies in an integrated manner, perfectly consistent with its underlying principles, the ADECO-RS approach opens up exciting new prospects for tunnelling, finally making industrialisation possible with the consequent certainty over construction times and costs previously not possible.

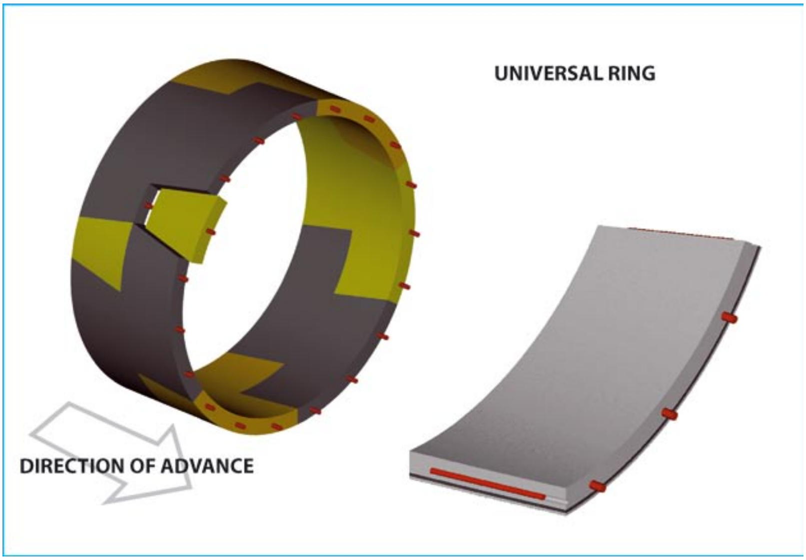


Fig. B.12

The Tartaiguille tunnel: the core-face reinforced with fibre glass elements, 1997, ground: clay, max. overburden: ~ 110 m

The Tartaiguille tunnel

The Tartaiguille tunnel, 2,330 m in length with a cross section of 180 m^2 is one of six tunnels on the TGV Méditerranée Line that connects Lyon to Marseilles on a route of approximately 250 km (Fig. C.1). The route passes through several Cretaceous formations which, from North to South, are as follows:

- upper Stampiem calcareous, fractured to a varying extent;
- lower Stampien marly clays;
- alternating Aptien blue marls and Albian sandstones.

The tunnel project described here passes through the lower Stampien formation consisting of alternating strata of marls, clays and silts with some calcareous material. The thickness of each strata varies from a few centimetres to some metres with the main discontinuities consisting of the stratification itself.

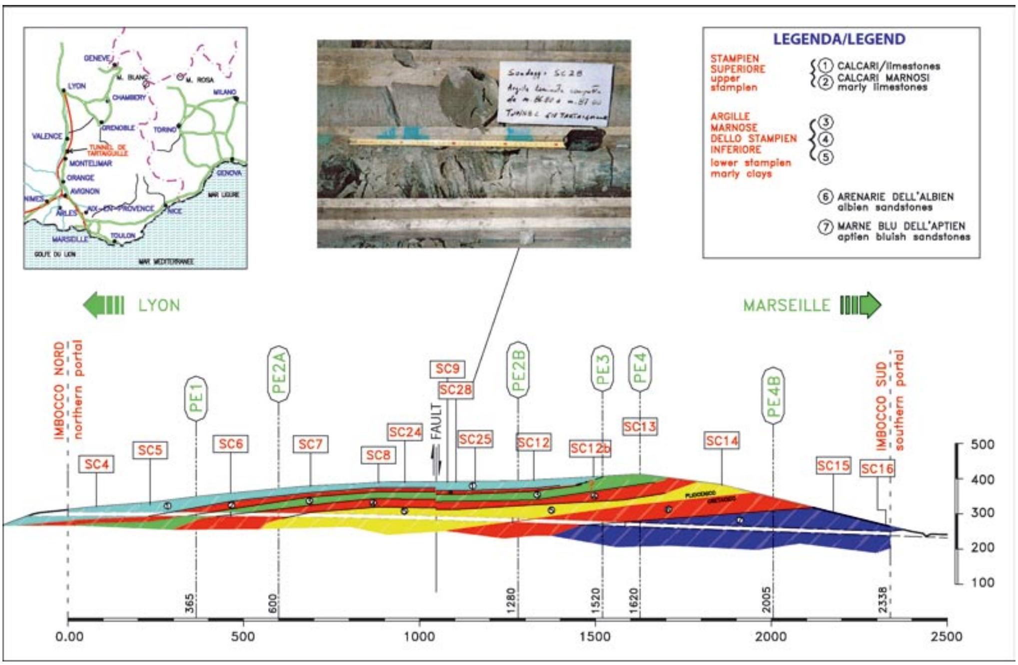


Fig. C.1 *Chorography and longitudinal geological profile of the tunnel*

This formation is extremely sensitive to the presence of water, which on contact causes a very rapid change in the strength properties of the ground. There is an immediate loss of consistency and swelling is triggered due to the high percentage of swelling material present in the clayey component (75% montmorillonite).

The maximum overburden for the whole tunnel is 137 m while that for the marly clay section is 106 m.

A brief history of the excavation

Excavation began from both portals (North and South) in February 1996.

The system of advance employed was to drive a top heading with a roadheader and bench excavation approximately 200 m behind using an excavator hammer.

The cross section types specified by the design for the long and short term stabilisation of the tunnel were of differing strength according to the geological characteristics of the ground that were encountered. They all consisted of shotcrete and steel ribs and end anchored radial rock bolts 4 m in length.

The design for crossing the 900 m stretch in the Stampien clays specified a cross section type consisting of a 25 cm layer of shotcrete, reinforced with HEB 240 ribs placed at 1.5 m intervals and micropiles beneath the base of the ribs and rock bolts after excavation of the bench (L = 4 m, one every 3 – 4 m^2) followed by casting of the sidewalls and the invert. When safety conditions required it the ground ahead of the face was reinforced with tubular fibre glass nails to improve stability. Final stabilisation of the tunnel was to be then obtained by casting a concrete lining in the crown (thickness 70 cm) after placing waterproofing.

At the end of September 1996, convergence at the South face increased well beyond predicted design values during excavation of the Aptien marls section and reached 60 mm during excavation of the top heading and 150 mm during bench excavation. The increase in convergence values manifested in the form of sizeable cracks in the shotcrete which also spalled. This not only made considerable barring down work necessary, but also steel mesh reinforcement had to be placed in the roof arch and double the number of rock bolts initially planned.

As a result of these events and worried above all by the imminent crossing of the marly clays, the site operator decided to perform a further geological survey to check the geomechanical parameters that had been assumed at the design stage.

The most important factor to emerge from the survey was the discovery of a thrust at rest coefficient K_o that was very much higher than assumed at the design stage. While the new calculations performed with correct parameters explained the convergence observed in the marls, it was clear that it would be impossible to use the planned cross section type since it would have involved accepting inadmissible deformation.

In order to find the best possible solution to the problem, the SNCF (*Societè National du Chemin de Fer*) set up a study group (*"Comité de pilotage"*), at the beginning of 1996, consisting of its own experts from French Rail, experts from the consortium formed by the companies Quillery & Bard and from their consultants Coyne et Bellier, of the geotechnical consultants of the Terrasol and Simecsol Consortium and finally of experts from the CETU. This study group then turned to major European tunnelling experts, inviting them to put forward their design proposals for crossing the clayey section of the tunnel in safety and on time.

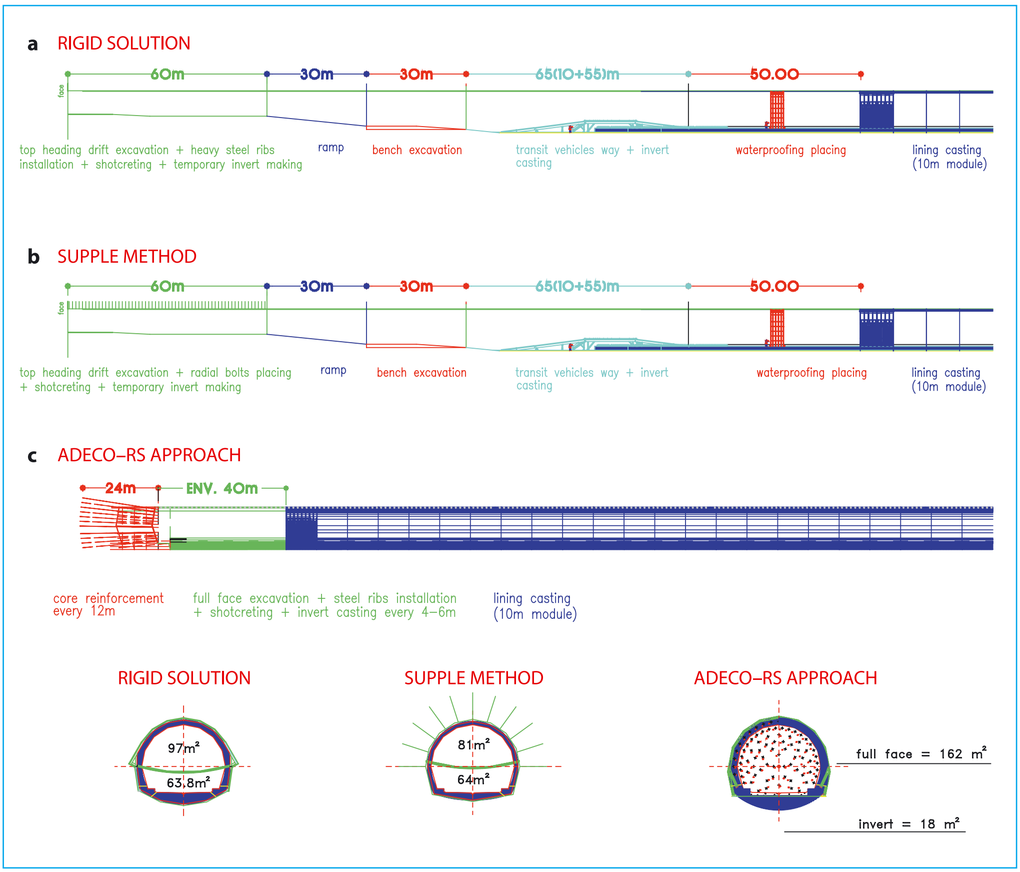


Fig. C.2 *The three solutions examined (French, Swiss, Italian)*

The first proposal (French) (Fig. C.2a) suggested half face advance with a much more rigid cross section type than was specified in the initial design, created by increasing the number of radial rock bolts placed around the excavation and by placing a stronger preliminary lining, thicker and with more reinforcement. However, this would have required very lengthy execution times and was immediately discarded.

The second proposal (Swiss) (Fig. C.2b) considered by the study group involved the use of special deformable ribs for the top heading. This would have allowed the rock mass to free itself of part of its potential energy through the development of unopposed and also considerable deformation. Reinforcement of the face using fibre glass tubes of reduced length was specified to guarantee the safety of site workers. This proposal was also discarded because of the numerous uncertainties connected with the difficulty in calculating the behaviour of the deformable ribs and due to the long time required for this type of advance (the final lining could not be cast until the strong convergence allowed by the deformable ribs had stabilised).

Before resigning itself to accepting long delays before the new line could be opened, the SNCF invited the author to give his opinion on the feasibility, and construction times and costs and to send in his own proposal. They had in fact heard of the major successes achieved in Italy in driving tunnels under very difficult conditions, by applying new design and construction principles. Consequently a third solution (Italian) (Fig. C.2c) was considered, radically different from those already described. This involved full face advance and was based on controlling deformation phenomena by, as we shall see, stiffening the advance core (ADECO-RS approach [41], [42]).

SNCF were favourably impressed by the preliminary designs presented and the construction times that were forecast and guaranteed by the consulting engineer on the basis of documentation produced for similar cases resolved using the same system. Consequently, in March 1997, the SNCF selected Rocksoil to design the remaining 860 m of tunnel to be driven in the marly clays of the Stampien.

The survey stage

The first stage in the design of the section of tunnel running through the Stampien marly clays consisted of an in-depth study of the considerable geological and geotechnical information already available when Rocksoil intervened.

Numerous laboratory (CD, CU and UU triaxial compression tests, edometer tests, tests for physical and chemical characterisation, swell tests, spectrophotometric tests) and *in situ* tests (dilatancy tests, “push-in” pressuremeter tests, plate load tests, direct shear tests) had been performed and the geomechanical characteristics of the ground had virtually been completely defined (Fig. C.3). The parameters to be used in the calculations and the principle geotechnical hypotheses to be assumed were decided jointly during a number of meetings of the study group in which consulting engineer participated directly. They are given below:

- γ = unit weight = 21.7 KN/m^3
- c_u = undrained cohesion = 1.2 MPa
- c' = drained cohesion = 0.2 MPa
- φ' = angle of friction = 27°
- E_u = undrained elastic modulus = 400 MPa
- E = drained elastic modulus = 200 MPa
- ν = Poisson modulus = 0.4
- K_0 = thrust coefficient at rest = 1.2
- p_g = swelling pressure = 0.2–0.3 MPa (below the tunnel invert)
- *fluage* of the ground: simulated as a reduction of 35% of the drained elastic modulus on a thickness equal to one radius of excavation (7.5 m) around the cavity;
- water table: approximately 25 m above the crown of the tunnel.

When the available geotechnical and geological information had been examined, the consulting engineer asked for four triaxial cell extrusion tests to be performed in order to study the response of the ground to excavation by simulating the release of stress produced in the ground by tunnel advance in the laboratory.

In this type of test a sample of the ground is inserted in a triaxial cell and put under pressure until the natural stress state of the rock mass is reached.

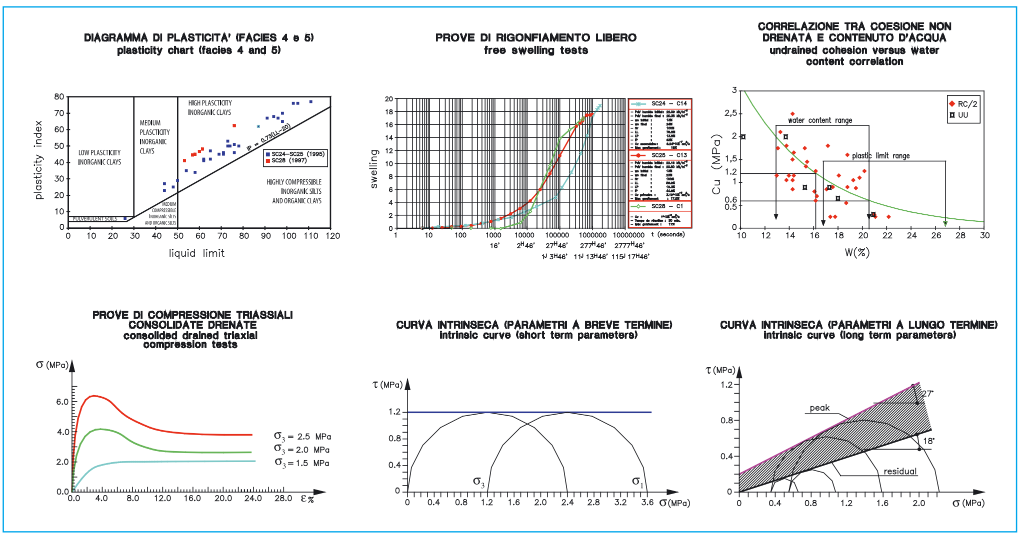


Fig. C.3 *Results of the tests performed in the laboratory on the Stampien clays*

By using a fluid under pressure, this stress state is also reproduced in a special cylindrical volume (which simulates the advance of a tunnel face) created inside the test sample and coaxial to it before the test. The stress state around the sample is maintained and the pressure of the fluid in the cylindrical volume is gradually reduced to simulate the changes in the stress state at the face induced by excavation. A prediction of the magnitude of extrusion at the face as a function of time is thereby obtained.

The use of these tests is fundamental for putting the finishing touches on the design and defining tunnel advance stages.

The diagnosis stage

In the diagnosis stage, the results of the swelling and extrusion tests (Figs. C3 and C4) and the characteristic lines at and at a distance from the face (Fig. C.5) were studied.

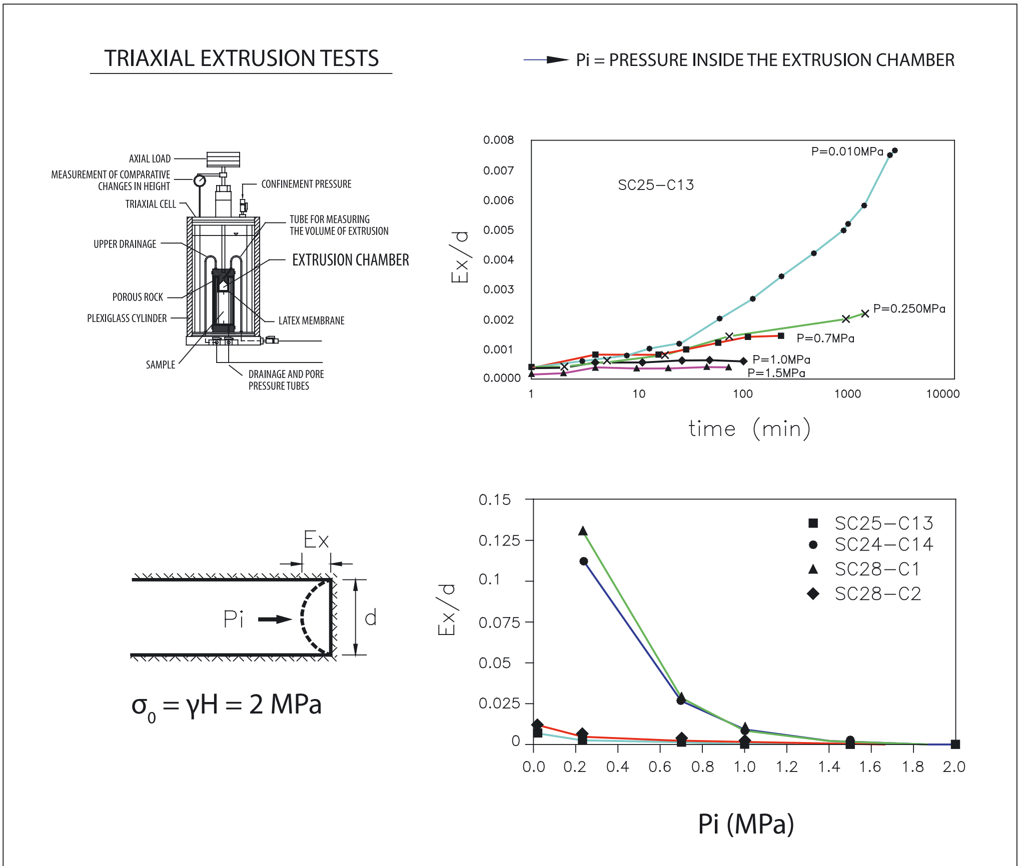


Fig. C.4 *Triaxial cell extrusion tests*

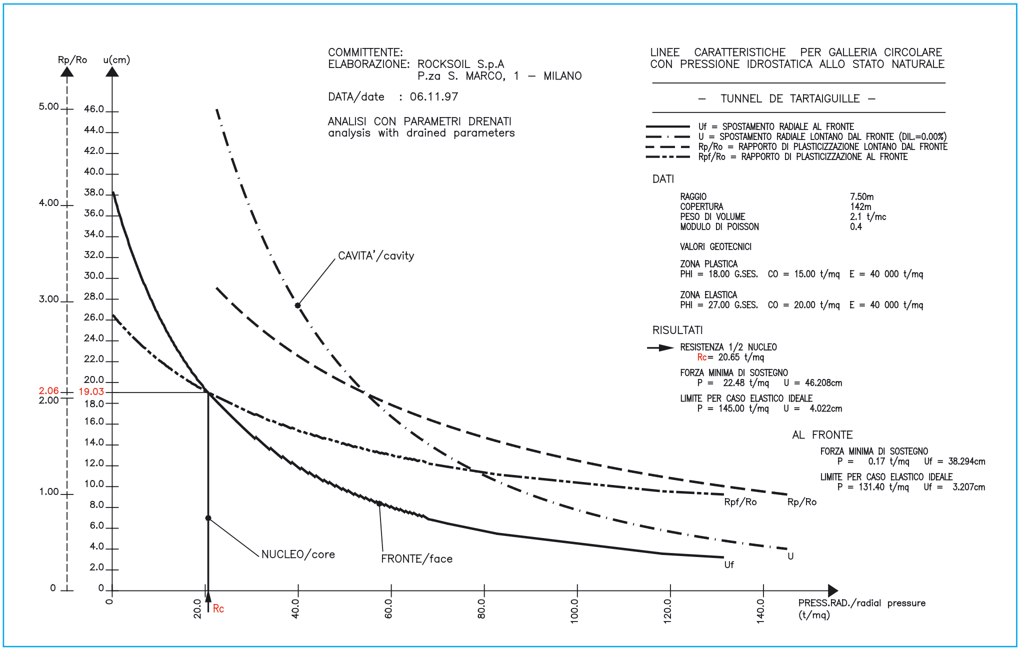


Fig. C.5 *Characteristic lines (diagnosis phase)*

A singular sudden, and rapid loss of consistency in the material was immediately seen and this placed the accent on the need to intervene appropriately ahead of the face to prevent a substantial band of plastic behaving material from developing around the cavity. Considering also the considerable presence of swelling minerals in the ground, this was felt to be the main cause of the disastrous deformation that had occurred and of the excessive loads on the linings that had been observed. Preventing this band of plastic material from developing was to be achieved by energetic preconfinement of the cavity capable of countering deformation before it is triggered.

The therapy stage

On the basis of the findings of the diagnosis stage, it was decided to cross the lower Stampien clays by adopting a cross section type that involved full face advance after first reinforcing the ground in the advance core and then almost immediately closing the preliminary lining by placing the tunnel invert.

The reinforcement of the advance core (Fig. C.6) was achieved by means of (an average of) 90 fibre glass structural elements 24 m in length, injected with an expanding cement mix. It also required the excavation and casting of the tunnel invert in steps of between 4 and 6 m behind the face and the placing of the final lining at between 20 and 40 m from the face.

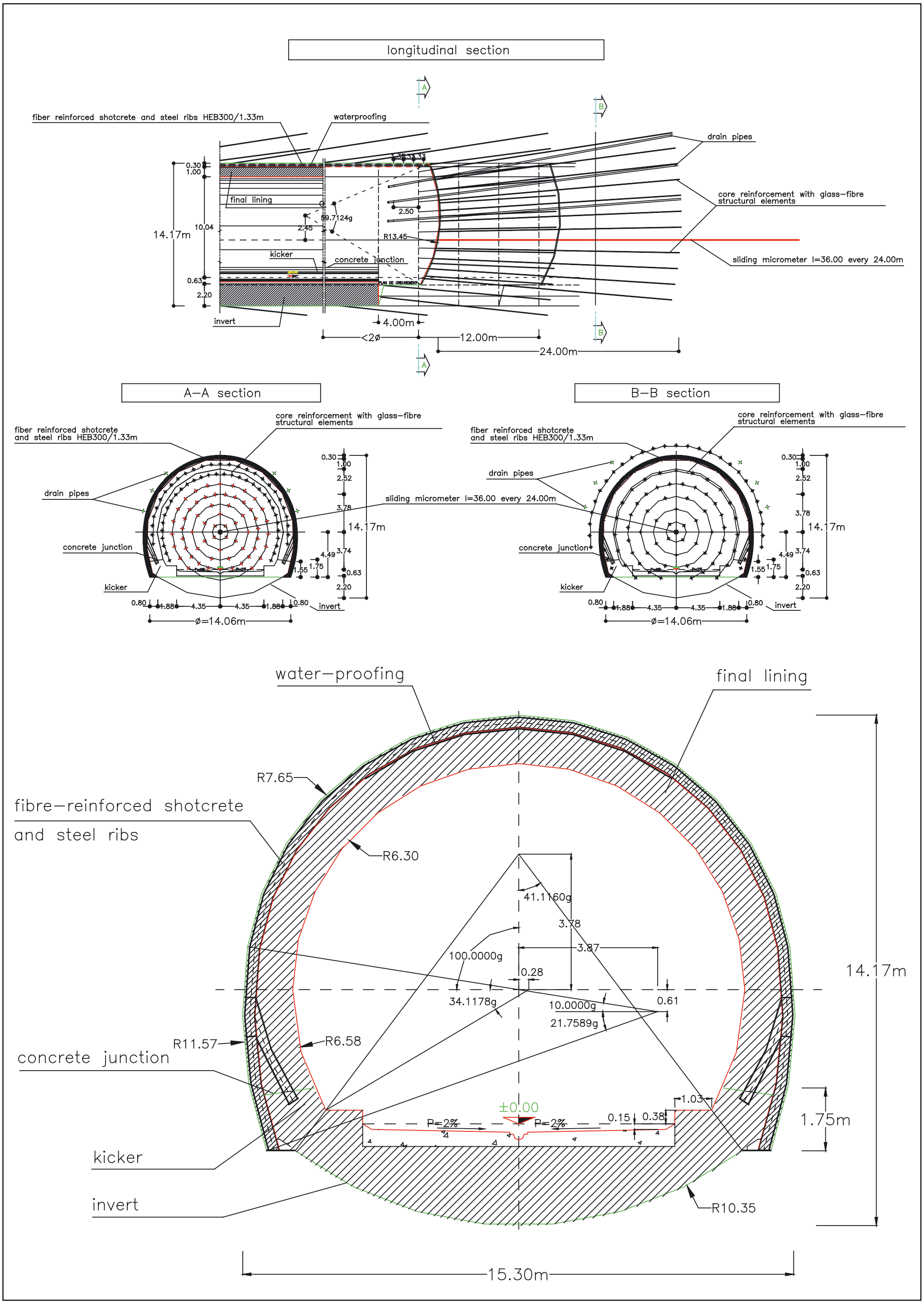


Fig. C.6 *Ground reinforcement and linings*

The preliminary lining was to have consisted of 30 cm of fibre reinforced shotcrete, also reinforced with HEB300 steel ribs placed at intervals of between 1.33 and 1.50 m. To meet the request of the contractor to maintain the formwork used for the section of tunnel already excavated (which would make it impossible to construct side walls high enough to contain the extremely high horizontal thrusts) it was finally decided to fit the steel ribs with a structural element (*jambe de force*) capable of transferring the loads from the preliminary lining onto the tunnel invert and vice versa (Fig. C.10).

Establishing the magnitude of the reinforcement of the advance core was initially performed by reprocessing the results of the extrusion tests (Fig. C.11) and by the characteristic line method. It was then tested using 2D and 3D finite element models in the elasto-plastic range (Fig. C.12) with which it was also possible to test the preliminary and final linings under the effects of complex stress-paths such as those due to *fluage* and swelling (Fig. C.13).

The operational stage during constrution

The full cross section of the "Tartaiguille" tunnel, as has been said, is approximately 180 m²; each steel rib (HEB300) weighed approximately 5 tonne, the volume of shotcrete placed in the lining was 13.3 m²/m and each reinforcement of the advance core required 2,160 m of fibre glass structural elements.

Despite the considerable quantities briefly described here, the advance system proposed (see diagram Fig. C.14) enabled contractual deadlines to be met and industrialisation of the various operating stages resulted in average advance rates of 1.7 metres/day (Fig. C.15), even faster than the 1.4 metres/day forecast by the consulting engineer. As a consequence, the tunnel was completed just one year after the start of full face advance (end of July 1997), a full month and a half ahead of schedule.

The monitoring phase during construction

Fine tuning of the tunnel section design in terms of:

- number of fibre glass structural elements at the face (90–150);
- length of overlap between them (9–12 m);
- length of tunnel invert steps (4–6 m);
- spacing between steel ribs (1.33–1.50 m);
- distance from the face at which the final lining was cast in the roof arch (20–40 m);

Fig. C.7 *Reinforcement of the advance core*

Fig. C.8 *Placing of steel ribs*

Fig. C.9 *Reinforcement of the core-face by means of fibre glass structural elements*

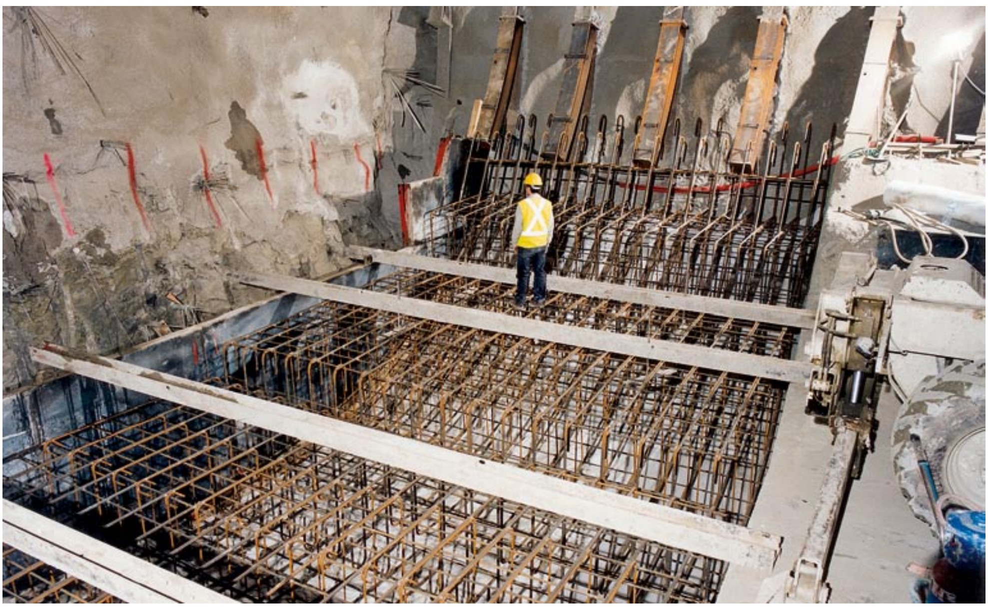


Fig. C.10 *Preparation of the tunnel invert. Note the "jambe de force"*

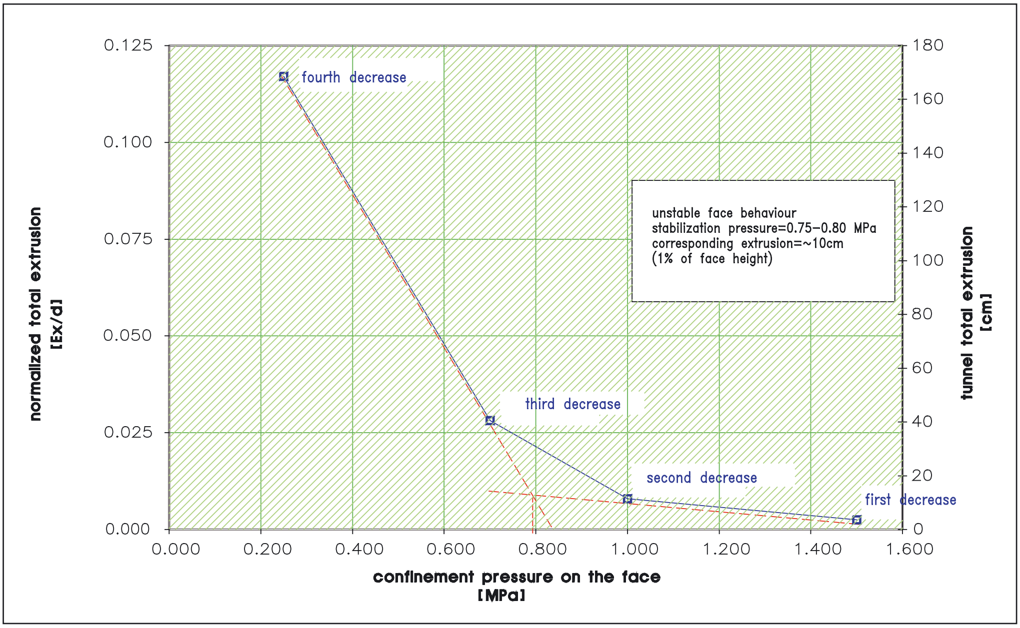


Fig. C.11 *Results of the 3D F.E.M. modelling of face extrusion (values in cm)*

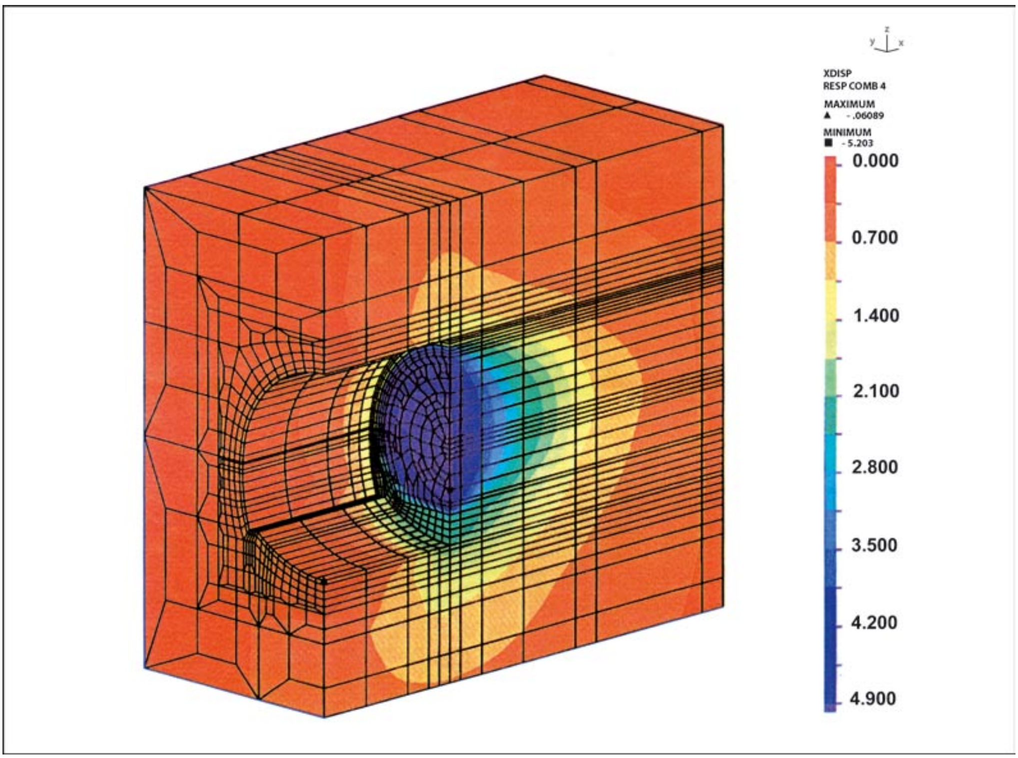


Fig. C.12 *Reprocessing the extrusion tests to calculate the dimensions of core-face reinforcement*

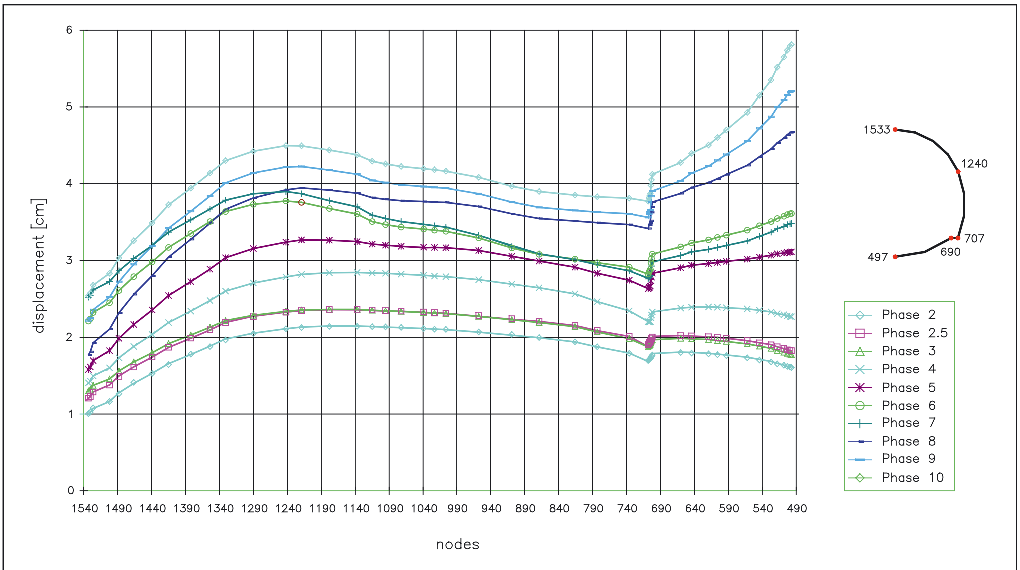


Fig. C.13 *Results of 2D modelling of radial displacements*

was performed on the basis of processing and interpretation of monitoring data and in particular on systematic measurement of extrusion of the advance core and convergence of the cavity.

With tunnel advance fully underway, average values (Figs. C.16 and C.17) for extrusion were around 3–4 cm and for diametric convergence around 5–7 cm (after casting the tunnel invert).

An examination of the extrusion curves in particular shows that the size of the zone of influence of the face varies between 8 and 13 m. This can be deduced from the appreciable change in the angle of the curves at this depth. Furthemore, it can be seen that the excavation of the last 6 m produces extrusion of 15–18 mm comparable to that of the first 6 m and that thanks to the ground improvement, the advance core behaved elastically confirming that the number of fibre glass structural elements placed had been well calibrated.

The extrusion curves obtained in the laboratory did in fact show that the ground tended to rapidly lose its strength above all in the clayey fraction and that action had to be taken to limit deformation to a minimum for excavation to occur in conditions of safety. As already stated, this is the principle that guided the design from the very beginning and was still maintained even during the stage of fine tuning the tunnel section during construction.

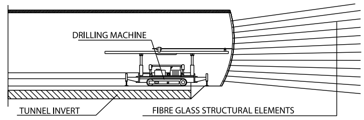
Convergence measurement stations were installed every 6 m with measurement points consisting of 7 optical targets inserted both directly into the shotcrete and on the steel ribs. Measurements obtained from the latter were 60–70% lower on average than those taken from the shotcrete.

FULL FACE ADVANCE AFTER REINFORCEMENT OF THE CORE-FACE

Operational stages

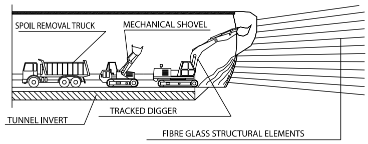
PHASE 1

Reinforcement of the core-face by means of fibre glass structural elements



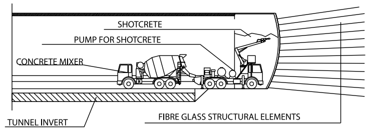
PHASE 2

Excavation step
(m 0.70 ~1.00 full face)



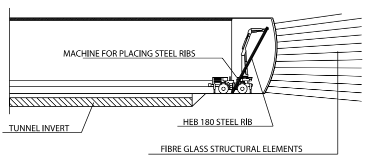
PHASE 3

Placing of shotcrete on the face and surface of the cavity to protect walls of the tunnel (thickness around 5 cm)



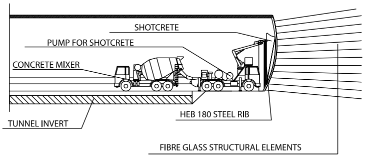
PHASE 4

Placing of the steel rib and connecting reinforcement in the section of tunnel excavated in phase 2



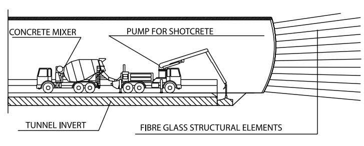
PHASE 5

Placing of the preliminary lining in fibre reinforced shotcrete (thickness around 35 cm)



PHASE 6

Excavation and casting of side walls and tunnel invert at a distance from the face of ≤1.5 Ø



PHASE 7

Casting of the final lining at distance from the face of ≤4–5 Ø

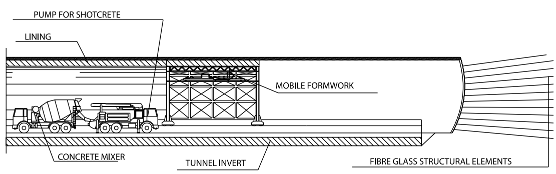


Fig. C.14 *Operating phases (with relative photographs)*

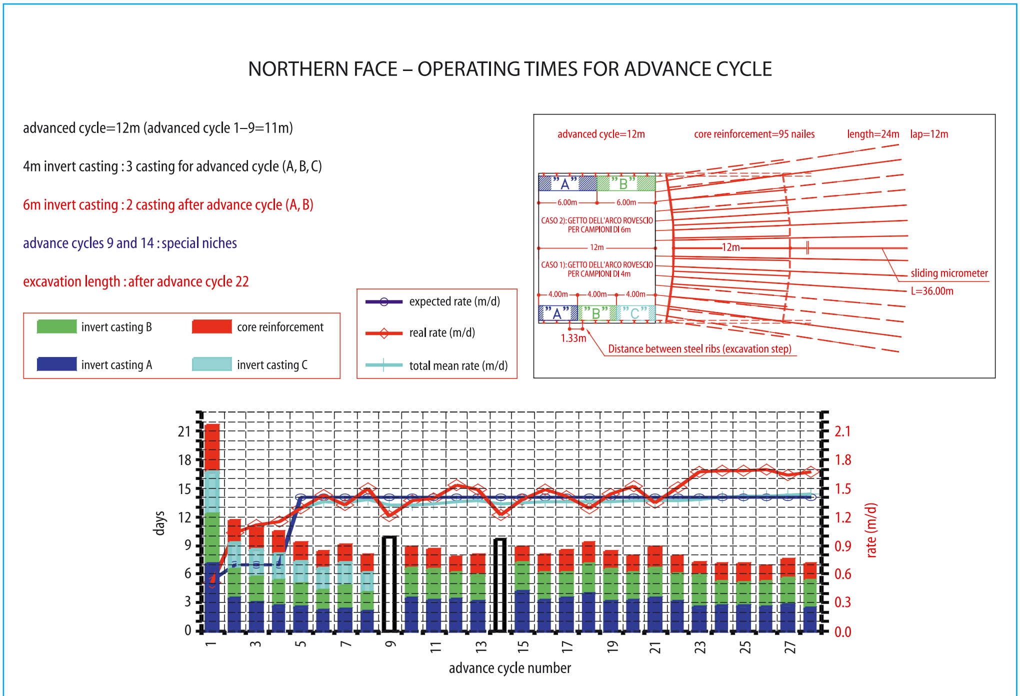


Fig. C.15 *Production data*

The measurement sections in which targets were installed before the tunnel invert was placed gave convergence readings of around 2 or 3 cm with gradients of 4–6 mm per day until the tunnel invert was cast. After the ring of the preliminary lining was closed, however, the gradient fell to 1–3 mm per day.

This data shows therefore that deformation (convergence-extrusion) was generally maintained within the elastic domain characteristic of the ground, thanks to the substantial limitations placed on the possibility of a band of ground with plastic behaviour developing around the cavity. This confirms the effectiveness of the intervention proposed and adopted both in terms of ground improvement and reinforcement and operating methods.

Each time a cross section of ground larger than the current section was excavated, as for example required for the construction of recesses in the tunnel walls, the consequent greater relaxation of the material was followed by a significant increase in deformation (up to 60%), confirming the extreme sensitivity of the ground and the importance of reducing decompression and the time taken to place linings to a minimum.

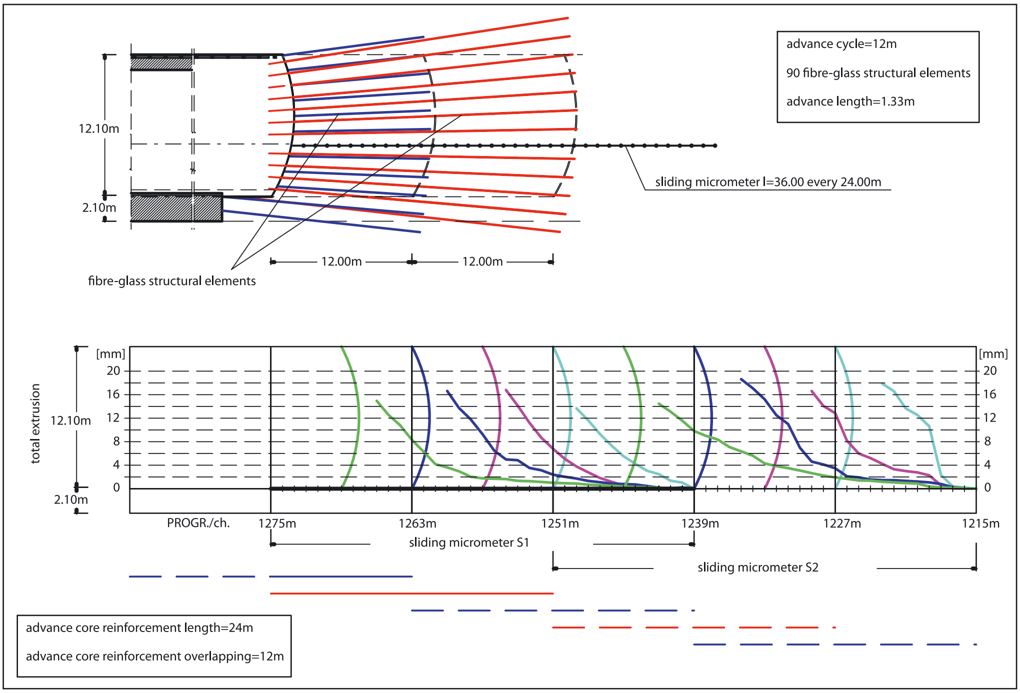


Fig. C.16 *Results of extrusion measurements*

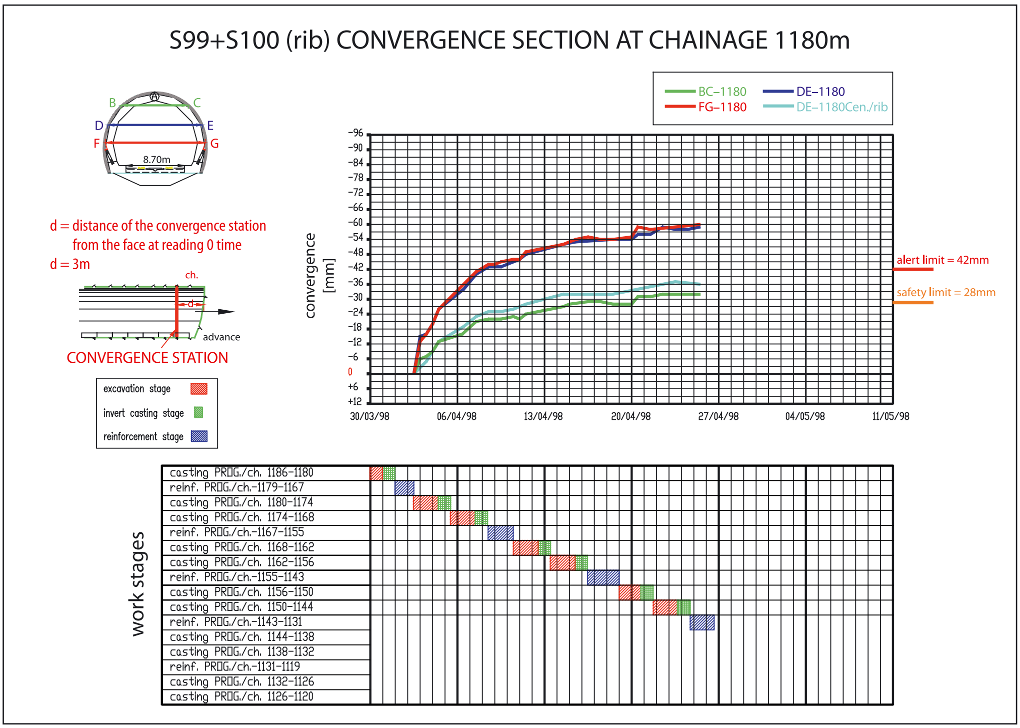


Fig. C.17 *Results of convergence measurements*

Conclusions

Construction of the central section of the "Tartaiguille" tunnel (180 m² cross section) in the clays of the Stampien formation was the cause of considerable concern to the consulting engineers and contractors involved, when the inadequacy of the design and construction systems adopted (NATM) soon became clear. They were unable to guarantee the feasibility and safety of the works with acceptable advance rates when faced with the extensive swelling phenomena characteristic of the ground concerned. Given these problems, SNCF sought a solution to the problem that would enable them to construct the tunnel in safety and on schedule without delaying the opening of the new line. They therefore set up a study group and invited the most renowned European tunnelling experts to form part of it. Various proposals were examined, but the only one that seemed able to meet all the requirements demanded was the Italian proposal suggested by the author and drawn up by Rocksoil S.p.A. of Milan. It was based on ADECO-RS principles and consisted of full face advance after first reinforcing the advance core using fibre glass structural elements. The adoption of this design, together with the on site assistance of engineers from Rocksoil S.p.A. enabled a difficult tunnel to be driven without problems, with very reduced deformation, ahead of schedule and at costs even lower than budgeted.

This considerable achievement made a great impression and aroused some degree of amazement in France, where the specialist press paid numerous tributes to all the Italian consulting engineers, technicians and construction contractors who supplied the know-how required to complete the project on schedule ["*Débuté en juillet, le chantier, qui fait travailler 200 personnes, posait principalement des difficultés liées aux pressions exercées par la montagne. Une nouvelle méthode a donc été instaurée sur l'idée d'un ingénieur italien: le percement en pleine section (plotôt qu'en demi-section) …*" (Tunnels et ouvrages souterrains, January/February 1998), "*Lorsqu'elle en prend les moyens, l'Italie peut réaliser des travaux à faire pâlir les entreprises françaises …*"(Le Moniteur, 20th February 1998). "*Le creusement du tunnel de Tartaiguille a été très difficile, en raison notamment de convergences inattendues du terrain qui ont nécessité un changement de méthode en cours de chantier: le professour italien Pietro Lunardi a convaincu la SNCF de travailler à la pelle en pleine section dans les argiles, en boulonnant le front sur 24 m …*" (Le Moniteur, 7th August 1998)]. It was therefore a great success, which has established Italian leadership in the field of underground works under difficult stress-strain conditions.


Cellular arch technology: view of the structure built for the Venezia station of the Milan urban link line (1990, ground: sand and gravel under the water table)

Cellular arch technology

The cellular arch is an innovative construction technology designed for the construction of large span tunnels in urban environments when the geotechnical and stress-strain situations, the shallow overburdens and the requirement for construction work to have negligible effects on surface constructions and activities are either not compatible with conventional tunnelling methods, or make them less reliable and competitive.

It is a composite structure with a trellis-like framework and a semi-circular cross section. The longitudinal members (cells) consist of pipes (minitunnels) filled with r.c. joined together by a series of large transverse ribs (arches) (Fig. D.1). Studies performed to find the limits to its application suggest that it can be employed successfully to build shallow overburden, bored tunnel cavities with a span of more than 60 m even in loose soils under the water table, without causing any appreciable surface settlement.

The advantage of this technique over traditional methods lies in the way that the passage from the initial equilibrium condition of the still undisturbed ground to the final equilibrium condition of the finished

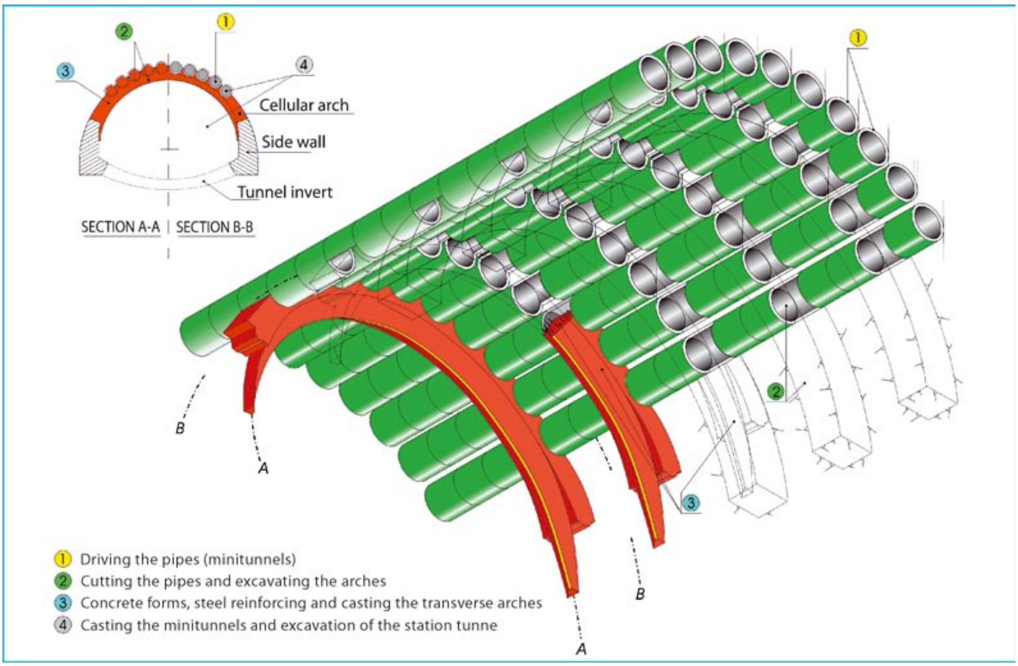
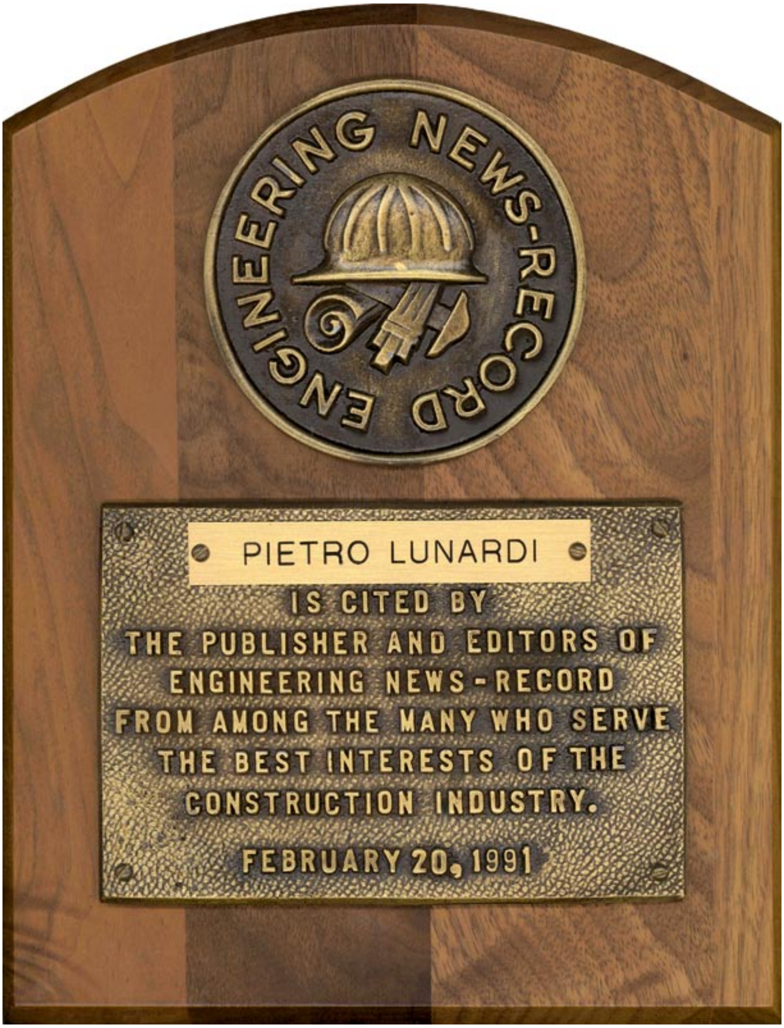


Fig. D.1 *Construction stages for the cellular arch*



The plaque awarded to the author by the United States publication Engineering News – Record, which makes a "Construction's man of the year" award each year tunnel is controlled to prevent the onset of decompression in the material and consequently also of surface settlement.

Excavation is in fact performed when the very rigid load bearing structure has already been constructed and is able to furnish the ground with the indispensable confinement required without suffering any appreciable deformation.

To achieve this the entire construction of the cellular arch structure takes place in the following stages (Fig. D.2):

- half face excavation of side drifts for the tunnel walls after first improving the ground from a pilot tunnel and completion of ground improvement around the final tunnel;
- completion of excavation of the tunnel wall side drifts and casting of the tunnel sidewalls inside them while, in a completely independent site above, a series of r.c. pipes (minitunnels) are driven side by side (2.10 m in diameter) into the ground along the profile of the future tunnel roof from a thrust pit (by pipe jacking them);
- excavation from the minitunnels of transverse tunnels to be used as the formwork (the walls of this consisting of the ground itself) for the casting of the connecting arches in r.c. and then placing of reinforcement and casting of the arches themselves;
- the ten longitudinal minitunnels forming the tunnel arch are filled with concrete and then the tunnel for the station is excavated beneath the protection of the “cellular” arch practically already active;
- the tunnel invert is cast in steps.

This innovative construction method for which the author, who invented it, was awarded a prestigious international award was used for the first time in the world to construct the “Venezia” station of the Milan urban link railway line. If conventional systems had been used its construction would have been much more troubled and costly, if not actually impossible.

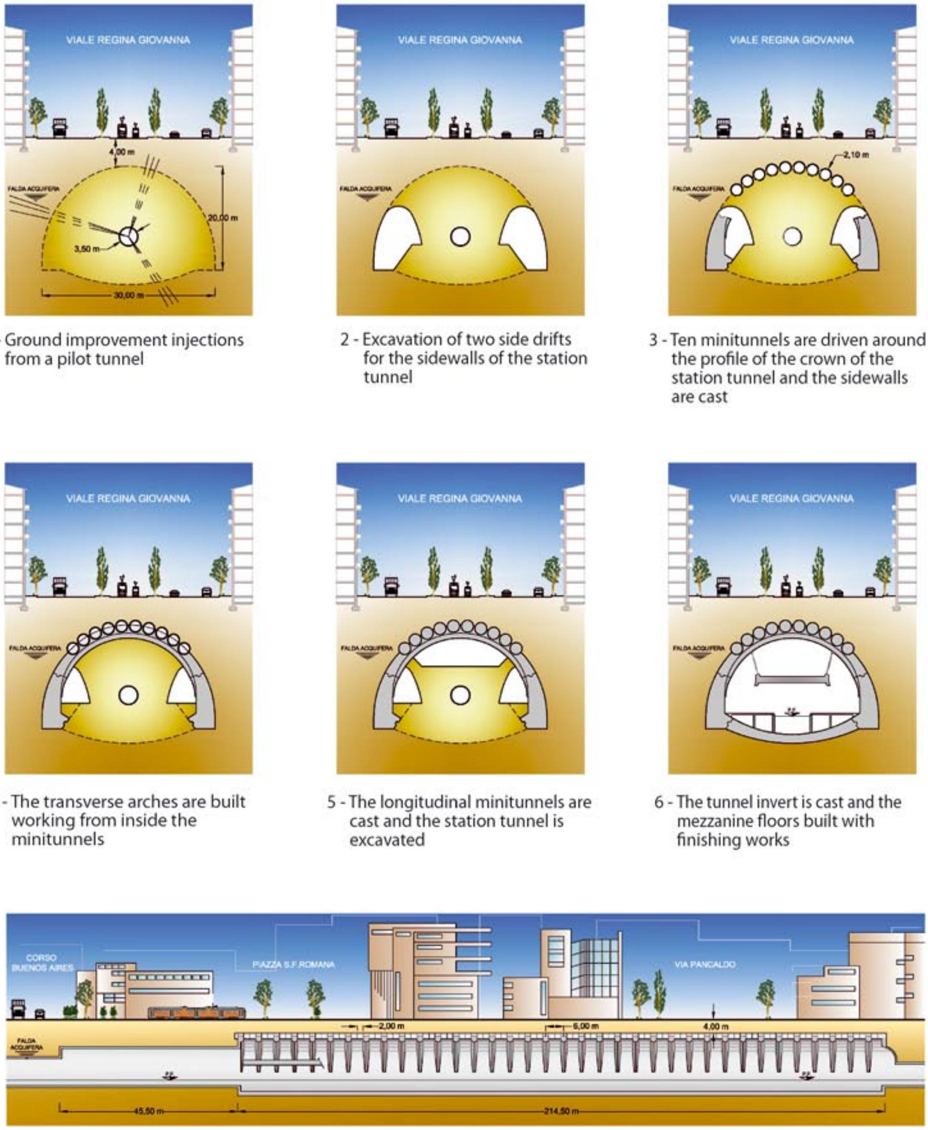
What follows is an attempt to fully illustrate the characteristics of the technology on the basis of the experience acquired during the construction of this exceptional civil engineering work with a focus also on the operational aspects of construction and the final results achieved.

The construction of “Venezia” station

Venezia station has now been in service for a number of years. It is located strategically in the commercial centre of Milan and is the largest underground construction in the whole of the regional transport system.

It is a large tunnel with an external diameter of approximately 30 metres and a length of 215 metres. It is bored tunnel that passes through cohesionless soil under the water table with an overburden

MILAN URBAN LINK RAILWAY LINE
THE CONSTRUCTION OF THE CELLULAR ARCH STRUCTURE FOR VENEZIA STATION



1 - Ground improvement injections from a pilot tunnel

2 - Excavation of two side drifts for the sidewalls of the station tunnel

3 - Ten minitunnels are driven around the profile of the crown of the station tunnel and the sidewalls are cast

4 - The transverse arches are built working from inside the minitunnels

5 - The longitudinal minitunnels are cast and the station tunnel is excavated

6 - The tunnel invert is cast and the mezzanine floors built with finishing works

Fig. D.2

Venezia Station on the Milan urban link railway line: the construction of a sidewall in a side drift

Venezia Station on the Milan urban link railway line: pipe jacking to drive minitunnels side by side around the perimeter of the roof arch of the station

Venezia Station on the Milan urban link railway line: a view of the thrust pit after the minitunnels have been driven of only 4 metres beneath the foundations of 16th century buildings (Fig. D.3). The 440 m² cross section of the station tunnel is six times larger than that of normal twin track metropolitan railway running tunnels and almost twice the size of the second largest tunnel built to-date in Milan. To be able to build it with the same geometry would certainly have been impossible without the cellular arch technology with which the final lining was built before excavation began. It was driven by boring through recent and cohesionless soils. The overall diameter was approximately 30 metres and the overburden under the foundations of ancient buildings was only 4–5 metres.

Such a shallow overburden caused considerable doubt at the design stage over what the results might be if traditional methods were employed based on advance in steps after first improving the ground with sprayed concrete and steel ribs placed immediately. It would in fact have been impossible to improve the ground over the arch of the tunnel before excavation began to a degree that would have been sufficient for the large dimensions of the tunnel cross section. Numerical analyses carried out using the finite element method also showed that a conventional confinement structure of steel ribs and shotcrete would have been too deformable and would not have been able, even temporarily, to contain surface settlement within the limits required to safeguard nearby structures and utilities.

That is why the development of the cellular arch method was necessary.

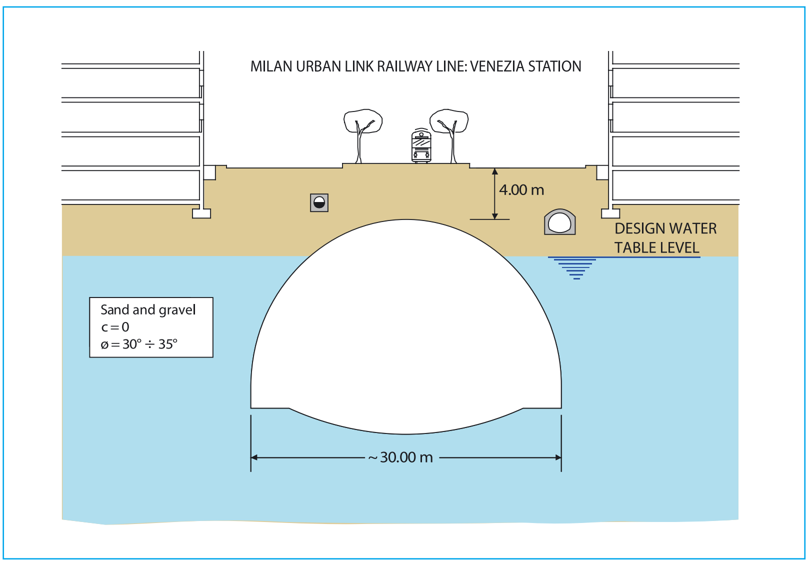
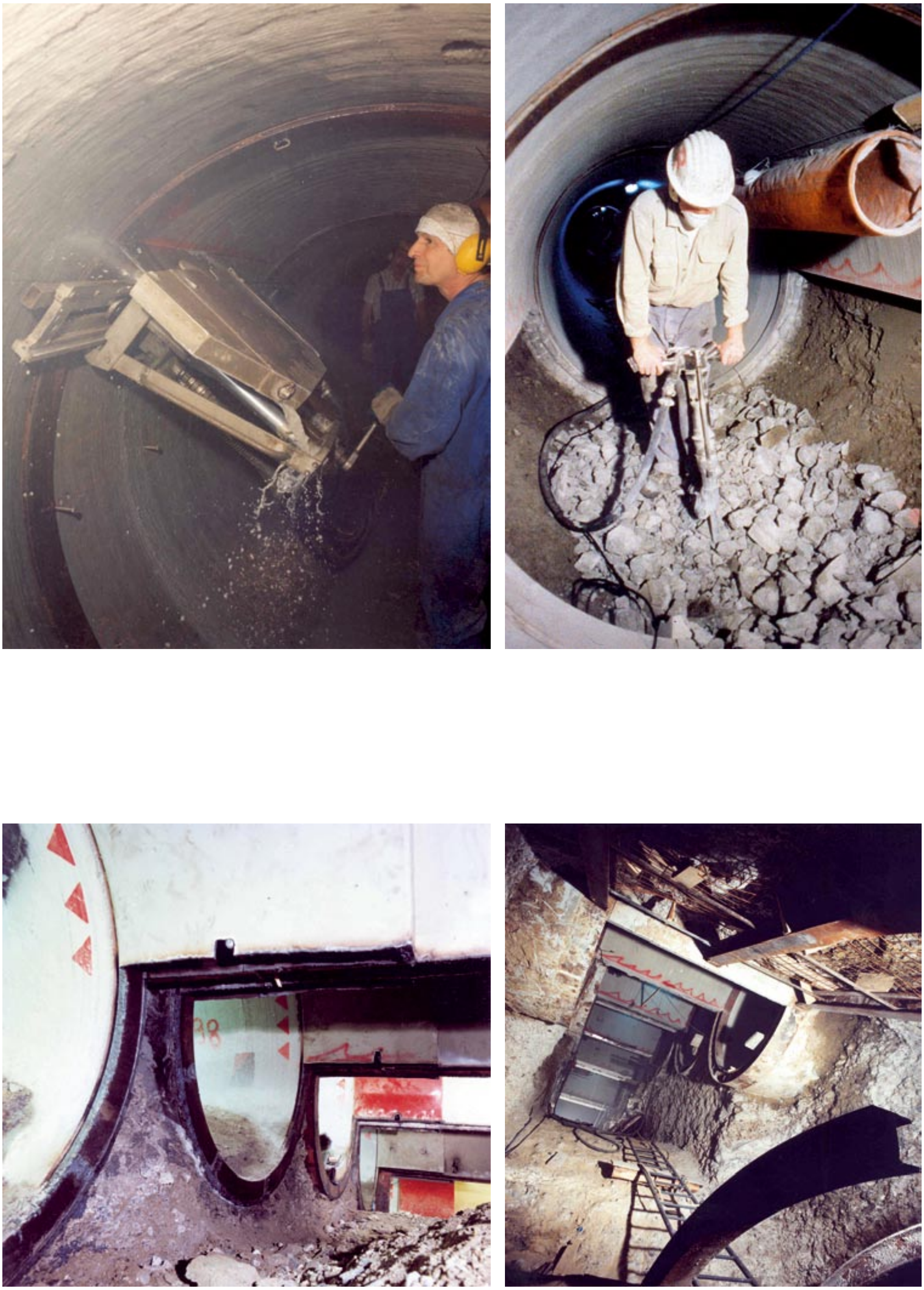


Fig. D.3
The overburden of the ground above the extrados of the tunnel was not sufficient to create a band of treated ground around it of adequate thickness



Venezia Station on the Milan urban link railway line: the cut of the pipes (minitunnels) for creating the transverse arches

The construction of the cellular arch structure for Venezia Station

From an operational point of view, the side drifts 60 m^2 (7.6 m in width and 11.0 m in height) being the same length as the future station tunnel were constructed in two stages:

1. excavation of the crown of 40 m^2 down to the water table;
2. ground improvement injections under the water table, from the floor of the first stage, under the future sidewalls of the tunnel and then subsequent deepening of the excavation down to the foot of the sidewalls.

The lining consisted of steel ribs, wire mesh reinforcement and shotcrete.

Once completed, the side walls of the future station tunnel were then cast inside them. Average production was approximately 2 metres per day of finished side wall, including the time taken for excavation work. It therefore took 11 months to complete the 430 metres of sidewall (215 m for each side), about the same time employed for pipe jacking (minitunnels) which was performed simultaneously from a completely independent site.

The design specification was for the excavation of ten minitunnels along the profile of the crown of the tunnel. This involved driving 1,080 pipes for a total length of about 2,160 m. The pipes were prefabricated, manufactured using the radial prestress system and high strength cement mixes, and had an outer diameter of 2,100 mm, an internal diameter of 1,800 mm and a length of 2 m.

They were pipe jacked into the ground from a thrust pit (Fig. D.4) using equipment consisting of a cylindrical metal shield with a diameter of 2,100 mm and a length of 7.7 m divided into two parts: the front part for cutting, which was jointed to allow the operator to adjust the vertical and horizontal movement, was fitted with a computer operated hydraulic cutter and a conveyor belt for mucking out and the rear part, 3.50 m long containing, motors, pumps and reservoirs for the hydraulic fluid. The thrust equipment included two hydraulic long stroke jacks, the indispensable load distribution structures and a hydraulic pump operating at a pressure of 600 bar.

Two sets of equipment were employed to obtain daily pipe advance rates of approximately 8–9 m per day. Topographic monitoring carried out during and after pipe jacking ensured and then confirmed that it was performed accurately with negligible deviations in direction and depth.

Once the side walls had been cast and all the pipes driven into place, construction of the load bearing cross members of the arch of the future tunnel started, undoubtedly the most characteristic part of the cellular arch technique.

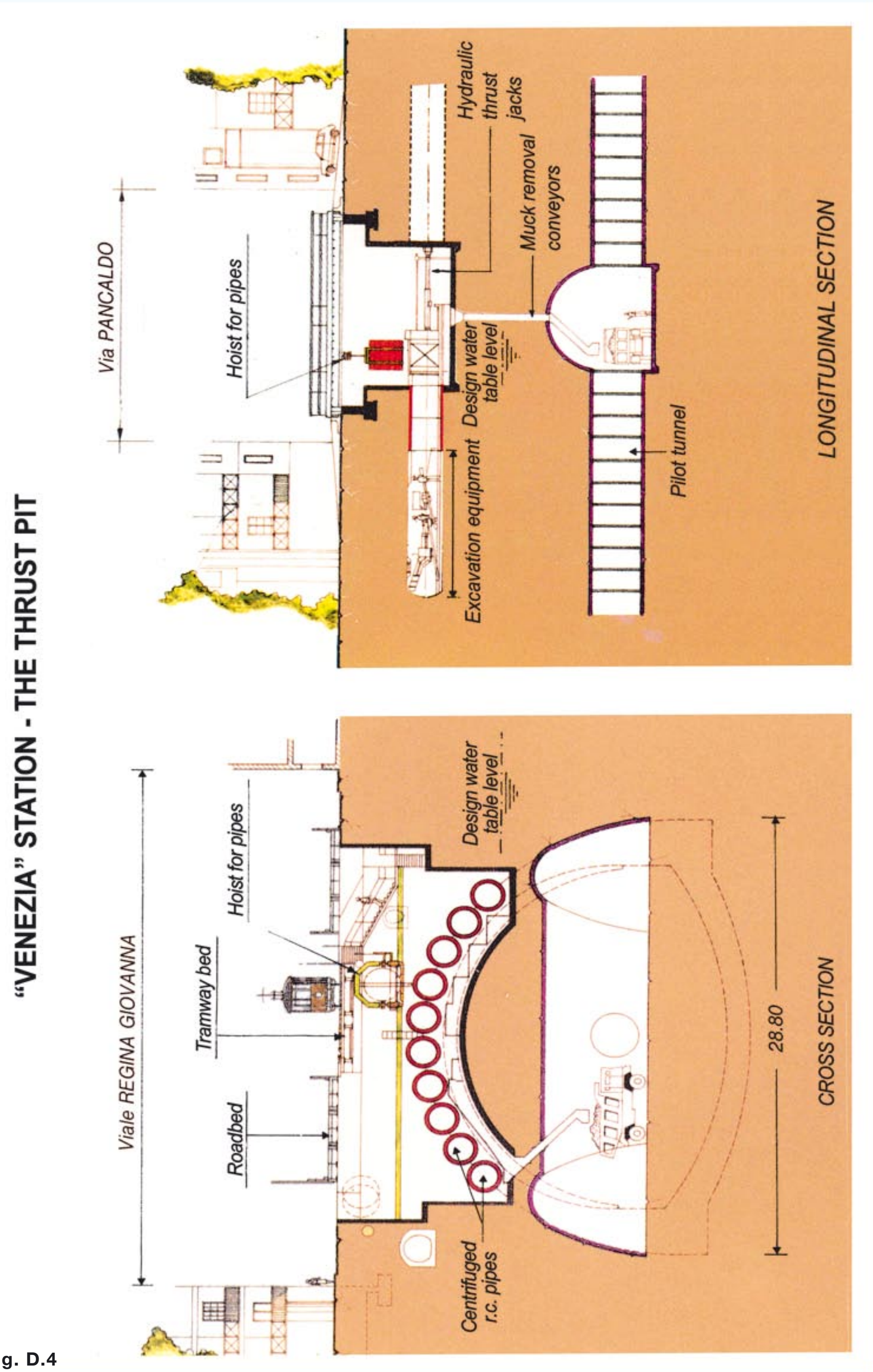
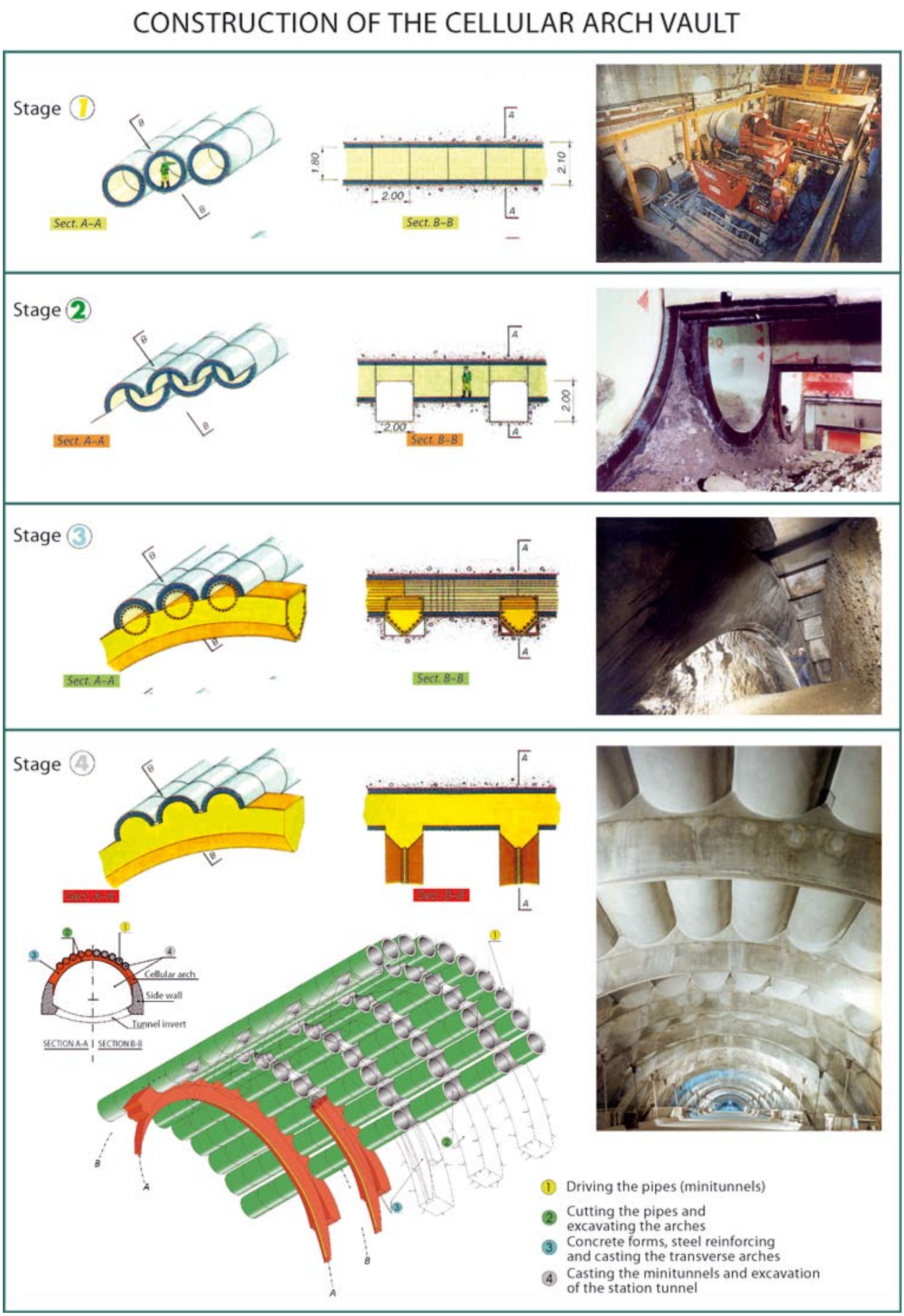
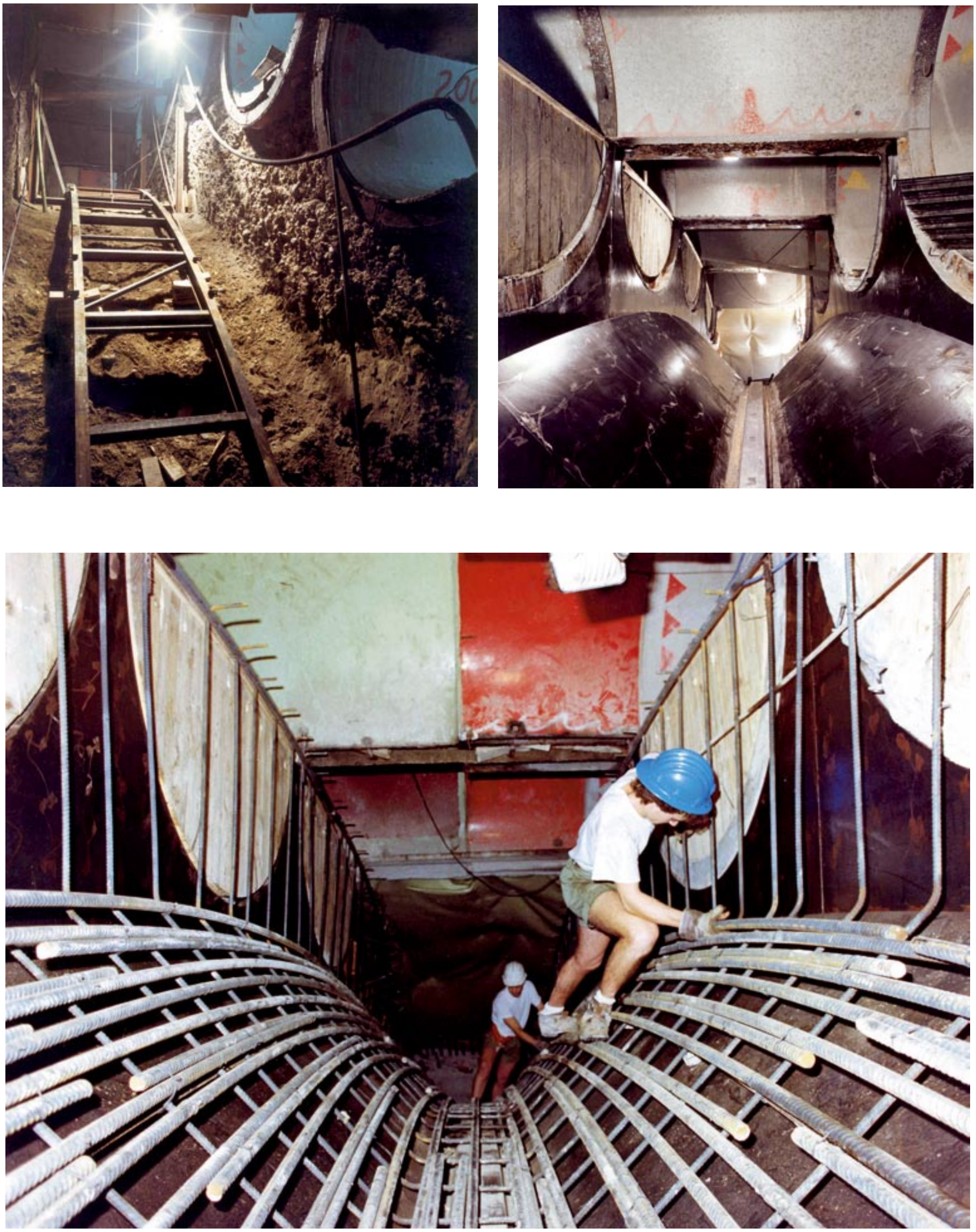

"VENEZIA" STATION - THE THRUST PIT
Via PANCALDO
Hoist for pipes
Hydraulic thrust jacks
Muck removal conveyors
Design water table level
Excavation equipment
Pilot tunnel
LONGITUDINAL SECTION
Viale REGINA GIOVANNA
Roadbed
Tramway bed
Hoist for pipes
Design water table level
Centrifuged r.c. pipes
28.80
CROSS SECTION

Fig. D.4



CONSTRUCTION OF THE CELLULAR ARCH VAULT
Stage 1
Sect. A–A
Sect. B–B
1.80
2.10
2.00
Stage 2
Sect. A–A
Sect. B–B
2.00
2.00
Stage 3
Sect. A–A
Sect. B–B
Stage 4
Cellular arch
Side wall
Tunnel invert
SECTION A-A
SECTION B-B
1 Driving the pipes (minitunnels)
2 Cutting the pipes and excavating the arches
3 Concrete forms, steel reinforcing and casting the transverse arches
4 Casting the minitunnels and excavation of the station tunnel

Fig. D.5



Venezia Station on the Milan urban link railway line: stages for the construction of the arches inside the transverse drifts

Venezia Station on the Milan urban link railway line: excavation of the station tunnel under the protection of the cellular arch already constructed. Note the presence of an arch with the formwork not yet removed

Venezia Station on the Milan urban link railway line: a view after half face excavation

Fig. D.6 *Venezia Station on the Milan urban link railway line: the full cross section*

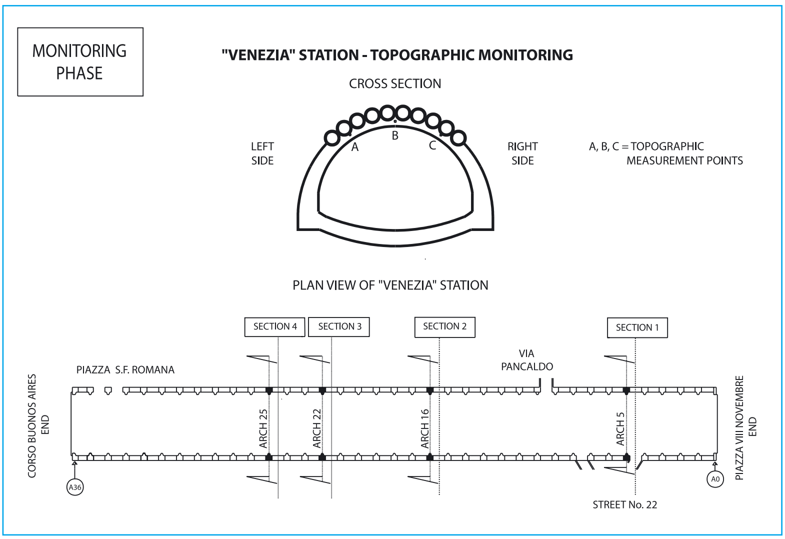


Fig. D.7

The cross members consisted of a series of 35 intermediate arches placed at intervals of 6.00 m plus the two end pieces. Construction was performed as follows (Fig. D.5):

1. cutting and removal of the part of the pipe intersected by the arches and excavation of the arches mainly by hand down to the tunnel wall side drifts;
2. assembly of the prefabricated steel forms inside the excavation, placing of the reinforcement for the cells (minitunnels) and the arches and casting of the latter on the side walls already in place;
3. casting of the cells.

Excavation in steps of the top heading in the crown and the bench of the large tunnel was then able to start in complete safety under the already active load bearing cellular arch structure. The lining was finally completed with an invert, varying from 1.5 to 2 m in thickness (Fig. D.6). It was cast in 5 m steps for a total length of 92 m with a total average cross section of 38 m^3/m. Each 5 m advance took a seven day working week to complete on average and work was co-ordinated so that the ring of the tunnel lining was left unclosed for three days only.

Finally, it is interesting to consider that the average overall production rate for the civil engineering works of Venezia station, constructed using the cellular arch technique, was 57 m^3 per day, for a final cost of approximately 516 euro/m^3, comparable to, if not less than, the current price of an ordinary one automobile garage in the centre of the city.

Monitoring system

The large dimensions of the cavity, the completely new construction method and the delicate surface constraints meant that a vast monitoring programme had to be designed and implemented to measure:

- settlement, at ground level, especially of buildings, during all phases of the works;
- deformation of the ground around the tunnels;
- stresses and strains in the final lining of the tunnel.

The programme included (Fig. D.7):

- topographic measurements;
- levelling, deflectometer and inclinometer measurements to monitor the development of deformation on existing buildings;
- convergence measurements in tunnels;
- pressure and deformation measurements on the lining structures.

Continuous recording and processing of the various measurements taken enabled the stress-strain conditions of the ground and the lining to be kept under control during the various stages of construction to provide a useful and constant comparison with both the design forecasts and the threshold limits within which existing structures will maintain their integrity.

Venezia Station on the Milan urban link railway line: the construction site was visited by more than forty delegations of engineers from all over the world during construction work

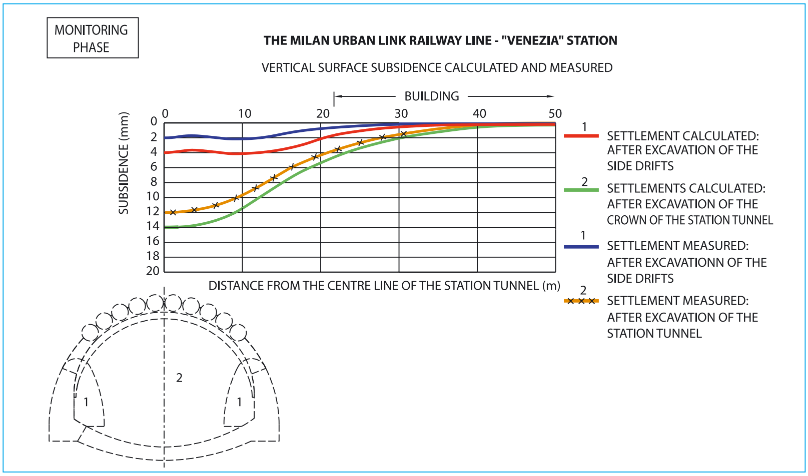


Fig. D.8

As Fig. D.8 shows, surface movements always remained below the calculated values. Of course, the greatest settlement occurred during excavation of the crown. The increase in deformation, slow at first and then more rapid as soon as the face passed the cross section measured, gradually died down as the face moved further away. This behaviour shown from datum points at street level above the route of the tunnel (Fig. D.9) was confirmed, though to a less marked extent, by those located on buildings for which maximum settlement during the passage of the face did not exceed 1–2 mm.

This monitoring system guaranteed constant surveillance over actual conditions demonstrating the compatibility between the efficiency of the construction system and the urban environment, to provide a reassuring overall picture.

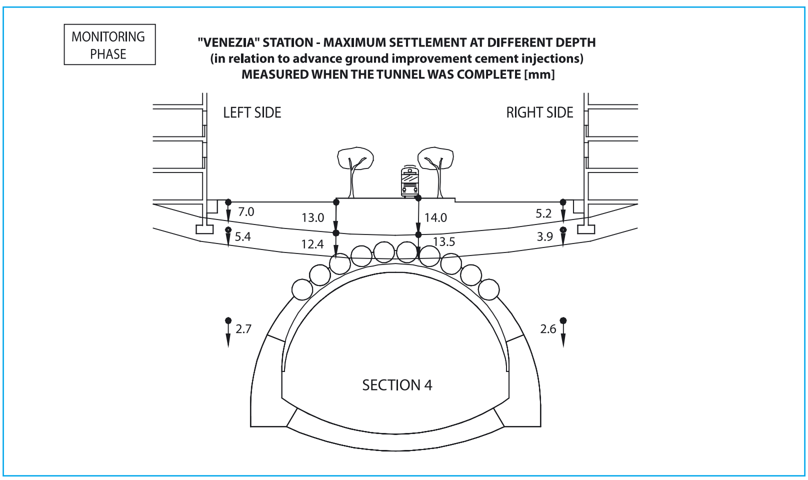
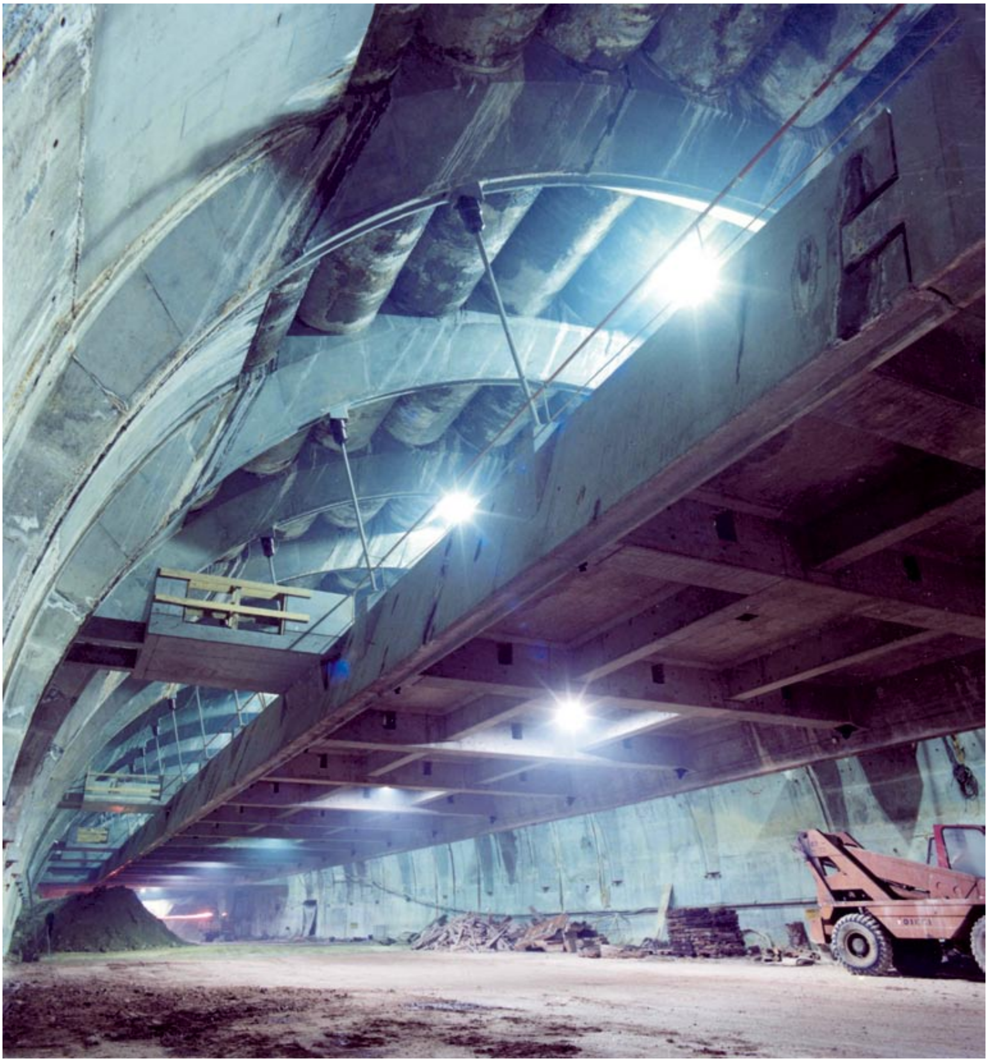


Fig. D.9



Venezia Station on the Milan urban link railway line: the mezzanine level

Possible developments of the system

Studies were performed to establish the limits of the cellular arch method for the construction of wide span tunnels in cohesionless soils, with shallow overburdens, under the water table.

Once a basic outline of the problem had been constructed and the variables determined, approximate dimensions of the main construction elements were calculated using a one dimensional finite element model to simulate the behaviour of the structure and its interaction with the ground. Three different geometries were considered with S:H ratios of 2.09, 1.73, and 1.5, with the span S varying up to 60 m.

The results of the calculation led to the production of tables giving the minimum thickness of the structural elements and surface settlement as a function of the geometry, the outer diameter, the depth of the water table and the size of the overburden (see the example in Fig. D.10). It would appear from the results that the cellular arch method could be employed with success on the bored tunnel construction of shallow tunnels with a span of over 60 m in cohesionless ground, under the water table without any significant surface settlement.

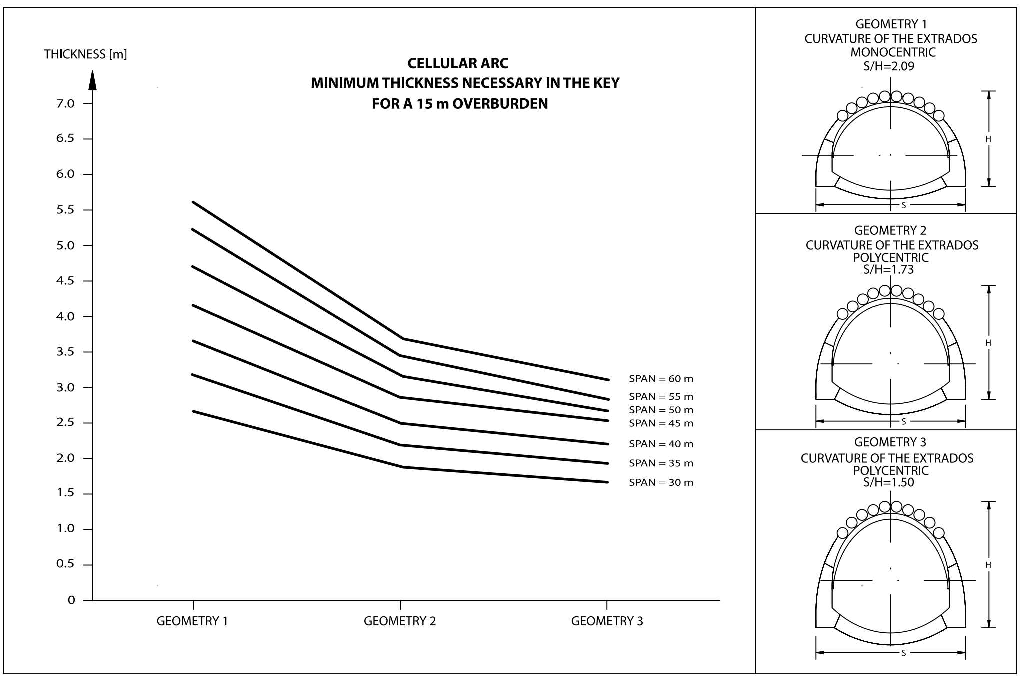


Fig. D.10

Venezia Station on the Milan urban link railway line: view of the mezzanine floor of the finished station

Venezia Station on the Milan urban link railway line: view of the cellular arch structure from the turnstiles

Artificial Ground Overburdens: the "Borgo Rinzelli" tunnel (high speed/high capacity railway line between Bologna and Florence), completely bored tunnel construction under A.G.O. with a very shallow overburden

Artificial Ground Overburdens (A.G.O.)

Some of the tunnels forming part of the works for the construction of the new high speed Rome-Naples railway line had very shallow overburdens because of their vertical location on the alignment and the gentle nature of the morphology. Consequently, the detailed design specified long sections of artificial tunnel. The tunnels concerned were Piccilli 1, Piccilli 2, Castagne, Santuario, Caianello and Briccelle. However, the construction of an artificial tunnel requires deep cuts to be made into the slopes to be crossed with consequent problems of:

- safety with regard to the stability of the slope itself;
- what to do with the huge volumes of material excavated;
- solving the problem of any interference there may be with existing surface structures;
- more difficult statics conditions if there is seismic activity;
- environmental and landscape impact.

These considerations, and also those of costs, led to the study of an alternative solution for the construction of these tunnels using direct bored tunnel methods, thereby avoiding all the problems which the execution of the original design would basically have involved.

Artificial Ground Overburdens

As is known the stability and therefore the long and short term existence of a tunnel is dependent on an arch effect being developed in the ground around the excavation by which the excess stresses, that are generated in the material as a result of excavation, are channelled and transmitted to the intact rock mass below it. In the case of a very shallow tunnel there is an insufficiently thick layer of ground above the crown for an arch effect to be generated naturally and consequently the design engineer must act to ensure that one is formed by taking appropriate construction measures. One conventional way of tackling the problem is, as has been said, to build an artificial tunnel. With this method, the arch effect is created by the concrete lining itself, which in the long term will be loaded with the weight of the backfill material.

Another system is to improve the ground around the cavity before driving the bored tunnel in order to give a sufficiently thick layer of it the indispensable strength it needs to channel the stresses. This path can only be taken if a minimum layer of ground exists over the crown of the tunnel to be driven on which to perform the improvement and, naturally, if the ground in question can be improved at a reasonable cost. The extremely shallow overburdens (see the longitudinal profile in Fig. E.1) in the case of the tunnels in question made this second path impossible, while the former solution was unattractive because of the difficulties mentioned in the introduction. An innovative solution was therefore studied which, by exploiting some of the particular characteristics of the pyroclastic soils in question, allowed the difficulties to be overcome, the tunnels to be driven using bored methods and the landscape and the environment to be respected.

The idea was triggered in the author's mind by considering not only the low unit weight ($\approx 1.3\,t/m^3$) and the interesting characteristics of the pyroclastic soils found on the sites concerned, but also the ease with which they can be mixed with lime to generate a product with reasonable strength. The method devised consisted of replacing the ground over the crown of the future tunnel with structural elements termed *Artificial Ground Overburdens (A.G.O.)*, obtained by using the ground itself after first treating it appropriately. Figure E.2 illustrates the various operational stages of the new construction methodology.

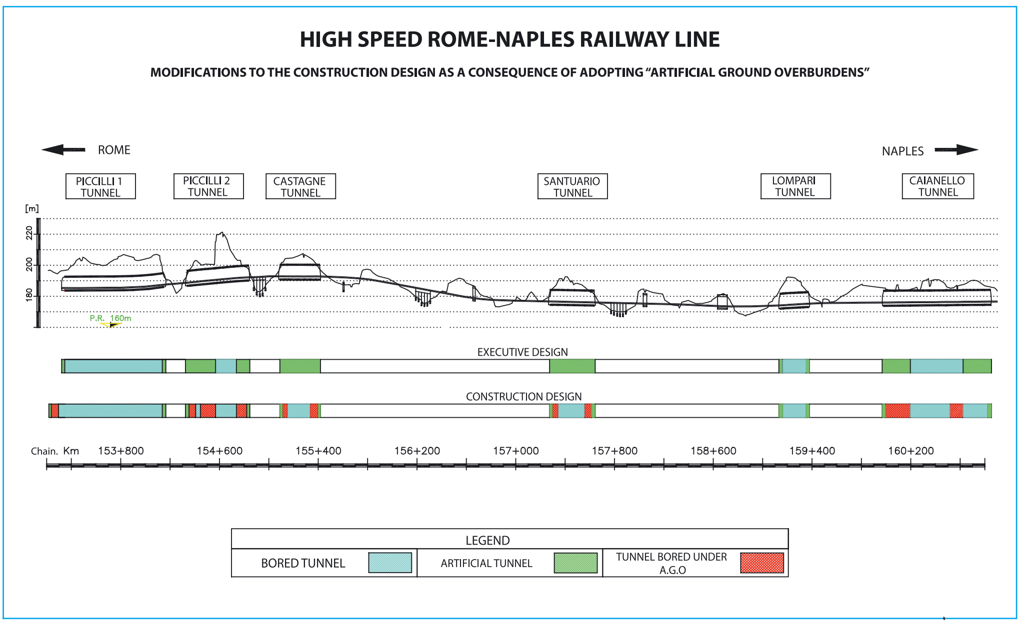


Fig. E.1

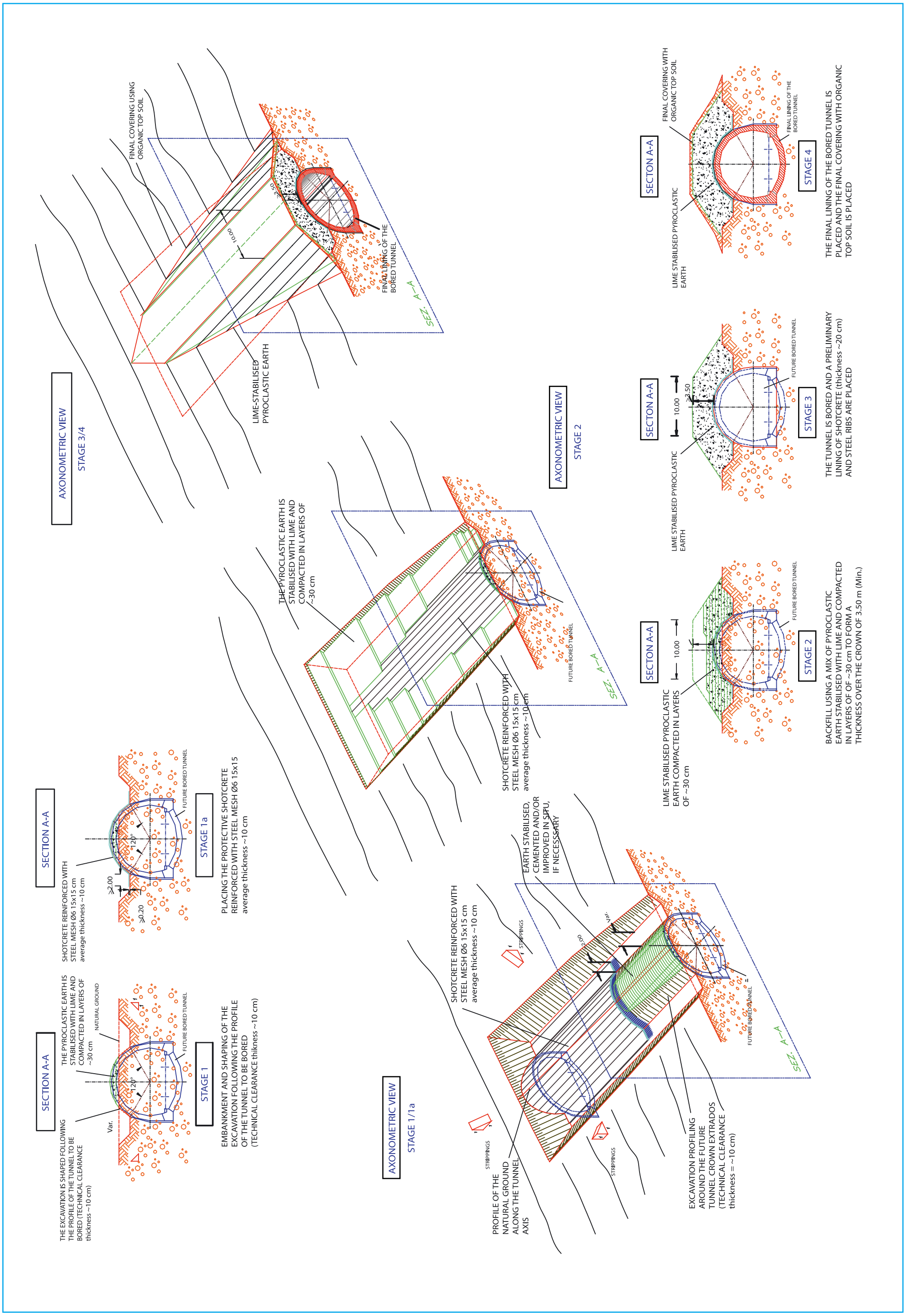

SECTION A-A
THE EXCAVATION IS SHAPED FOLLOWING THE PROFILE OF THE TUNNEL TO BE BORED (TECHNICAL CLEARANCE thickness ~10 cm)
THE PYROCLASTIC EARTH IS STABILISED WITH LIME AND COMPACTED IN LAYERS OF ~30 cm
NATURAL GROUND
FUTURE BORED TUNNEL
STAGE 1
EMBANKMENT AND SHAPING OF THE EXCAVATION FOLLOWING THE PROFILE OF THE TUNNEL TO BE BORED (TECHNICAL CLEARANCE thikness ~10 cm)
SECTION A-A
SHOTCRETE REINFORCED WITH STEEL MESH Ø6 15x15 cm average thickness ~10 cm
STAGE 1a
PLACING THE PROTECTIVE SHOTCRETE REINFORCED WITH STEEL MESH Ø6 15x15 average thickness ~10 cm
AXONOMETRIC VIEW
STAGE 1/1a
PROFILE OF THE NATURAL GROUND ALONG THE TUNNEL AXIS
STRIPPINGS
SHOTCRETE REINFORCED WITH STEEL MESH Ø6 15x15 cm average thickness ~10 cm
EARTH STABILISED, CEMENTED AND/OR IMPROVED IN SITU, IF NECESSARY
EXCAVATION PROFILING AROUND THE FUTURE TUNNEL CROWN EXTRADOS (TECHNICAL CLEARANCE thickness = ~10 cm)
FUTURE BORED TUNNEL
SEZ. A–A
AXONOMETRIC VIEW
STAGE 2
THE PYROCLASTIC EARTH IS STABILISED WITH LIME AND COMPACTED IN LAYERS OF ~30 cm
SHOTCRETE REINFORCED WITH STEEL MESH Ø6 15x15cm average thickness ~10cm
AXONOMETRIC VIEW
STAGE 3/4
FINAL COVERING USING ORGANIC TOP SOIL
LIME-STABILISED PYROCLASTIC EARTH
FINAL LINING OF THE BORED TUNNEL
SECTON A-A
LIME STABILISED PYROCLASTIC EARTH COMPACTED IN LAYERS OF ~30 cm
STAGE 2
BACKFILL USING A MIX OF PYROCLASTIC EARTH STABILISED WITH LIME AND COMPACTED IN LAYERS OF ~30 cm TO FORM A THICKNESS OVER THE CROWN OF 3.50 m (Min.)
SECTON A-A
LIME STABILISED PYROCLASTIC EARTH
STAGE 3
THE TUNNEL IS BORED AND A PRELIMINARY LINING OF SHOTCRETE (thickness ~20 cm) AND STEEL RIBS ARE PLACED
SECTON A-A
FINAL COVERING WITH ORGANIC TOP SOIL
LIME STABILISED PYROCLASTIC EARTH
STAGE 4
THE FINAL LINING OF THE BORED TUNNEL IS PLACED AND THE FINAL COVERING WITH ORGANIC TOP SOIL IS PLACED

Fig. E.2

After first having removed the surface layer of ground present over the tunnel to be constructed, following the profile of the crown, with a technical clearance of 10 cm, down to the springline, according to the geometry shown in the figure, a 10 cm layer of steel mesh (6 Ø 15 × 15 cm) reinforced shotcrete is then sprayed on the floor of the excavation shaped in this way with the function of:

- shaping the future tunnel;
- distributing the future loads that will weigh on the crown.

At this point it is filled in and embanked with the same ground previously removed, after adding 3 – 6% of lime to increase its strength, until the whole of the crown of the future tunnel is covered in individually compacted 30 cm layers to a thickness of at least 3.5 m.

Bored tunnel excavation of the tunnel can now start.

The application of the method on the tunnels of the new Rome-Naples railway line

The application of new methods of driving tunnels always requires very thorough preliminary studies to assess whether it is genuinely feasible and compatible with the specific local conditions. In the case of the six tunnels on the new Rome-Naples railway line, these studies regarded primarily the soils found in the ground on the sites in question to test and optimise the effect of the treatment with lime with which it was intended to improve them.

The geology, the survey campaign and experimentation

From a geological viewpoint the route of the tunnels borders on the Roccamonfina volcano system and only marginally intersects the lava deposits, running almost entirely through eruption material. The various lithotypes consist of successions of stratified pyroclastites (tuffs), sub-horizontally positioned, with granulometry varying from sandy silt with inclusions of gravel to coarse sand, to give a morphology characterised by undulating hills with valley depressions originated by seasonal streams.

A geological survey campaign was conducted consisting of continuous core bore sampling and exploration trenches (one every 7–10 m) to assess the quality of the weathered material to improve before placing the A.G.O. The main geophysical characteristics of the material were measured in the laboratory in view of the subsequent study of the treatment with lime: unit weight and granulometry and modified ASSHTO compaction tests were also performed.

Samples of ground were then prepared mixed with lime in percentages varying from 3% to 5% using the optimum humidity found with the modified AASHTO test. They were then left to harden in a saturated water vapour environment.

Compressive strength tests were then performed after 3, 7 and 28 days of hardening (Figs. E.3 and E.4), which gave average results of higher than 200 t/m², 250 t/m², 320 t/m² (2 MPa, 2.5 MPa, 3.2 MPa) respectively. Compaction tests did not give results significantly different from those performed before the treatment.

Fig. E.3
Test piece subjected to mono-axial compression test

The optimum percentage of lime identified by the experiments was 3–4%.

Full scale field trials were also performed to check the results obtained in the laboratory. The results confirmed expectations.

Cure (days)	3 % lime mix			4 % lime mix			5 % lime mix		
	Test 1	Test 2	Average	Test 1	Test 2	Average	Test 1	Test 2	Average
3	200	200	**200**	180	190	**185**	100	110	**106**
7	240	230	**235**	250	250	**250**	160	170	**165**
28	340	300	**320**	340	320	**330**	370	380	**375**

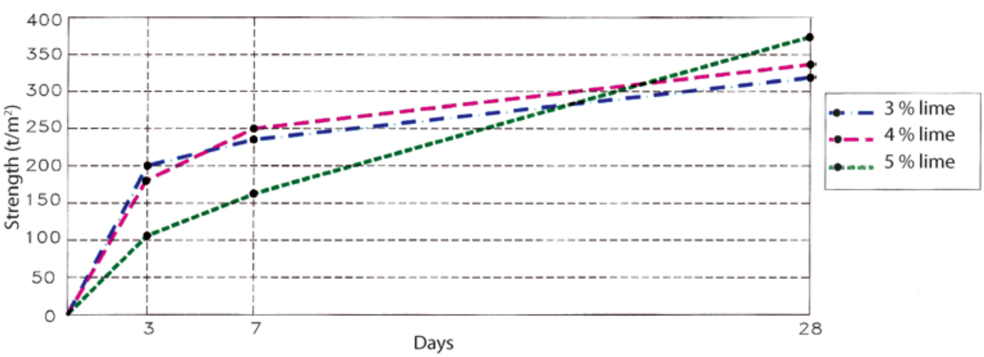


Fig. E.4

Statics tests

The feasibility of the new solution and the reliability of the statics were tested by means of finite element numerical analysis conducted in the non linear field using version 6.0 of the ADINA software application.

A typical cross section of the running tunnel was considered for this purpose (with final lining in r.c., 80 cm thick in the crown and 90 cm in the invert), constructed with isoparametric, plane strain, four node elements.

An elasto-plastic stress-strain response model according to Drucker-Prager was chosen for those representing both the natural and the treated ground, while a linear elastic model was chosen for those representing the preliminary and final linings.

As the analysis was intended to study the stress-strain behaviour of the entire ground-structure system at the different stages of construction and after commissioning, seven calculation 'times' were performed (see Fig. E.5) to model the succession of those stages and the implementation of stabilisation intervention during construction work as realistically as possible.

STAGES OF THE FINITE ELEMENT CALCULATION

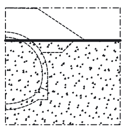
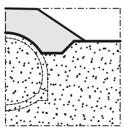
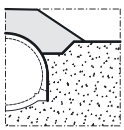
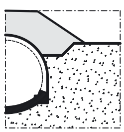
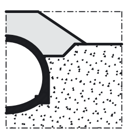
TIME	MODEL	DESCRIPTION
1		CALCULATION OF THE GEOSTATICS
2		IMPROVEMENT AND BACKFILL WITH PYROCLASTITES TREATED WITH LIME
3		THE TUNNEL IS DRIVEN FULL FACE AND THE PRELIMINARY LINING OF SHOTCRETE AND STEEL RIBS IS PLACED RELAXATION OF THE CORE 70%
4		THE TUNNEL INVERT AND KICKERS ARE CAST RELAXATION OF THE CORE: 80%
5	...	CURING OF THE TUNNEL INVERT RELAXATION OF THE CORE: 90%
6		THE TUNNEL ARCH IS CAST AND THE GEOMECHANICAL CHARACTERISTICS OF THE GROUND TREATED WITH LIME FALL RELAXATION OF THE CORE: 100%
7	...	SEISMIC ACTION

Fig. E.5

The progressive demolition of the face due to tunnel advance was simulated in the model by progressively reducing the strength and deformability characteristics of the elements representing the material excavated to zero.

The calculation parameters adopted for the natural and the treated ground are summarised in table E.I. Extremely low long term (service phase) values for the strength and deformability parameters of the treated ground were chosen to err on the side of safety, while 2.00 t/m³ was assumed for unit weight of both (natural ground and treated ground) was assumed. Finally an extra live load of 2.00 t/m² was considered on the summit of the model.

Table E.I

SOILS	γ (t/m³)	E (t/m²)	ν	c (t/m²)	φ (°)
Weathered surface ground	1.8	7,000	0.30	2.00	30
Pyroclastites	1.8	20,000	0.30	10.00	30
Lime treated Pyroclastites (*time* 2 – 5)	2.0	40,000	0.30	20.00	30
Lime treated Pyroclastites (*time* 6 – 7)	2.0	7,000	0.30	0	30

Results of the statics tests

The results of the calculation confirmed that bored tunnel full face advance was possible under the protection of the artificial ground treated with lime, by employing the geometry and construction methods illustrated.

From the viewpoint of deformation, they gave very low values for convergence (less than 2 mm), while the role played by the treatment in the crown of the tunnel before excavation started was very important for stress. Thanks to its arched shape the improved ground was subject to contained compressive stress action only (σ_{max} = 12.9 t/m² at *time* 5), which were appropriately channelled around the tunnel and transmitted to the natural ground on the sides of the tunnel (Fig. E.6).

Similarly the preliminary lining in shotcrete was found to be fully compressed ($\sigma_{average} \leq 210$ t/m²). The final lining in r.c. with the ring closed with a tunnel invert tested well even without reinforcement (which was placed anyway, in case of potential asymmetry or local anomalies) since it was subject to extremely low tensile stress (≤ 18 t/m²) in the crown and at the joint with the tunnel invert and to maximum compressive strength of less than 220 t/m².

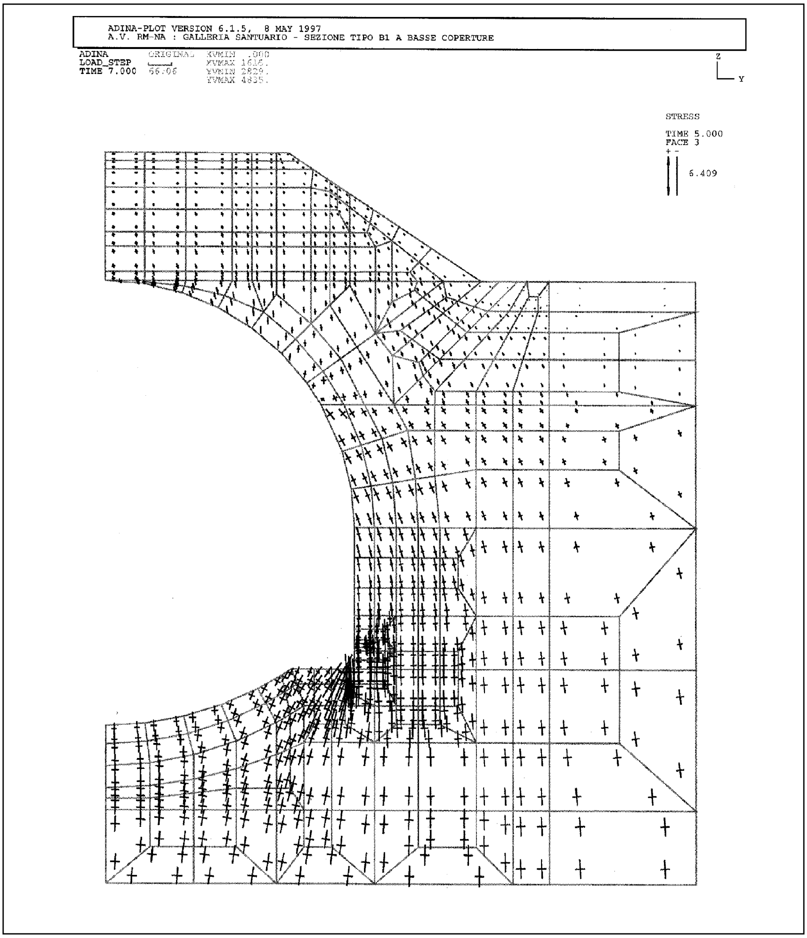


Fig. E.6 *High Speed Rome-Naples: Santuario Tunnel – Section type B1 for shallow overburdens*

Construction

Once it had been ascertained that the solution studied was feasible and that the statics were reliable, experimental implementation began following the procedures and phases already described and illustrated in figures E.2 and E.6.

Work started from the Picilli 2 tunnel where bored tunnel advance was performed under the protection of the A.G.O. for more than 250 m. The photographs in Figs. E.7 and E.8 illustrate some of the different stages of the work.

Two different procedures were employed on site to mix the tuff with lime:

- a plant with a hopper was used to store the lime (Fig. E.9) and the mix was prepared at the plant; it was then transported to the construction site, spread in layers of not more than 30 cm and rolled at optimum humidity, or
- the pozzolana spread and covered with an adequate quantity of lime was milled and then rolled as above.

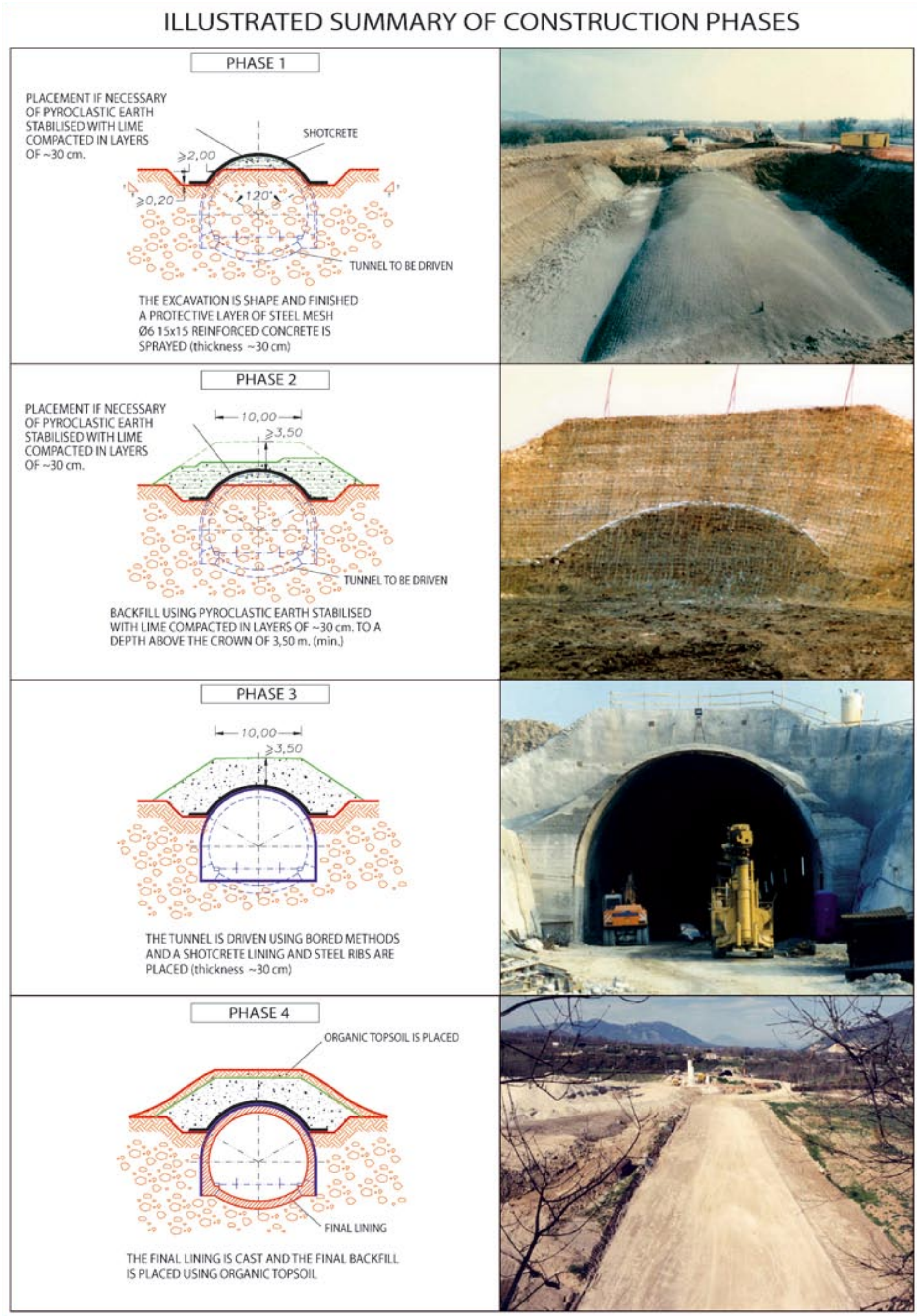

ILLUSTRATED SUMMARY OF CONSTRUCTION PHASES
PHASE 1
PLACEMENT IF NECESSARY OF PYROCLASTIC EARTH STABILISED WITH LIME COMPACTED IN LAYERS OF ~30 cm.
SHOTCRETE
≥2,00
≥0,20
120°
TUNNEL TO BE DRIVEN
THE EXCAVATION IS SHAPE AND FINISHED A PROTECTIVE LAYER OF STEEL MESH Ø6 15x15 REINFORCED CONCRETE IS SPRAYED (thickness ~30 cm)
PHASE 2
PLACEMENT IF NECESSARY OF PYROCLASTIC EARTH STABILISED WITH LIME COMPACTED IN LAYERS OF ~30 cm.
10,00
≥3,50
TUNNEL TO BE DRIVEN
BACKFILL USING PYROCLASTIC EARTH STABILISED WITH LIME COMPACTED IN LAYERS OF ~30 cm. TO A DEPTH ABOVE THE CROWN OF 3,50 m. (min.)
PHASE 3
10,00
≥3,50
THE TUNNEL IS DRIVEN USING BORED METHODS AND A SHOTCRETE LINING AND STEEL RIBS ARE PLACED (thickness ~30 cm)
PHASE 4
ORGANIC TOPSOIL IS PLACED
FINAL LINING
THE FINAL LINING IS CAST AND THE FINAL BACKFILL IS PLACED USING ORGANIC TOPSOIL

Fig. E.7

Fig. E.8 *Bored tunnel advance: shaping the crown under the protection of the A.G.O.*

Fig. E.9 *Plant for mixing the pyroclastites ground with lime*

The work continued smoothly for the whole of the 251 m specified without any problems at all arising. It was therefore decided to extend the new method to the other tunnels of the lot with similar problems.

Table E.II contains data on the application of the A.G.O. method. It can be seen that a total of 1,346 metres of tunnel have been driven to advantage using bored tunnel methods since the first experimentation thanks to this system.

Table E.II

TUNNEL	TOTAL LENGTH OF THE TUNNEL [m]	LENGTH OF TUNNEL DRIVEN UNDER A.G.O. [m]
Piccilli 1 (State Railway, High Speed Rome-Naples)	907	58
Piccilli 2 (State Railway, High Speed Rome-Naples)	485	251
Castagne (State Railway, High Speed Rome-Naples)	289	73
Santuario (State Railway, High Speed Rome-Naples)	322	80
Caianello (State Railway, High Speed Rome-Naples)	832	363
Sadurano (State Railway, High Speed Bologna-Florence)	3767	68
Borgo Rinzelli (State Railway, High Speed Bologna-Florence)	528	73
Morticine (State Railway, High Speed Bologna-Florence)	654	380
Total length driver under A.G.O.		**1346**

Monitoring and measurements during construction

Design of the A.G.O. required a series of measurements to be taken during construction to ensure that its performance matched the design predictions. More specifically:

- for the pozzolana-lime mix:
  - average compressive strength after 7 days of hardening must not be less than 80 t/m²;
  - the *in situ* density achieved must not fall below 95% of the maximum density achieved in the laboratory;
- for the shotcrete:
  - average compressive strength after 48 hours must not be less than 1300 t/m²;
  - after 28 days it must not be less than 2000 t/m².

In the case of the Piccilli 2 tunnel samples were taken systematically during construction, which always gave results above the minimum specified limits when tested. Average strength of 300 t/m² was measured for the pozzolana-lime mix, similar to the results obtained in the laboratory.

In addition to the tests for quality control of the materials, various load measurements were taken by positioning load cells at the level of the removed ground at around the height of the springline of the future tunnel and also at the foot of the steel ribs.

Piccilli 1 Tunnel: treatment of the ground with lime

Piccilli 1 Tunnel: shaping the extrados of the future tunnel

The former gave average readings of $2\,t/m^2$, taken, however, before driving the bored tunnel and therefore of little significance, while the latter gave average readings of $30\,t/m^2$.

Tunnel convergence was also monitored systematically: the average readings obtained from the five-nail stations fluctuated between 1 and 2 mm, and were therefore very close to those predicted by the FEM calculations.

Final considerations

The construction of artificial tunnels for sections of tunnel without the necessary overburden above the future crown of a tunnel needed for bored tunnel technology, inevitably requires deep cuts to be made in the slopes to be crossed with consequent problems of safety, environmental impact, etc.

The alternative solution, illustrated here, using A.G.O. allows bored tunnel technology to be employed, thereby avoiding all the problems connected with the construction of artificial tunnels.

This new solution, initially studied and implemented for the construction of tunnels on the new Rome-Naples high speed railway line, was found to be extremely practical, safe and also advantageous economically (Fig. E.10), as well as from an environmental and landscape viewpoint. It has been applied with considerable success on various other tunnels suffering from similar problems due to insufficient overburdens.

COMPARISON OF CONSTRUCTION COSTS AND TIMES FOR BORED TUNNELS DRIVEN THROUGH ARTIFICIAL GROUND OVERBURDEN (A.G.O.) AND FOR THE SAME TUNNELS CONSTRUCTED ARTIFICIALLY

WORK*	ARTIFICIAL TUNNEL €/m	BORED TUNNEL UNDER A.G.O. €/m	DIFFERENCE
EARTH REMOVAL, GROUND IMPROVEMENT, TUNNEL EXCAVATION	2,464.00	4,450.00	-1,986.00
BACKFILL	915.00	0.00	915.00
LINING OF CROWN, KICKERS AND SIDEWALLS	3,421.00	2,383.00	1,038.00
TUNNEL INVERT	1,664.00	790.00	874.00
TOTALS	8,464.00	7,623.00	841.00

*ONLY THOSE ITEMS OF THE TWO CONSTRUCTION METHODS WHICH ARE DIFFERENT

TIME EMPLOYED FOR THE CAIANELLO TUNNEL (FOR 365 m) WITH THE SAME MEANS EMPLOYED

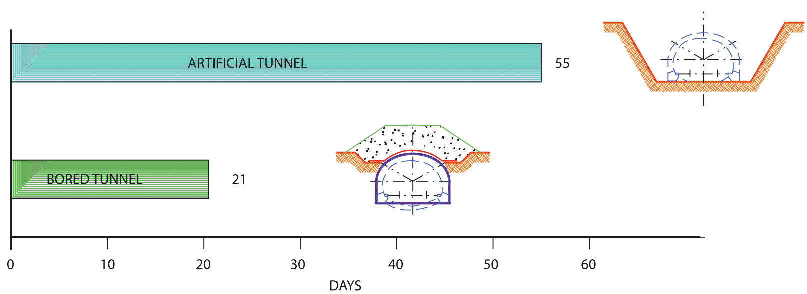


Fig. E.10

Sadurano Tunnel (high speed/capacity Bologna-Florence railway line): view of the 'sprayed' crown of the future bored tunnel

Piccilli 2 Tunnel: backfill with stabilised ground and compaction

Piccilli Tunnel 1 (above) and 2 (below): two stages in the construction of the A.G.O.

Portals in difficult grounds: aerial view of a portal constructed using vertical jet-grouting technology for a tunnel on the Sibari-Cosenza railway line (1985, ground: clay)

Portals in difficult ground

The construction of tunnel portals often involves solving problems that are closely connected with the morphology of the slope to be entered, with the existence of nearby constructions, with the geometry of the structures to be constructed and the type of material involved. The preparation of a portal site and of the wall to be excavated, very frequently requires substantial cuttings which are of no particular concern when working in rock, but are very problematic when working in soft soils, especially if they are loose.

If the decompression caused by excavation in ground with little cohesion is not adequately confined, there is a risk that it will easily and rapidly propagate into the medium with serious effects for the whole slope. The only way the entrance to the portal of a tunnel can be excavated without decompressing the ground is clearly by creating a structure to confine (or better preconfine) the ground in advance ahead of the future excavation which is capable of conserving the existing natural equilibriums. Until just a few years ago, the only systems available for achieving this goal were to place the following structures at the entrance of the bored tunnel:

- large diameter pile walls, anchored if necessary;
- “Berlin” or soldier pile walls;
- r.c. diaphragm walls.

The construction of large diameter piles on slopes tending towards instability is, however, often difficult, if not impossible, because, it is often the case that the morphology of the slopes does not allow the use of the heavy operating machinery required.

On the other hand the lability of Berlin type structures themselves, which rely to a large extent on anchors which reach into stable zones of the ground for their effectiveness, mean that these systems are not always sufficiently reliable.

Not even r.c. diaphragm walls are sufficient for the most delicate situations: earth removal and the introduction of water have the effect of reducing the shear strength of the ground with consequences for which there is no remedy in some situations.

These problems were then made worse because the lack of adequate techniques to improve the ground in advance and of suitable operating systems, required bored tunnels to be driven with overburdens measurable in terms of tunnel radii. This required huge portions of the ground to be removed with the risk of triggering ground decompression that is very difficult to confine, as has been demonstrated by the

A very deep cut is made into the slope for a tunnel portal in semi-cohesive soils using conventional systems (anchored Berlin wall; overburden: twice the diameter of the tunnel)

The same portal as the one above, after it had collapsed because of the decompression induced in the slope

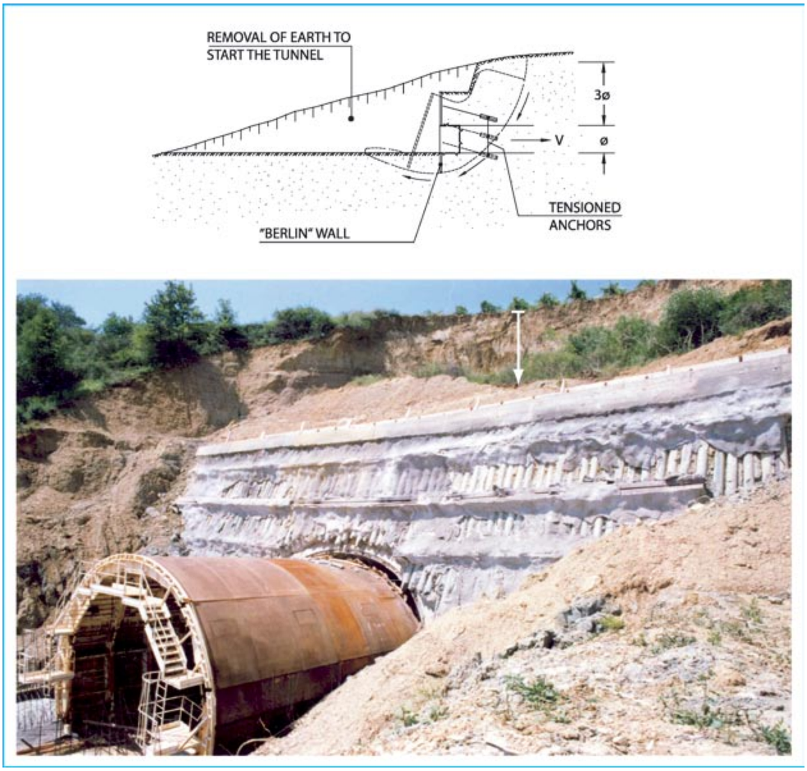


Fig. F.1

many cases of failure seen in the past (see Fig. F.1 and the photographs on the page opposite).

The availability of new jet-grouting systems for improving ground, introduced in Italy around twenty years ago, allowed the author to invent and experiment an innovative solution to create preconfinement structures in loose or poorly cohesive soils with properties that would overcome the problems that have been described.

The basic concepts are explained below, together with a few of the most significant case histories.

Shells of improved ground formed by means of jet-grouting

The idea consists of creating a confinement shell, before excavation for the portal commences, consisting of rows of columns of ground improved by means of jet-grouting, which geometrically enfold the section of artificial tunnel (see Figs. F.2 and F.3).

The magnitude and distribution of the treatment are decided on the basis of each specific operating and geotechnical context. A top beam in r.c. joins the tops of the columns to help make it a single rigid structure.

TUNNEL PORTALS WITH SHELLS OF IMPROVED GROUND BY MEANS OF VERTICAL JET-GROUTING

Construction stages

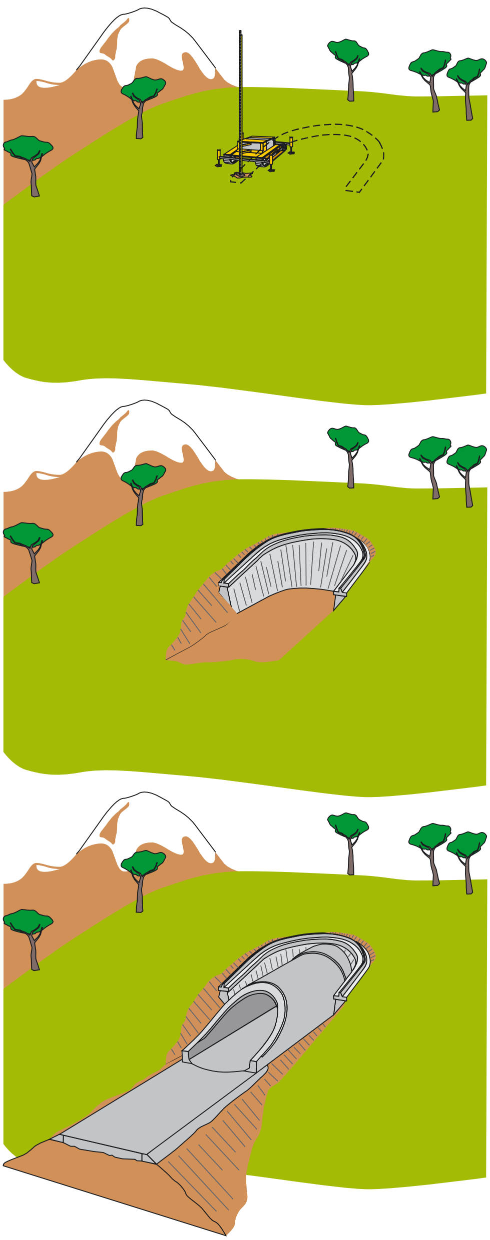


Fig. F.2

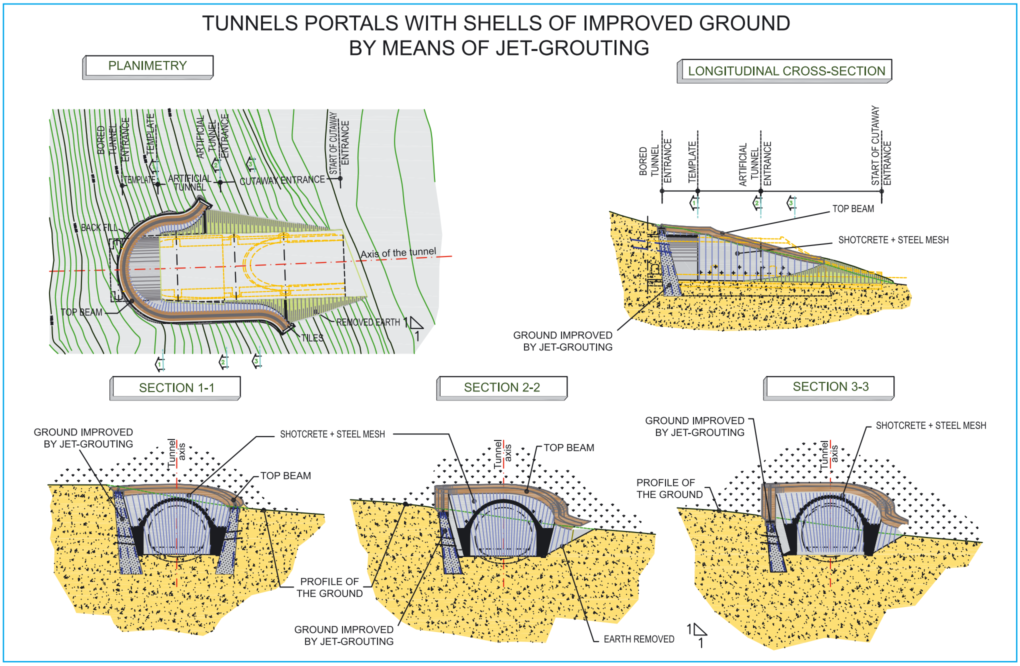


Fig. F.3

Once the earth has been removed for the entrance to the portal, the work is completed with:

- a layer of shotcrete sprayed on the whole surface of the exposed wall;
- drainage pipes placed sub-horizontally through the improved ground to prevent the formation of heads of water behind it.

The structure thus built functions by means of the sub-horizontal arches and is subject mainly to compressive and shear stress.

The creation of shells of this type is strictly dependent on the subsequent tunnel being driven using horizontal jet-grouting methods. Thanks to the characteristics of this ground preconfinement technology, which requires only minimum overburdens, bored tunnels can be driven with extremely low cover, with many important advantages, including the way it fits unobtrusively into the environment (see the photographs in this appendix).

Applications

The first time that sub-vertical jet-grouting to preconfine excavation was experimented was in 1980 at Sesto San Giovanni (Milan), in an area destined to contain a ventilation chamber 9.80 m deep on a section of line 1 of the Milan metro then under construction. The first application for the construction of tunnel portals occurred five years later for the T1 portal on the Pontebba side of the S. Leopoldo tunnel on the Pontebba-Tarvisio. railway line.

A shell of columns of ground improved by vertical jet-grouting constructed for the portal of the S. Leopoldo tunnel under the motorway embankment

Fig. F.4 *The portal of the same tunnel during excavation works*

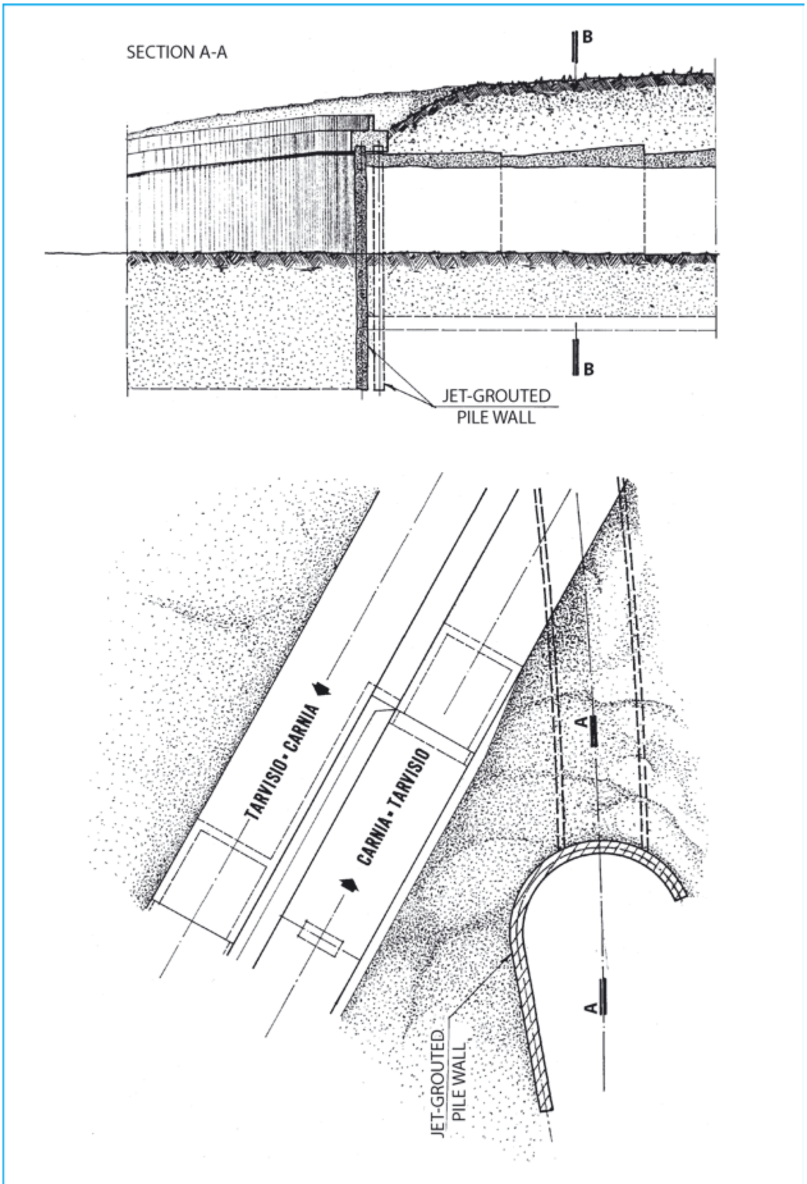
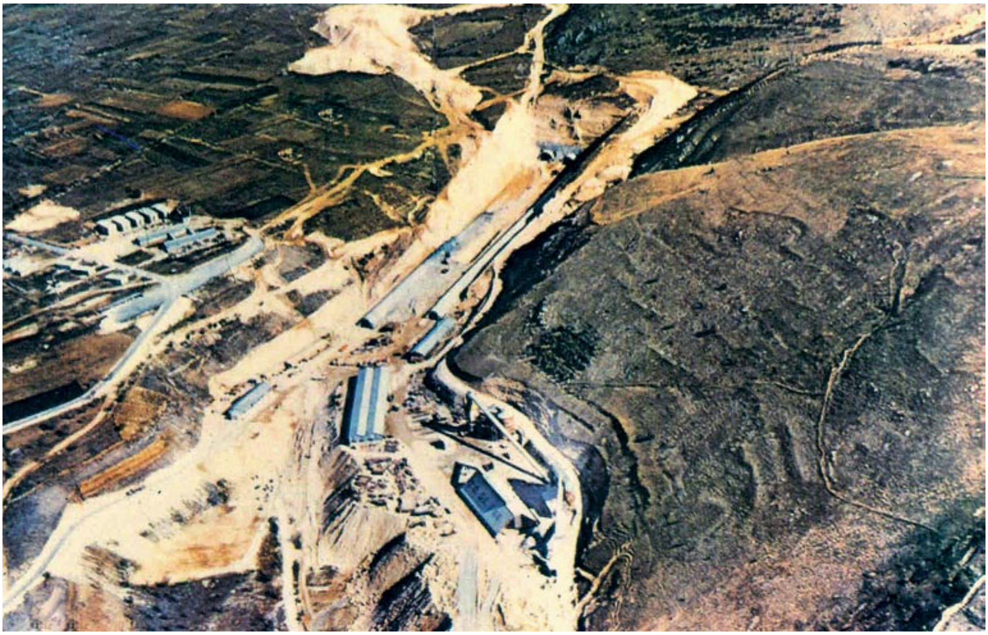


Fig. F.5 *Plan view and vertical cross section of the T1 portal of the S. Leopoldo tunnel*

The design problem consisted of the simultaneous presence of both the embankment for the Carnia-Tarvisio motorway under which the tunnel was to pass and the shoulder of the motorway viaduct almost immediately on top of the point of the chainage for the portal, which meant that the tunnel had to advance without causing deformation in the ground (see Figs. F.4 and F.5). Since this seemed impossible using conventional techniques, it was decided to employ jet-grouting technology.



Aerial view of the construction site for the portal on the Assergi side of the Gran Sasso Tunnel

The Gran Sasso Tunnel, portal on the Assergi side: the depth of the cut made in the slope using conventional methods for the portal of the tunnel certainly makes an impression

The study of the statics, conducted from the outset with the assistance of computers on a finite element model, confirmed the feasibility of the intervention and provided useful indications for the final fine tuning of the design in order to obtain a structure stressed mainly by compressive action. It was therefore decided to create a shell with three rows of sub-vertical jet-grouted columns arranged according to the geometry shown in Fig. F.5 and to then employ horizontal jet-grouting to start and drive the tunnel. It was possible in this manner to start driving a bored tunnel with an overburden of just two metres.

The works were performed without any particular difficulties and without any damage at all to either the embankment or to the shoulder of the motorway viaduct. The success achieved gave rise to numerous new applications of the same type performed in terrains of various kinds.

These included the portal constructed for the S. Elia tunnel, illustrated below (lot 33 *bis* on the Messina-Palermo motorway) positioned close to the provincial Cefalù-Gibilmanna road, which cut across the West slope of the S. Elia valley in conditions of precarious stability, given the non cohesive nature of the soil.

Geological and geotechnical picture

The morphology of the area affected by excavation was characterised by the presence of slopes of varying steepness depending on the lithology of the outcropping terrain and the structural conditions. The overall picture was typical of a young, little developed morphology and this was confirmed by the almost rectilinear geometry of the main water courses and by the almost total absence of secondary incisions.

The zone was investigated by carrying out an intense geological survey campaign in which the following were performed:

- 3 boreholes (S2, S3, S4) with a total of 56,5 m drilled;
- 60 standard penetration tests;
- 4 tests with an excavator down to 5 m from ground level (Sa3, Sa4, Sa5, Sa6);
- 1 sample was taken in the proximity of the trench Sa3 and was reassembled and reconstituted in the laboratory (C4).

Thorough examination of all the documentation collected established the following:

- the portal zone of the tunnel was located in a substantial layer of detritus, 12–19 m thick, consisting of light brown coloured silty-clayey soils;
- this layer included a high percentage of sharp grey-greenish arenaceous elements with dimensions of between 0.1 and 1 m chaotically dispersed;

Pianoro Tunnel (new high speed/capacity railway line between Bologna and Florence): the progression of the works for the South portal built in semi-cohesive ground using jet-grouting technology

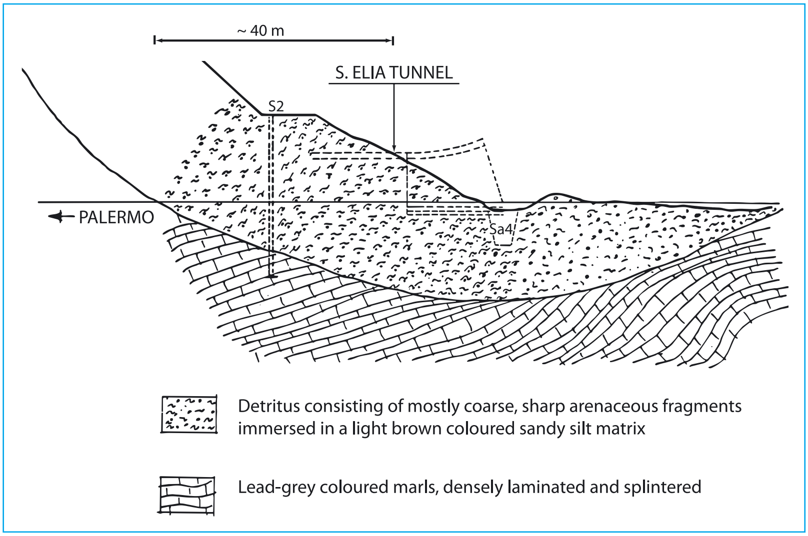


Fig. F.6

- as is shown in the geological cross section in Fig. F.6, it was resting on a densely stratified and fractured substrate of clayey marls belonging to the Polizzi formation;
- a free water table located at −3.14 m from ground level, but subject to variations in level as a function of precipitation on the slope;
- as concerns granulometry, the clayey fraction of the detritus was around 20%.

The presence in the ground of some tensile fractures also revealed the state of decompression of the existing material, a symptom of precarious stability.

From a geotechnical viewpoint the standard penetration tests performed in the zone surveyed gave average values of more than 30 blows per foot. This result seemed, however, excessively optimistic when compared with the other data acquired and was certainly influenced by the abundant presence of coarse rocky material dispersed in the matrix.

The following parameters were therefore attributed on initial analysis to the soils in question for the necessary design and verification calculations:

• unit weight	1.8–2 t/m^3 (18.0–20.0 kN/m^3)
• effective angle of friction	30°
• effective cohesion	0 Mpa

S. Elia Tunnel (Messina-Palermo motorway): portal on the Messina side under construction under the provincial Cefalù-Gibilmanna road

MESSINA - PALERMO MOTORWAY
S. ELIA TUNNEL - PORTAL ON THE MESSINA SIDE

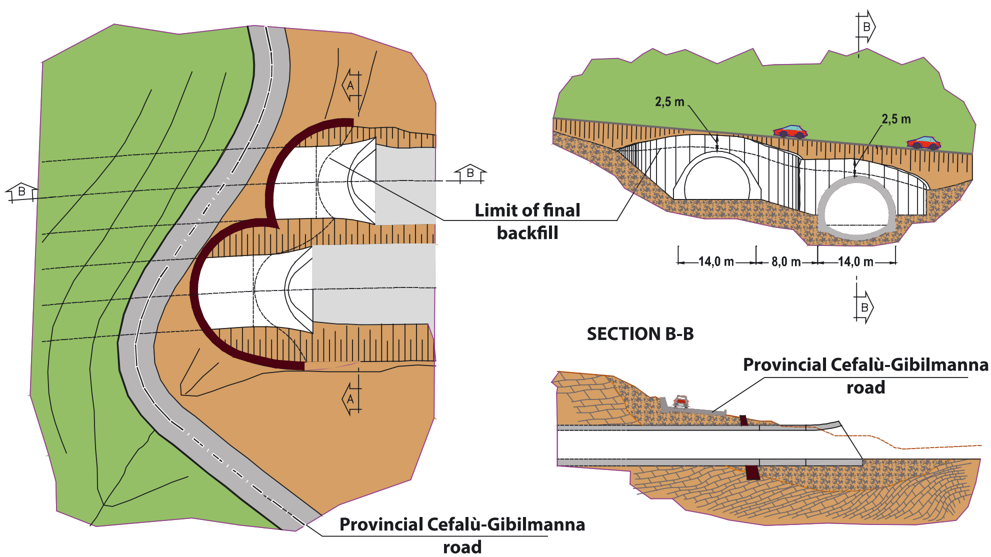


Fig. F.7

Tunnel No. 2 (Sibari-Cosenza railway line): the use of jet-grouting technology enables tunnel portals to be built in difficult grounds with full respect for natural equilibriums and for the landscape

The same portal as above: notice the extremely shallow overburden of the tunnel and the way it fits perfectly into the landscape of the environment

Malenchini Tunnel (Livorno-Civitavecchia motorway): North portal built by first placing a shell of ground improved by means of jet-grouting

The same portal as above, as it is now

General stability of the slope

As shown in Fig. F.6, the tunnel portals were to lie in the detritus deposits for a length of approximately 40 m from the start of the bored tunnels. Given the heterogeneous and non cohesive nature of the ground to be excavated and tunnelled, it was clearly going to be rather difficult both because of the presence of the provincial road over which the tunnels were to pass precisely in the vicinity of the portals and also because any land slip/slide would have immediately affected the stability of a small villa located just ahead of the portals.

As can be seen from the cross section through the axis of the tunnel shown in Fig. F.7, the overburden below the provincial road did not exceed 4 m, while the distance from the edge of it was around 7 m.

The original design solved the problem by moving the road further ahead of the portals and then making a deep cut into the slope.

However after direct *in situ* observation and examination of the results of the geological survey, it was decided in agreement with the project management to discard this solution. The reasons were as follows:

- moving the provincial road farther up the slope would have required cutting substantially into the layer of detritus, which was already in a precarious condition and would therefore have required adequate works to improve the ground and protect the excavation;
- deep cuts into the slope to construct the artificial tunnels, as specified in the design, would have caused dangerous decompressions in the layer of detritus and would have drawn water into the excavation. This would have resulted in serious danger for the stability of the residential structures above;
- it was impossible to make cuts with escarpments with a gradient of 1:1 even if only temporary, as would have been required with the presence of nearby buildings;
- the length of the construction times, without considering the risks of triggering uncontainable deformation phenomena in the slope.

In consideration of the reasons listed above, changes had to be made to construction plans to enable the tunnel portals to be built without dangerously affecting the pre-existing natural equilibriums given the already precarious stability of the slope. Obviously this could only be achieved by not starting excavation until the future perimeter walls had been stabilised. Systematic ground improvement by jet-grouting it around the portals of the bored tunnel and around the foundations and portals of the artificial tunnels seemed to be the best means of achieving this and it would also eliminate the need to move the provincial road and halt traffic on it.

The geometry of the ground improvement

Following similar procedures to those already successfully employed experimentally for the portal on the Pontebba side of the San Leopoldo tunnel, a systematic campaign of ground improvement was performed along three main lines (Fig. F.8):

- for the faces for driving the two bores: ground improvement and reinforcement by creating a pile wall of vertical jet-grouted columns, which would confine the excavation required to prepare the tunnel access portals and fully prevent any decompression of the slope alongside the provincial road;
- for the perimeter of the two bores: ground improvement using horizontal jet-grouting in the band of ground around the extrados of the theoretical profile of the tunnel in order to eliminate convergence of the future tunnel and, as a consequence, prevent even the slightest settlement of the structure of the road;
- for the foundations of the artificial tunnels and portals: vertical jet-grouting treatment with a column approximately every 3 m², designed to increase the shear strength of the ground locally to eliminate the need to dig deep foundations for the structures.

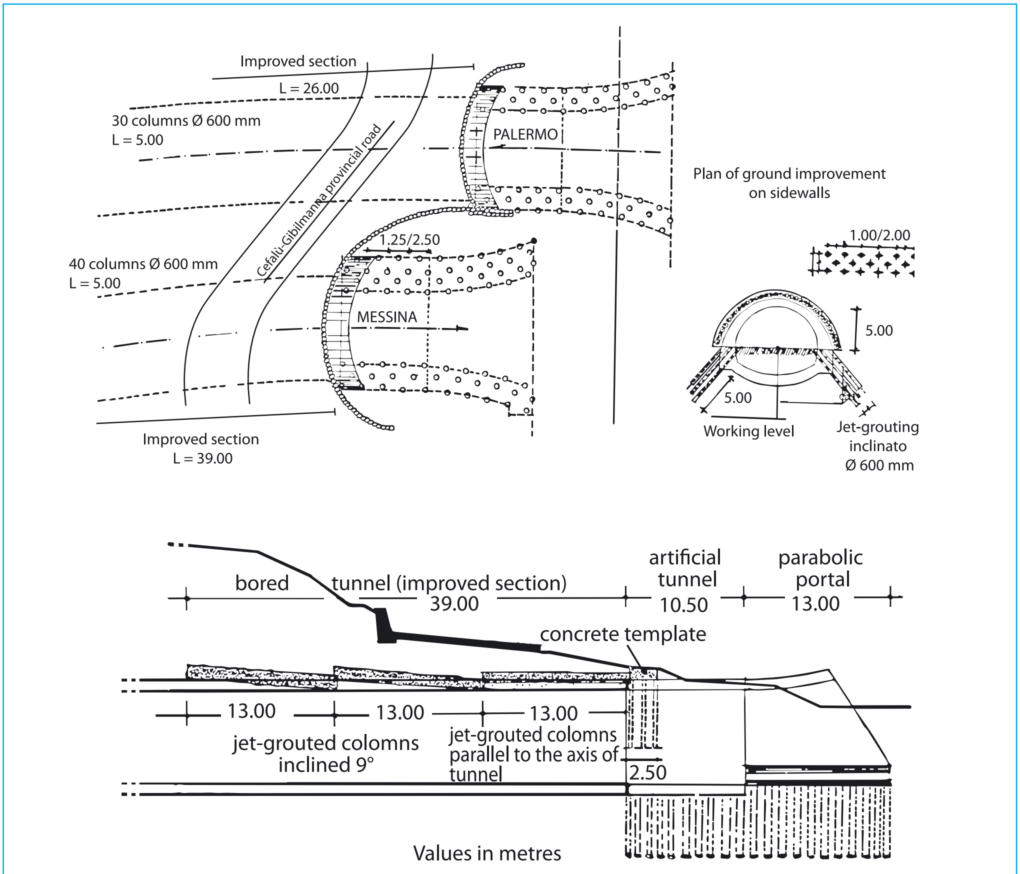


Fig. F.8

Ground improvement and reinforcement

Determination of the operating parameters

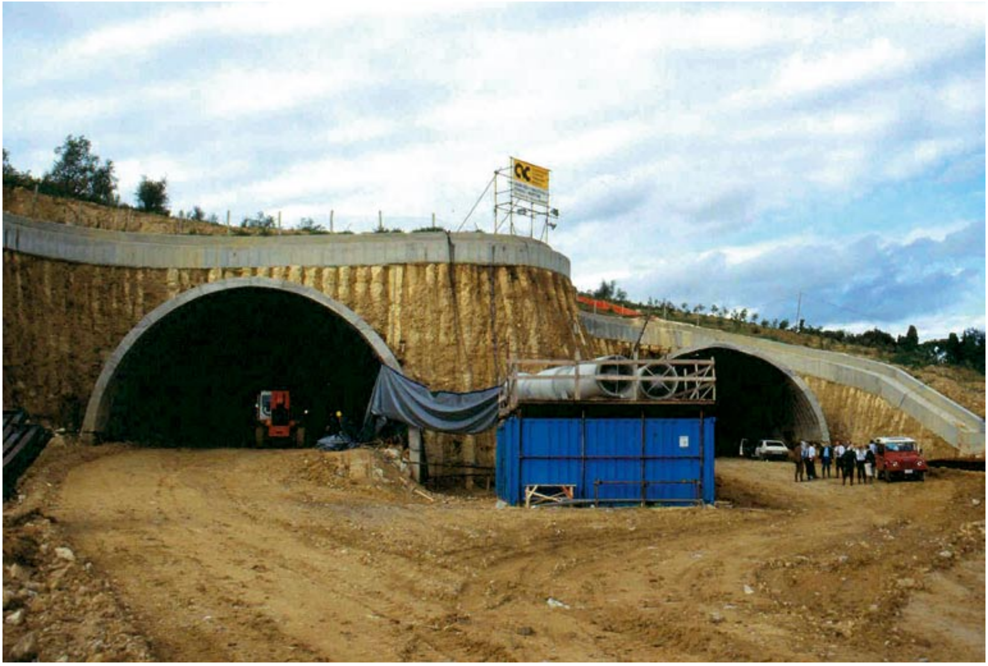
It is absolutely critical to the successful outcome of intervention when using jet-grouting to improve ground to use operating parameters that are appropriate to the geotechnical context and to obtain the desired results. It was therefore decided to perform six test treatments with varying combinations of the dosage of the mix, the injection steps, pressure and injection speed. It was decided that the zone most appropriate for performing the tests was the portal on the Messina side of the escarpment between the two alignments at the depth of the working level of the higher bore and that is at a depth of approximately 5.00 m from the intrados of the lining of the bored tunnel. The operational procedures for implementing the test columns, positioned geometrically as indicated in Fig. F.9, are summarised in table F.I below.

Column No.	Depth m.	Pressure Atm.	Quantity		Ratio H_2O/cm^3 cement mix	Extraction time in seconds per 4 cm	Total volume of back-flow	Notes
			Cement quintal/m	Mixture L./m.				
1		300	2.5	333		15	450	Fissures open parallel to the escarpment
2		500	2.5	333		11	700	It is noticed that fissures open parallel to the escarpment during injections
3	5.00	400	1.5	200	100 $\gamma = 1.5$	8	200	Drilling water is absorbed into the ground
4		400	1.5	200		8	300	Drilling water flows out along all the hole
5		400	1.5	200		8	nil	Drilling water is absorbed
6		400	2.5	333		13	500	Fissures open parallel to the escarpment

Table F.I *Operational procedures for jet-grouting column tests*

Selection of optimum parameters

The improved columns of ground were laid bare after they had hardened for three days. The random presence of boulders was found which, combined with the dishomogeneity of the ground, led to continuous zones of ground in which the cement mixture had mixed with and consolidated the clay component and wrapped around the blocks present in it to form blades of hydrofracturing (*claquage*), which extended for more than a metre from the axis of the columns themselves. The following was decided on the basis of the examination of the columns:



Malenchini Tunnel (Livorno-Civitavecchia motorway): South portal built by first placing a shell of ground improved by means of jet-grouting

The same portal as above, as it is now

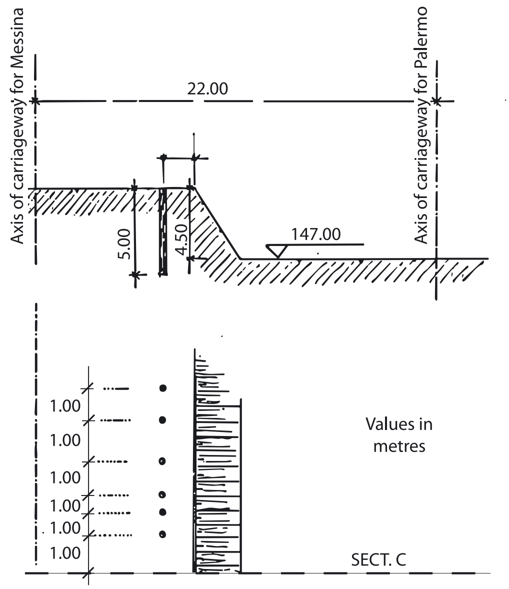


Fig. F.9

- to adopt the operating parameters used for columns 3, 4 and 5 for the ground treatment;
- to reduce the distance between centres of the vertical columns to 0.45 m and to set that distance at 0.40 m for the horizontal columns to take account of the dishomogeneity of the ground, the presence of erratic boulders and the possibility of potentially large deviations during drilling.

Geotechnical results

Table F.II below reports the main results of the comparative tests performed on soils before and after ground improvement. The values for c (cohesion) and φ (internal angle of friction) measured on samples of natural ground were obtained using undrained anisotropic triaxial tests on samples from the preliminary core drilling, S2 and the trench Sa4 (see Fig. F.6) and from samples taken during the preparation of the portals, where only one sample was undisturbed.

NATURAL GROUND			IMROVED GROUND		
Sample No.	c [Mpa]	φ [°]	Sample No.	c [Mpa]	φ [°]
21	0.015	17	31	1.2	39
22	0.02	16	32	1.5	41
23	0.019	17	33	1.3	39
24	0.02	18	34	1.5	40
25	0.016	15	35	1.2	38

Table F.II

View of the portal constructed using a shell of ground improved by means of vertical jet-grouting under the railway station of the city of Campinas (Brazil)

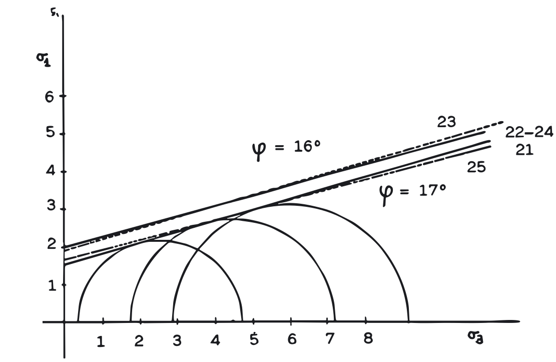


Fig. F.10
Undrained anisotropic triaxial cell test (ground not improved)

The samples of improved ground, however, were taken as cubes and then shaped into cores with a diameter of 50.4 mm and length of 100.8 mm in the laboratory. The triaxial tests on the latter were performed on a 70 MPa Hoek cell.

Each sample was composed of three test pieces, each of which was given a σ_3 which was a multiple of the previous one to give three points on the same straight line (see Figs. F.10, F.11). The piezometric pressure was calculated analytically for the first group of samples, but not for the triaxial Hoek cell test on the second group.

The outcome of the ground improvement was considered satisfactory given the results of these tests, which registered an increase in the average cohesion value from 0.018 to 1.34 MPa and in the average internal angle of friction from around 17° to around 40°. Three direct shear tests (Hoek type) performed on the improved ground confirmed this opinion with average values of 1,12 MPa for cohesion and 36° for the internal angle of friction.

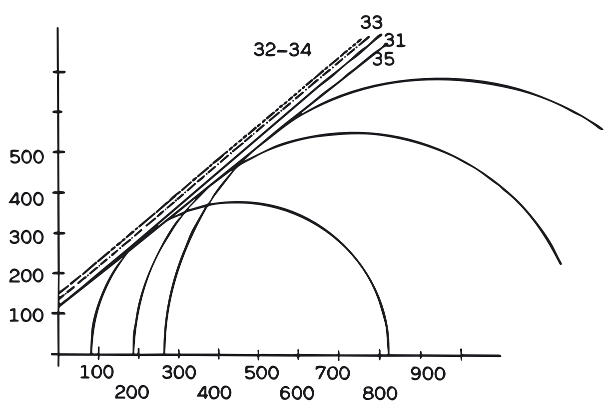
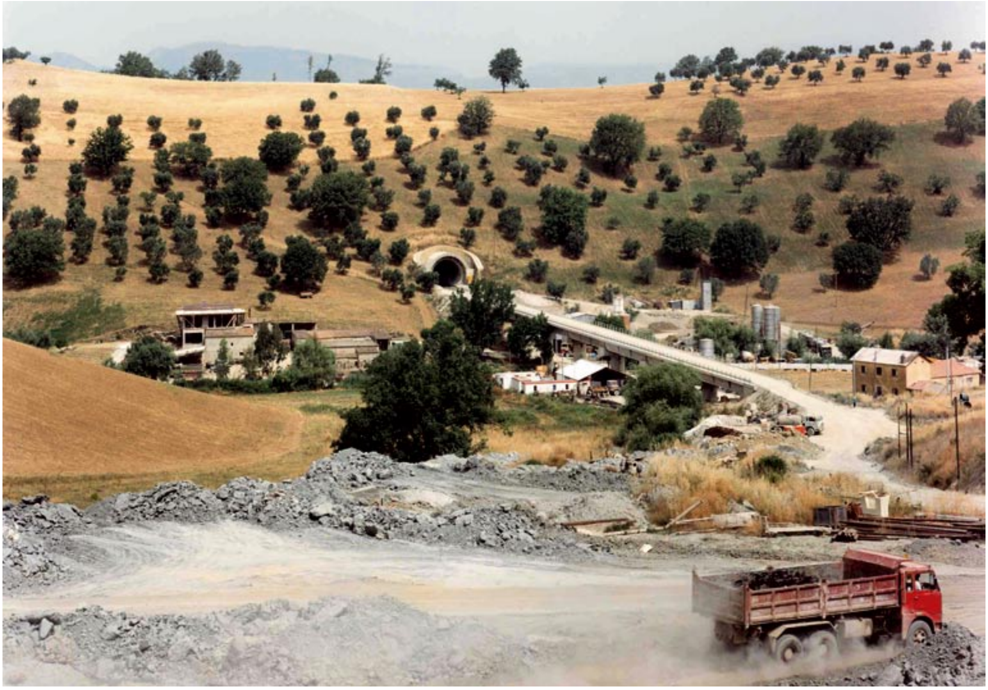


Fig. F.11
Triaxial Hoek cell test (improved ground)



Tunnel No.1 (Sibari-Cosenza railway line): jet-grouting technology enabled tunnel portals to be built without damage to the environment and landscape even in grounds with the poorest geotechnical characteristics

The same portal as above, seen from the front

Technical and operational considerations

Given the satisfactory results and having taken account of the full scale tests in the field, work commenced on the construction of the portal according to the design that has been illustrated.

In actual practice the work to improve the ground before starting to drive the tunnel was performed in the following six distinct stages starting with the lower bore and then on the upper bore, alternating between the two (Fig. F.7).

Stage 1: removal of the earth from the approach after having first stabilised the future walls of the cut with vertical jet-grouting.

Stage 2: the creation of a ring of drainage pipes, if necessary, and horizontal jet-grouting treatment in the crown of the future tunnel for more than 13 m ahead of the face. The telescoped horizontal columns of improved ground were performed with a distance between centres of 40–45 cm in the crown and of 50 cm in the zone of the springline.

Stage 3: half face advance[1], for approximately 10 m, positioning of the steel ribs and immediate placing of the steel mesh reinforced concrete for a thickness of approximately 10–15 cm.

Stage 4: a new series of drainage pipes are created (if necessary) from the new face and another section of sub horizontal columns of treated ground is created (13 m approx.), positioned with a distance between centres of 45–50 cm around the whole perimeter of the tunnel with a maximum inclination of 9% in the crown with respect to the axis of the tunnel; ground treatment is performed for the sidewalls of the rear section already excavated.

Stage 5: excavation of the bench in the section already fully treated with ground improvement and the application of shotcrete on the walls; excavation for the tunnel invert in the section of tunnel already open and subsequent casting of the sidewalls and the tunnel invert.

Stage 6: the final lining is cast in concrete reinforced with INP 200 steel ribs.

1 It is interesting to note that in the first tunnels driven using the new jet-grouting technology, tunnel advance was generally half face, while today, with much greater familiarity with this technology and as has been affirmed many times in this book, full face advance is always preferred, naturally using adequate designs and operational procedures. The advantages are considerable in terms of safety, production rates and the long and short term stability of the tunnels.

Colli Albani Tunnel (new Rome-Naples high speed railway line): aerial view of the works for the construction of the South portal of the tunnel

Apart from the reasonable certainty of obtaining good technical results from the use of jet-grouting for ground improvement, one important factor for the contractor was the relative fast construction times and the fact that it interfered little with construction schedules, favoured undoubtedly by the possibility of working on two adjacent bores at the same time. As concerns the latter aspect, it was objectively imperative to create an arch effect in advance as it was indispensable to the stability of the excavation and to the absence of even the slightest convergence. From this viewpoint:

- the main function of the band of jet-grout improved ground is not only to prevent immediate loosening of the ground, which would otherwise have direct effects on the surface, with serious damage to the structure of the road and the building above. but also to distribute asymmetric thrusts which manifest in the long term in soils like those of the S. Elia tunnel, redistributing them on the final lining which therefore need not be so thick. Given the reduced overburden between the crown of the future tunnel and the provincial road, as well as the permanent loads and live loads due to the passage of vehicles, the band of improved ground around the extrados of the S. Elia tunnel could be expected to be subject to compressive stress of around 6.00 MPa and tangential stresses of around 0.30 MPa, values compatible, temporarily, with the strength of the ground treated. Radial convergence measurable in millimetres was therefore to be expected given those stresses;
- thanks to its excellent strength and deformability properties, the layer of steel mesh reinforced shotcrete placed in direct contact with the treated ground acts as a protective mantle capable of developing confinement action of greater than 0.20 Mpa and it therefore increases the strength of the jet-grouted columns considerably. It also plays an important role as an element of transition in the overall economy of the intervention and of intermediate rigidity between the improved ground and the final lining: in the event of an earth quake, the wave of the shear stress on the final lining would be cushioned greatly so that it could guarantee the integrity of the tunnel even if immersed in non cohesive material;
- finally the task of the concrete structure reinforced with steel ribs and closed with the tunnel invert is to prevent long term settlement, which might result not only from viscous phenomena, but also from heavily asymmetrical individual thrusts, which are not improbable in non cohesive soils with extremely heterogeneous granulometry like those of the S. Elia tunnel, which can also be easily affected by external meteorological events, given the very shallow overburdens.

Vertical jet-grouting for a portal shell

Final considerations

Experiences to-date in the application of vertical jet-grouting to solve the problems of constructing tunnel portals in soft grounds have demonstrated that it can be used to start tunnels in slopes consisting of non cohesive or poorly cohesive soils without causing practically any movement in the surrounding ground.

When used in conjunction with horizontal jet-grouting, it allows bored tunnel excavation to start under extremely shallow overburdens of a little more than one metre, thereby minimising the problems of harmonising new constructions set in existing environments. Other advantages can also be gained from the application of these methods:

- great flexibility in adapting to any morphological situation;
- fast construction times;
- competitive costs.

Although jet-grouting technology does require considerable expenditure, it nevertheless guarantees an excellent ratio between costs and the quality and safety of the results (obviously provided the necessary morphological conditions exist).

Quite apart from any considerations concerning traffic, there was also a modest saving on costs in the construction of the portal for the S. Elia tunnel, as is shown by the summary figures given in table F.III below.

• Ground improvement work using jet-grouting to pass under the provincial road	Lump sum	610	
• Extension of the embankment wall	m 20	25	
• Additional bored tunnel to replace the artificial tunnel	m 39	275 910	910 +
• Cost of artificial tunnels eliminated	m 59	575	
• Savings for not having to change the route of the provincial road	a corpo	156	
• Savings for not having to improve the ground of the slope	a corpo	219	
• Savings for not having to expropriate property	a corpo	69 1019	1019 –
Total savings			**109**

Table F.III
Comparison of construction costs for the two alternatives (in millions of lire)

Preparation for casting the upper r.c. beam over a shell of ground improved by vertical jet-grouting

Detail of the wall of a shell of columns of ground improved by means of vertical jet-grouting

Pianoro Tunnel (new high speed/capacity railway line between Bologna and Florence): work in progress on the southern portal of the tunnel

S. Giovanni Tunnel (Catanzaro ring road East): tunnel portal with diameter 14 m approx., excavation under urban buildings

Widening of tunnels without interrupting traffic: Milan-Rome motorway, Nazzano tunnel, placing an arch of the lining constructed using prefabricated segments (2005)

Widening road, motorway and railway tunnels without interrupting use

As traffic increases, there is an increasing need to widen roads, motorways or railways to increase their capacity. Meeting these needs is easy if the routes run completely on the surface, however it is very complicated when they run through stretches of tunnel, because it is indispensable in these cases to resort to costly alternative routes to construct the new tunnels in addition to those that already exist.

And then a tunnel can only be widened if it is possible to:

- guarantee the necessary safety of users and limit inconvenience within an acceptable threshold;
- solve the technical and operational problems connected with driving the face to widen the tunnel in ground that has already been disturbed by the previous excavation;
- construct the new load bearing structure at the same time as the old one is demolished and deal adequately with any stress-strain conditions, even unexpected conditions, which might be met during construction, without any danger to tunnel users and to any human activity there may be on the surface.

Currently as part of the work on the widening of the "Nazzano" tunnel, on the A1 Milan-Naples motorway, from two to three lanes, plus an emergency lane in each direction, a new innovative construction method is being used, proposed by the author, which will allow the requirments listed above to be met and road, motorway and rail tunnels to be widened without interrupting traffic during construction work (Fig. G.1). This appendix first discusses the major problems that had to be solved and then illustrates the basic concepts of this new technology with references to some of the first results of its application.

The indispensable requirements for the technique to be employed in the presence of traffic

If a technique to widen a tunnel in service without interrupting traffic is to be really possible, as mentioned in the introduction, it must solve at least two types of problem satisfactorily:

- the problem of performing the works required to excavate and construct the lining of the widened tunnel and demolish the existing

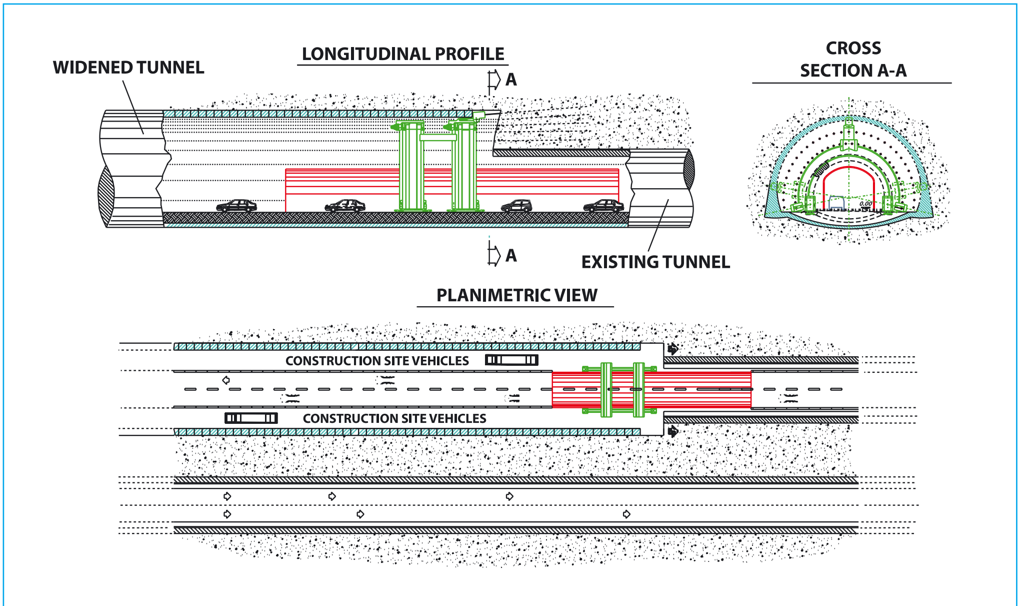


Fig. G.1

tunnel, while ensuring the safety of tunnel users and minimising inconvenience;
- the problem of adapting the technique for use in any type of ground and stress-strain conditions it might encounter, minimising the effects on the ground surrounding the tunnel and ensuring safety constantly during operations.

Clearly the development of a specific construction approach is required to solve these problems. It must allow all types of ground improvement in advance that may be required at the face and around the tunnel to be performed without any danger to traffic as well as the placing and activation of the final lining at a very short distance from the widening face.

It is only by operating in this way that it will be possible to:

- control the effects of the probable presence around the existing cavity of a band of ground that has already been plasticised and must not be subject to further disturbance;
- widen the cross section of the tunnel without triggering harmful deformation in the ground which would translate into huge thrusts on the lining of the final widened tunnel and differential settlement at the surface, dangerous for any constructions that might be present;
- ensure at the design stage that construction schedules are observed independently of the type of ground and stress-strain condi-

tions to be addressed, with costs contained and with reliable planning of construction costs and time schedules in order to minimise route deviations for traffic and inconvenience for users.

How the idea developed

The conviction that it was possible to develop a construction method suitable for these purposes, capable, that is, of satisfying the requirements listed above without interfering with normal motorway traffic, first began to take shape in the mind of the author at the time of the construction of the "Baldo degli Ubaldi" underground station in Rome. The large dimensions (span of 21.5 m and a height of 16 m) and the severe constraints on surface settlement required the design of an innovative construction method. Using that method the tunnel for the station was constructed in four main phases (Fig. G.2):

1a. two side tunnels were driven 5 m wide and 9 m high from two access shafts, which would house the future side walls of station tunnel, after first reinforcing the ground ahead of the face with fibre glass structural elements and lining the excavation with fibre reinforced shotcrete and steel ribs equipped with invert struts;

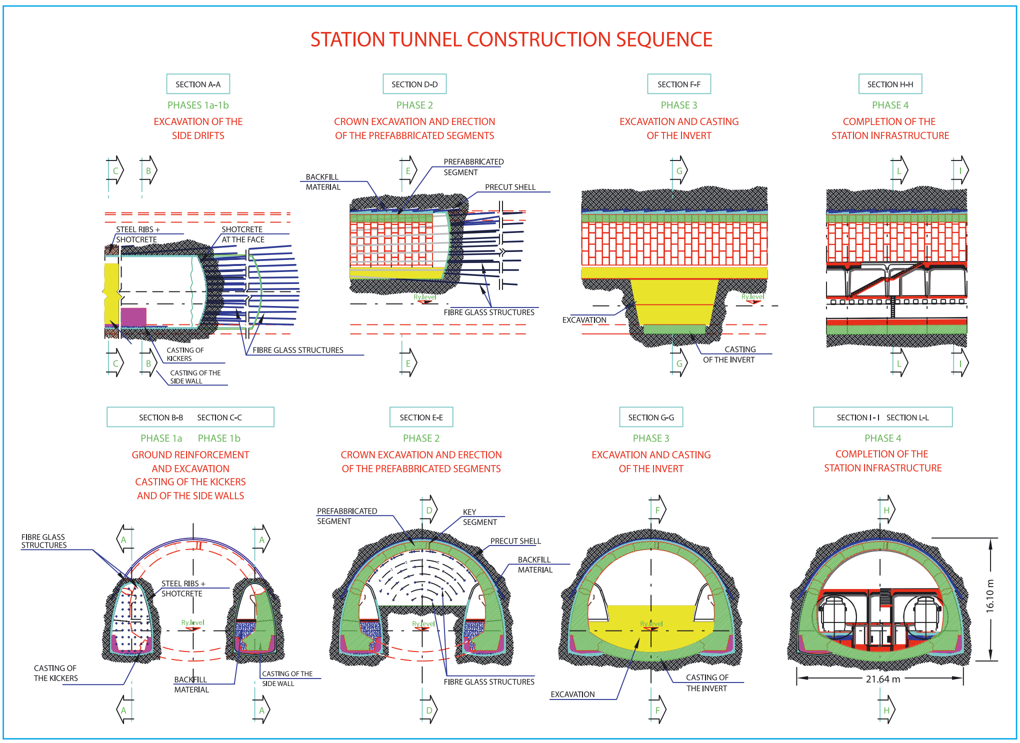


Fig. G.2 *Baldo degli Ubaldi Station – The construction stages*

1b. casting of the side walls with reinforced concrete;
2. driving the tunnel for the crown of the station tunnel (21.5 m span, 8.5 m high with a cross section of 125 m²) after first reinforcing the ground ahead of the face with fibre glass structural elements and protecting it with a strong shell created using the mechanical precutting method, and then immediately placing the lining of the crown with an “active arch” of prefabricated concrete segments;
3. excavation downwards of the station tunnel (cross section of 90 m²) and immediate casting of the tunnel inverts in steps after the construction of the crown;
4. completion of the station infrastructure.

The new construction system contained essentially two new elements:

- one was ground improvement ground ahead of the face with fibre glass structural elements and the mechanical precutting technology (employed for the first time in the world on a span of 21.5 m) combined with the “active arch” principle;
- the other was the extremely high level of industrialisation achieved with intense use of machinery.

In fact a special machine was designed, developed and constructed jointly with STAC S.p.A. of Mozzate (Como) to combine the technologies employed, all fairly recent, in a single construction system It consisted (Fig. G.3) of a large metal portal, with the same geometry as the profile of the crown of the station tunnel, which rested on the inside of the sidewall tunnels with stabilisers positioned on its side members to enable it to travel backwards and forwards. The portal not only contained the equipment for mechanical precutting but also housed that required for handling and placing the prefrabricated concrete segments of the final lining.

Once the machine and its accessory equipment were in operation, the author noticed during the construction operations that the area consisting of the bench of the future station tunnel, with a cross section of similar size to that of a normal motorway or rail tunnel, was not used at all for construction operations (Fig. G.4). These operations could have been performed in exactly the same way on the extrados of an existing tunnel, to widen it, without having to close it to traffic, naturally as long as appropriate measures were taken to protect tunnel users. It was, in the final analysis, a question of extending the half section system used on the Baldo degli Ubaldi tunnel to the full cross section.

That is how the idea of a technique was born using machinery and equipment based on those used for the Baldo degli Ubaldi station, which would be capable of widening a tunnel without putting it out of service during construction work (Fig. G.1).

Fig. G.3

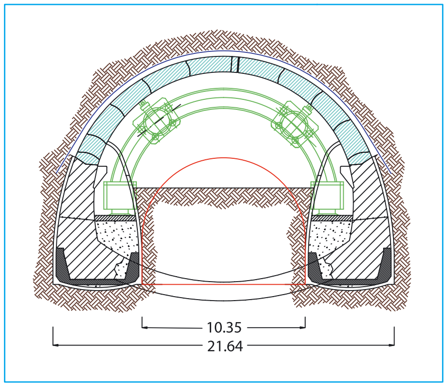


Fig. G.4

Illustration of the technique

The idea on which to work basically consisted of the following (Figs. G.5 and G.6)

- *the first stage*: operations, if necessary, to reinforce the widening face and/or for preconfinement of the cavity, and then excavation of the ground in steps between the theoretical profile of the future widened tunnel and that of the old existing tunnel;
- *the second stage*: the construction immediately behind the face of the final lining with the placing of one or more arches of prefabricated concrete segments according to the "active arch" principle;
- *the third phase*: the construction of the foundation (tunnel invert), if necessary.

During the first two stages, which should be performed in very regular cycles inside the profile of the old tunnel, there would be in place a *steel traffic protection shell* and all the machinery used in the operations would move and work above this. The hollow space between this steel protection and the lining of the existing tunnel would be filled with sound proofing and anti-shock material. The steel shell would be at least four times longer than the diameter of the tunnel to be widened and would occupy a relatively small space within it and allow construction work without interrupting traffic in the existing lanes. When tunnel widening advances to the point where the distance between the face and the front end of the shell reaches what is considered the minimum safety limit, the shell must be moved forward and the various stages repeated in cycles until the entire tunnel has been widened.

The paragraphs below give a detailed description from an operating viewpoint of each stage of the technique.

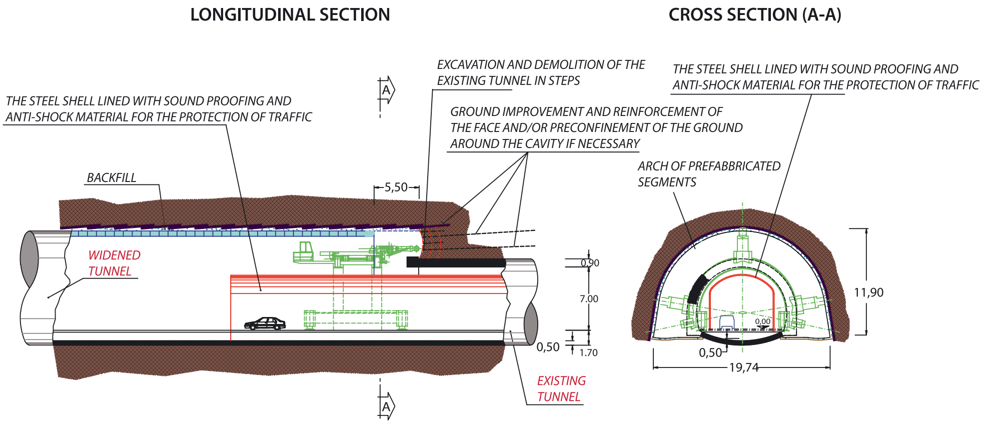


Fig. G.5 *Excavation of the ground at the widening face and demolition of the existing tunnel*

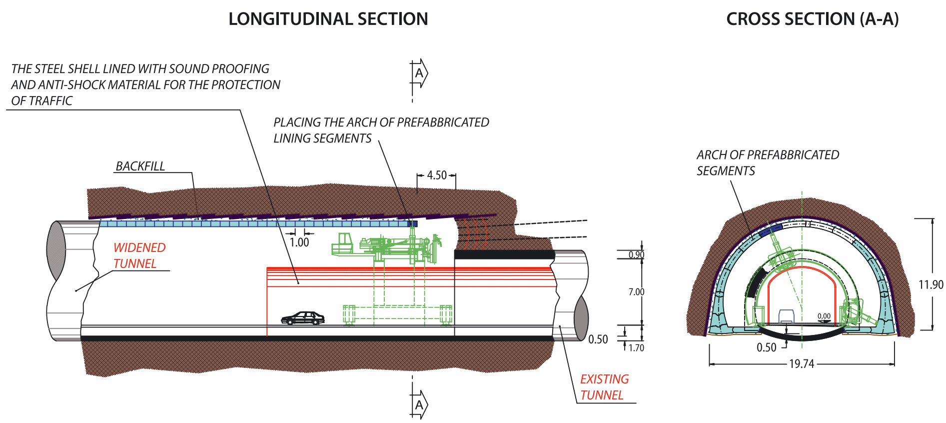


Fig. G.6 *Placing the arch of prefabricated lining segments, which alternates with excavation of the widening face*

The first operating stage

The first stage of the work, as has been said, involves first of all the ground improvement operations ahead of the face, if necessary, and then the excavation in steps (how large they are depends on the characteristics of the ground concerned) of the ground between the design profile of the future widened tunnel and that of the old existing tunnel.

The ground improvement operations ahead of the face performed on the basis of the geological and geotechnical context may consist of reinforcement in the widening face and/or preconfinement of the cavity, such as: horizontal jet-grouting, mechanical precutting or improvement using valved and injected fibre glass structural elements around the future widening face. They may be placed in advance or radially, working from inside the existing tunnel but in any case always above the "steel traffic protection shell".

After the ground improvement ahead of the face has been performed, work starts on driving the widening face (see Fig. G.5). This is done by excavating the ground and demolishing the lining of the existing tunnel in small steps [from 60 cm to 200 cm depending on the stress-strain conditions of the material being tunnelled and the size of the prefabricated segments designed for the final lining (see the section: *The second construction stage*)]. If the stress-strain conditions allow, tunnel advance may proceed in steps that are as long as several lining segment lengths.

The machinery used for ground reinforcement and improvement in advance and for lining the tunnel operates completely above the steel traffic protection shell and is equipped with all the equipment needed for the necessary ground improvement operations.

The second construction stage

The second construction stage entails placing the final lining of the widened tunnel consisting of prefabricated concrete segments.

Excavation in steps with the final lining consisting of prefabricated concrete segments placed immediately according to the "active arch" principle constitutes the key factor. It halts any possible deformation phenomena before it starts at a short distance from the face and overcomes all the problems of the deformation response of the rock mass. This is the most characteristic feature of the technique presented here.

The work involved the following operations (Fig. G.6):

a. transport of the concrete segments to the face using conveyor belts and fork lift trucks positioned on one side of the widened tunnel;

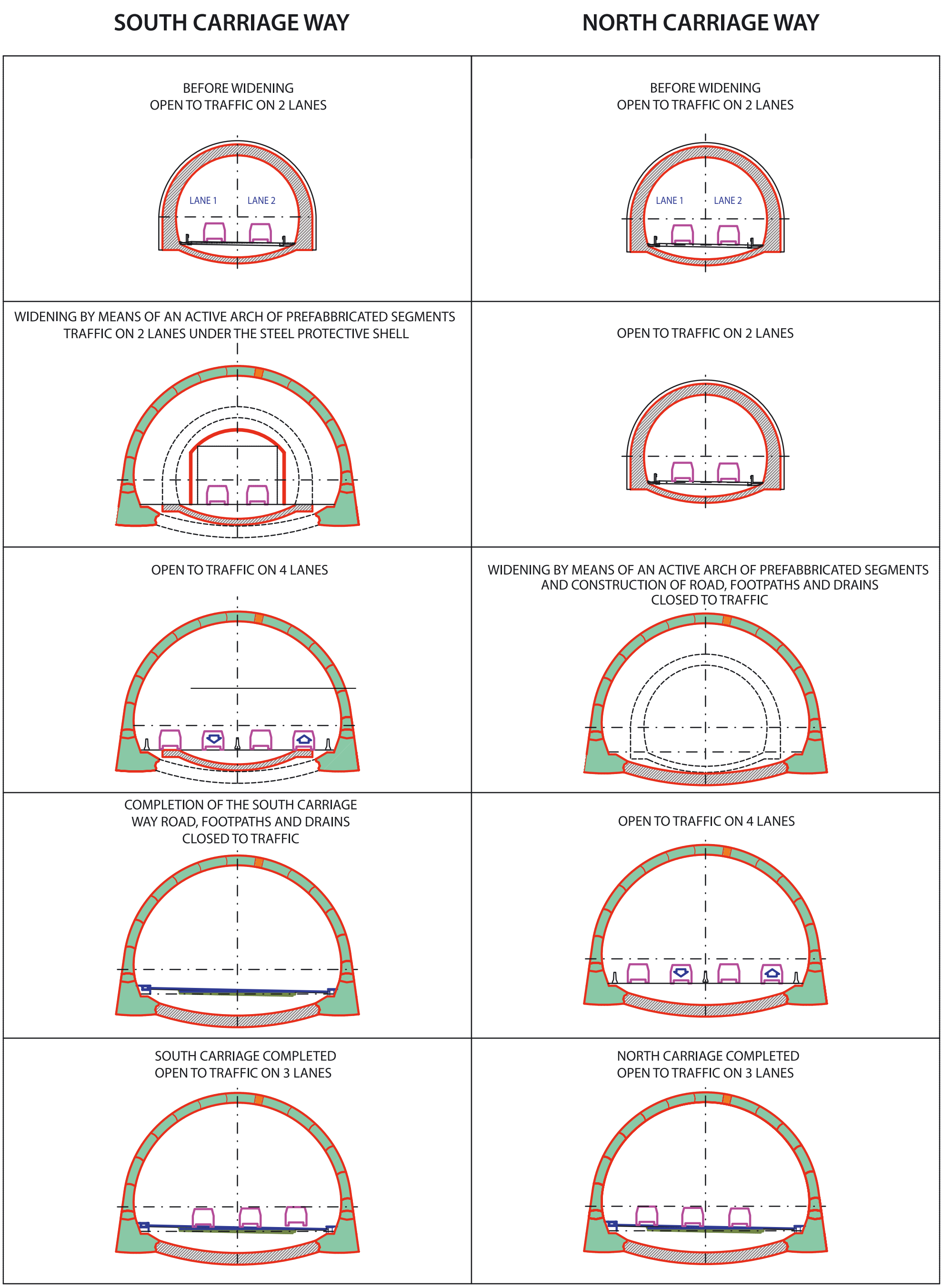


Fig. G.7 *The traffic regulation plan during widening of the carriage way*

b. the application of slow setting epoxy resins on the two sides of the segment to be placed and on the front end that will be in contact with the arch of the lining already placed;
c. raising and positioning of the segments using a special erector machine which places the lower segments first on both sides of the tunnel and then the upper segments until the arch is completed with the key segment in the roof;
d. mortar is introduced between the extrados of the prefabricated segments and the wall of the excavation behind it;
e. the pressure jack in the key segment is then activated to bring the segments into firm contact with each other and immediately produce the confinement pressure required on the ground around the tunnel according to the "active arch" principle.

The third construction stage

If a foundation structure is needed, traffic is first appropriately deviated, as we will see, and the lining of the new widened tunnel is simply joined to the tunnel invert of the old tunnel or a genuine new tunnel invert is cast.

Resolution of particular problems to maintain traffic flow during construction work

While the first two construction stages do not pose any particular problems to the maintenance of traffic flow since everything takes place above the steel traffic protection shell, with the third stage there are two distinct cases.

Rail tunnels

Once the tunnel has been widened, rail traffic will have to be interrupted to change the layout of the tracks in the new situation. The structure to join the final lining of the widened tunnel with the existing tunnel invert or, alternatively (if the statics conditions require it), the casting of a new tunnel invert can be performed in this interval of time.

Road (single bore) or motorway (twin bore) with 2 lanes in each direction

Traffic in road tunnels can always be kept flowing on at least one lane in each direction by appropriate organisation of the works to construct the foundations and to widen the road itself. Similarly two lanes in each direction can always be kept open with two bores by switching the works between the two tunnels and deviating traffic flow accordingly onto lanes as they become free (as illustrated in the example in Fig. G.7).

Application of the system for the widening of the "Nazzano" tunnel

The new technology is being applied experimentally for the first time in the world on the "Nazzano" tunnel located on the A1 Rome-Naples motorway betweem Orte and Fiano Romano, between chainage km 522 + 00 and km 523 + 200 (Fig. G.8).
The route of this tunnel is completely rectilinear and lies at an altitude of 166 m a.s.l.. It is 337 m long and runs under an overburden of 45 m From a geological viewpoint, the tunnel runs through sandy and silty-clayey ground of the Plio-Pleistocene series on which the town of Nazzano is located (Fig. G.9).
Given the type of ground to be tackled, the design first specifies the creation of a shell of fibre reinforced shotcrete around the tunnel using mechanical precutting before starting excavation to widen the tunnel.
Tunnel widening will therefore take place in the following four main stages (Figs. G.10 and G.11):

1. creation of a mechanically precut shell around the future tunnel (19.74 m span) 5.5 m in length and 35 cm thick and ground improvement ahead of the widening face if necessary;
2. demolition of the old lining in steps under the protection of the previously improved ground and excavation of the ground until the design profile of the widened tunnel is reached;
3. immediate erection of the final lining behind the face (4.5 – 6.5 m max.) by placing an arch of prefabricated concrete segments, using the "active arch" principle;
4. construction of the foundation structure (new tunnel invert).

All the operations for the first three stages will be performed protecting the road with a self propelled steel traffic protection shell under which vehicles may continue to pass in safety. The shield is designed with a length of 60 m and will extend for a length of approximately 40 m ahead of the widening face. It consists of a modular steel structure and is equipped with runner guides, anchors, motors, sound proof and anti shock panels to absorb the shock of falling blocks of material during excavation and demolition of the existing tunnel including any ground that falls accidentally.

All the machinery for performing the various operations will move and operate above the shell. When tunnel widening advances to the point where the distance between the face and the front end of the shell reaches what is considered the minimum safety limit for vehicle traffic, the shell will be moved forward and the various stages repeated in cycles until the whole tunnel has been widened.

The machine and its equipment

The design and construction of the machine prototype and its equipment required particular effort because a series of operating functions had to be optimised to work in a very limited space between the finished tunnel and the shield: precutting at the face, excavation, placing of the segments, various grouting operations, demolition of the existing tunnel.

The problems were solved by using innovative technology and the result was a versatile and compact design, highly computerised capable of performing all the functions required with movements and therefore also operating times reduced to a minimum.

Basically it consisted of a robust double arch steel structure (Fig. G.12) connected at the bottom by telescopic beams which gave rapid and precise longitudinal movement forwards and backwards.

Centring in the cross section and correct positioning of the height are achieved by hydraulic control systems.

A particularly sophisticated carriage is fitted on the arch at the face which carries the precutting blade and the cutter for excavation and demolition or alternatively a demolition hammer.

The circular movement of the carriage around the arch is obtained by gear reduction motors and a rack and pinion and the single and

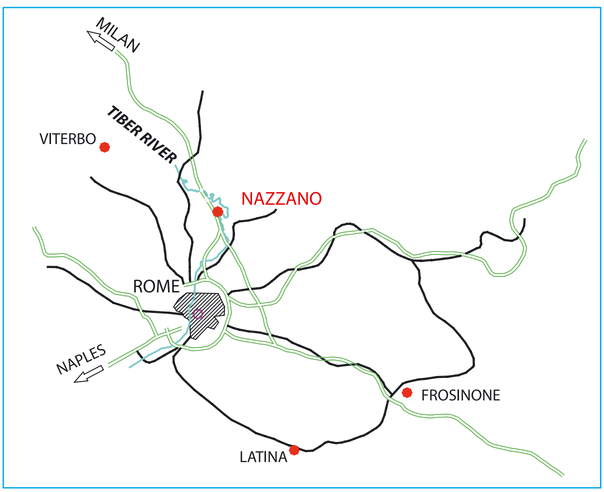


Fig. G.8 *Chorography*

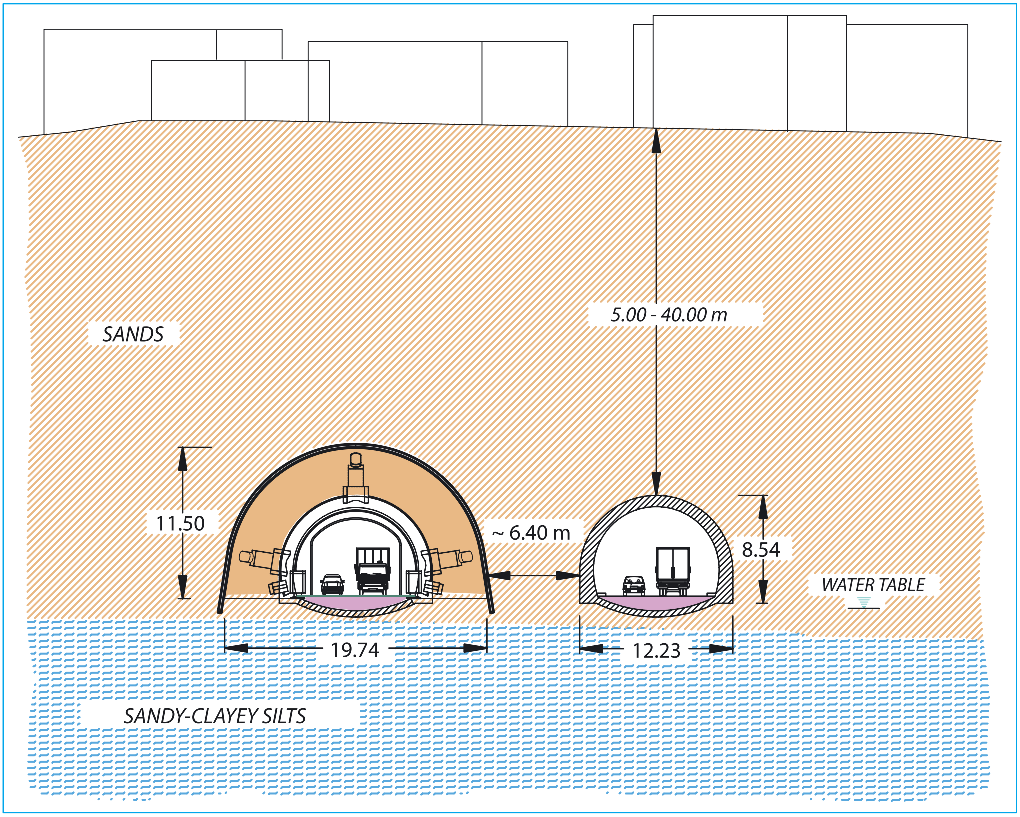


Fig. G.9

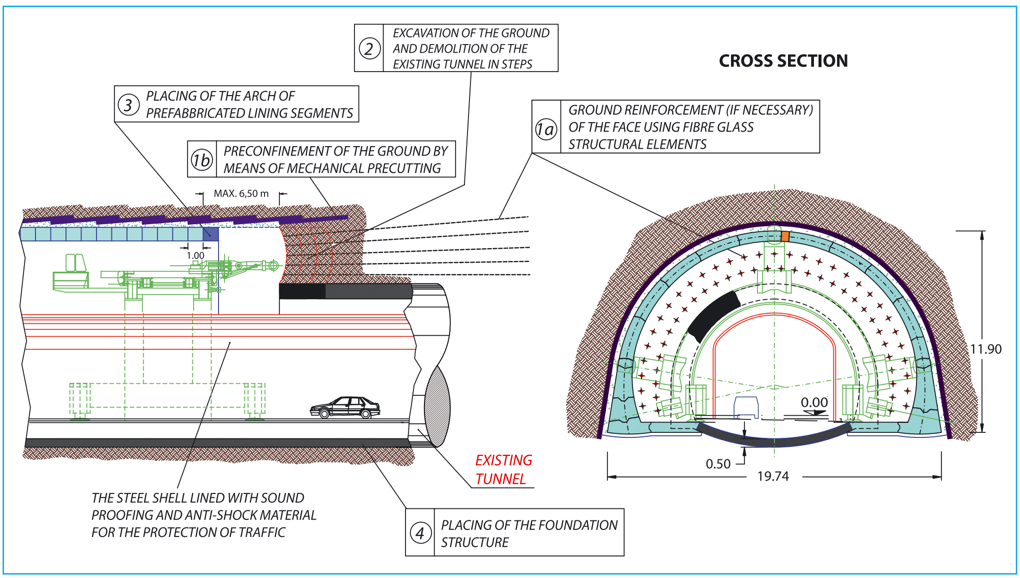


Fig. G.10

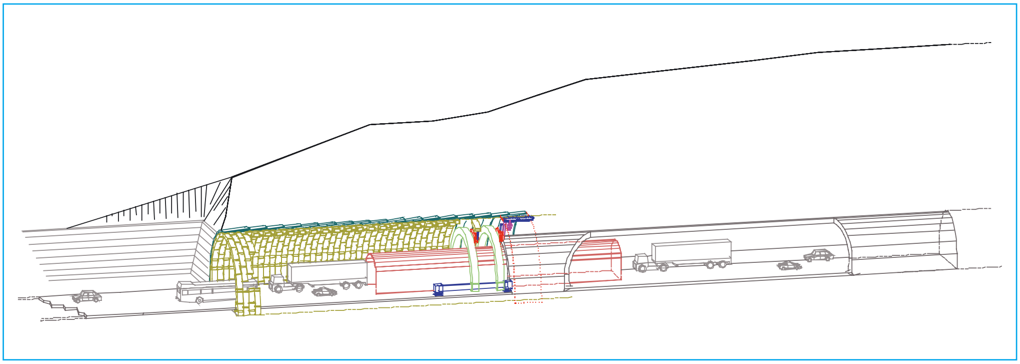


Fig. G.11 *Three dimensional view*

complex movements of the different parts allow the different operations specified in the design to be performed.

A dual system is also appropriately positioned on the same arch to manage the tubing used for filling the cut made with the precutting blade with mortar and the space between the segments and the walls of the excavation.

The rear arch was designed for placing the concrete segments. A carriage runs on it equipped with an *erector* capable of "grabbing" the segments and placing them. The movement of the erector is totally powered by electricity and hydraulics and it is controlled from a panel

Fig. G.12 *View of the machine with the large chain saw for making the precut in the foreground* equipped with a display which gives information on manoeuvres and errors that may have been committed.

Before the key segment of the arch is placed and it consequently becomes self supporting, the segments rest on special telescopic structures anchored to the arch. They are equipped with sensors which allow the different manoeuvres to be made in safety.

The structure is equipped with various service gangways to allow personnel to work with a clear view of operations.

The various functions of the equipment are controlled by a PLC *(Programmable Logic Controller)*, which recognises commands it receives and sends information to the display for correct and safe control of the equipment.

Table G.I below contains the main technical specifications of the machine in question.

Table G.I

TECHNICAL SPECIFICATIONS OF THE MACHINE	
Cutting capacity of the blade	L = 550 cm; th = 30 cm
Erection of the segments	max load = 7 ton at 10.70 m
Rated power	214 KW
Means of power	electricity-hydraulics
Movement	hydraulic controlled by a *Programmable Logic Controller*

The progress of works

Once a series of affairs had been resolved, attributable exclusively to contractual and financial problems, which delayed the start of widening work several times and after solving a series of problems connected with passing through the portal zone with a very shallow overburden, which was more heavily affected by the work already performed for the construction of the existing tunnel, regular advance rates were finally achieved for tunnel widening after the system was fine tuned.

On 17th November 2005, widening work was completed on the bore with North moving vehicles in the full presence of traffic. Average advance rates achieved were between 0.7 and 0.8 m/day.

Optimisation of the system and the advance cycle mainly concerned the adoption of a longer cutting tool capable of making a “precut” 5.5 m long and 35 cm thick (compared to the initial tool of 4.5 m and 30 cm).

The precut was then followed by two stages of excavation, the demolition of the existing tunnel and the placement of shotcrete at the face, for an advance of 2 m each, interposed by the erection of two consecutive “active arches” in the crown with a length of 1 m each.

Fig. G.13 *The work to widen the tunnel was performed with the motorway in service*

5. Conclusions

The first results of the experiment indicate that the technique illustrated solves the specific problem of widening a road, motorway or rail tunnel while allowing traffic to continue to flow during construction work.

Its main features are:

1. the adoption of a final lining consisting of prefabricated concrete segments to stabilise the widened tunnel placed in short steps according to the "active arch" principle, which therefore comes into operation at a very short distance from the widening face (4.5–5.6 m max). As a consequence passive stabilisation operations such as shotcrete and steel ribs are avoided;
2. the final lining can be put under pressure by using jacks in the key segment designed to recentre asymmetrical loads should there be bending moments sufficient to make the resisting section of the arch of prefabricated segments act partially;
3. the ability to perform ground improvement ahead of the face, if required, to contain or even completely eliminate deformation of the

Fig. G.14 *The steel shell to protect traffic. The tracks on which the machine moves can be seen at the bottom of the photo*

Fig. G.15 *The widening face during demolition*

Fig. G.16 *Backfill of the precut with pumped mortar*

Fig. G.17 *Erection of the prefabricated lining segments in pretressed r.c.*

Fig. G.18 *View of the widened tunnel*

face and of the cavity and therefore avoid the uncontrolled loosening of the rock mass and thereby ensure operational safety;

4. intense mechanisation of the various construction stages, including the operations for ground improvement ahead of the face if required, with consequent *regular advance rates* and shorter construction times, all factors that have advantageous repercussions for construction economics and the production rates that can be achieved;
5. the extremely linear production rates obtainable (industrialised tunnelling), which it is predicted will be around 0.6–1.2 m/day of finished tunnel;
6. the ability to perform all construction operation while protecting the road with a "steel shell" under which traffic can continue to flow in safety;
7. The extreme versatility of the machine used, which is able to operate under extremely varied ground and stress-strain conditions.

After a significant period spent fine tuning the system, connected with the fact that it was the first time that this technology had ever been used to solve the problem of widening a tunnel with traffic running, the experimentation in progress demonstrated that the following can be achieved with this technology:

- controlling the effects of the probable presence around the existing cavity of a band of ground that has already been subjected to plasticisation and must not be disturbed any further;
- widening the cross section of the tunnel without causing damaging deformation of the ground and therefore preventing substantial thrusts on the lining of the widened tunnel from developing with differential surface settlement dangerous for existing structures;
- ensuring that construction occurs on schedule as specified in the design, independently of the type of ground and the stress-strain conditions, with construction times and costs contained and planned in order to reduce traffic deviations and inconvenience to users to a minimum.

Glossary

Acquired consistency: the consistency assumed by a material, which varies as a function of the magnitude and anisotropy of the stress tensor acting on it.

Arch effect: a phenomenon of *channelling stresses* around an excavation, which can be performed either by natural or artificial means, inside the mass of the ground, as a result of which underground cavities are able to exist and can be created.

Back-analysis: a reverse calculation based on measurements performed during the monitoring phase (extrusion, convergence, pressure on linings, etc.), used to calculate the strength and deformability parameters of the ground with greater accuracy (dependent also on the calculation models employed). It is therefore an important tool for testing the accuracy of the design hypotheses and procedures.

Barring down: an operation that is performed for safety purposes after each round of shots when blasting, which consists of either manually, or using mechanical equipment, knocking down fragments of rock which remain partially attached to the crown or walls of the tunnel.

Behaviour Category: a general concept with which tunnels to be constructed or which are being constructed can be interpreted and classified according to the universal criterion of the stress-strain response of the core-face in the absence of stabilisation intervention. The ADECO-RS approach identifies three distinct stress-strain behaviour categories: category A: stable core-face; category B: stable core-face in the short term; category C: unstable core-face, with the observation that all known cases of tunnels built fall within these three categories. The design engineer defines the action which must be produced to achieve the complete stabilisation of a cavity in the long and short term and as a consequence the most appropriate stabilisation intervention to employ on the basis of the expected behaviour category of the ground's response to excavation.

Bench: the intermediate part of the cross section of a tunnel between the crown and the tunnel invert.

Cavity: a space artificially hollowed out of a mass of ground, the existence of which in time depends on the formation in the mass around the cavity of an arch effect which, if it is not created by natural means, can be created by artificial means, by intervening to stabilise the ground.

Channelling of stresses: a phenomenon which is produced in ground subject to natural stresses after a disturbance is generated in the stress equilibrium by the creation of a cavity, generally consisting of one or more series of parallel flow lines, where the space between the lines depends on the intensity of the field. If the geomechanical characteristics of the medium are sufficient to support the intensity, then the continuity of the stress flow is conserved by natural means by deviating the lines intercepted by the creation of a cavity in an outwards direction. This deviation of the stress flow lines away from the cavity and the consequent concentration of the lines in a narrow space around the perimeter of the cavity is identified by this term the *channelling of stresses.*

Characteristic lines (method of the): calculation method based on plotting curves which place the confinement pressure σ_3 applied on the perimeter of a tunnel in relation to its *radial convergence*.

Confinement pressure: radial pressure that acts on the perimeter of the excavation. It is given by the principal minor stress σ_3.

Confinement: action exerted on the walls and roof and/or on the face of a tunnel to prevent the principal minor stress σ_3 in the surrounding ground from falling to zero. This action can be produced either directly or indirectly with either active of passive action. As long as it is not produced in statics conditions that have already been compromised, confinement action will reduce the magnitude of the plasticisation which normally affects the ground around a tunnel when the material is stressed beyond its elastic limit (thickness of the plasticised ring). It therefore favours the formation of an arch effect, which is indispensable for the stability of a tunnel in the long and short term.

Convergence: reciprocal movement between two points located on the perimeter of a cavity which is produced as a consequence of the deformation response of the rock mass to the action of excavation. Convergence can develop in time either instantaneously or in a deferred manner, depending on whether the deformation response develops in the elastic or the elasto-plastic range, and on the rheological characteristics of the material.

Core-face: the system consisting of the face and the advance core.

Cut: in conventional excavation by blasting this is a hole or cavity created in the face, generally in the centre to create a free surface in the direction of which the surrounding rock can be demolished with excavation blasts.

Decompression (of the ground, of the medium): decay, by the decrease or disappearance of the principal minor stress σ_3, of the geomechanical characteristics of the ground (of its cohesion in particular) to residual values, which can occur in the band of ground surrounding the excavation in the absence of adequate intervention when the material is stressed beyond its peak strength.

Deformation phenomena: see the *deformation response*.

Deformation response: the reaction of the medium to the action of excavation. It manifests in the form of extrusion, preconvergence and convergence.

Diagnosis phase: that phase in the *design stage,* during which the design engineer uses the information collected during the survey phase to divide the tunnel to be bored into sections with uniform stress-strain behaviour (categories A, B, C), defining how deformation will develop and the types of load that will be mobilised for each of them;

Disturbance in the medium: modification of the natural field of stresses that is produced in the ground following tunnel advance. This field, which can be described as a network of flow lines, is deviated by the presence of the excavation and is concentrated in its proximity (*channelling of stresses*), producing an increase in the deviator of the stresses.

Extrusion: movement of the ground that forms the core-face of a tunnel into the tunnel itself in a longitudinal direction along the axis of the tunnel. It is a primary component of the deformation response of the medium to the action of excavation, which develops mainly inside the advance core as a function of the strength and the deformability of the core and of the original field of stresses it is subject to. It manifests at the surface of the tunnel face in a longitudinal direction along the axis of the tunnel, either with more or less axial symmetrical deformation geometry (bellying of the face) or as gravitational turning (rotation of the face). It is generally measured as a function of time (face halted) or of tunnel advance.

Face: the front wall of a tunnel which is being driven, which closes its foremost extremity in the mass of the ground.
Its position moves progressively forwards as the medium (ground) is excavated.

Failure range: this is identified in the three dimensional space of the principal stresses as the dominion of stresses that are incompatible with the strength capacity of the material.

Ground improvement: action performed on the ground to conserve or improve its natural strength, deformability and permeability properties.

Ground improvement in advance: intervention to stabilise the ground ahead of the face, by improving or conserving the geomechanical characteristics of the ground in the advance core.

Interpretation of measurements: processing and critical analysis of measurement readings taken in the monitoring phase in order to obtain a sufficiently accurate and detailed understanding of how the stress-strain conditions in the ground are evolving.

Intrinsic curve: line of failure of the ground or an earth mass which is normally obtained as the envelope of the circles of principal failure stresses, according to the criterion employed (Mohr-Coulomb, Hoeck-Brown, Drucker-Prager etc.).

Measurement station: section of tunnel in which instruments are installed, either before or after the passage of the face, to take measurements of stress and/or deformation and/or hydraulics (e.g.: extensometers, pressure and load cells, piezometers, distometer nails, dilatometers etc.).

Muck/spoil removal: the operation which consists of removing loading and transporting excavated earth to the surface.

Natural consistency: the consistency typical of a material which depends exclusively on its intrinsic physical and mechanical properties.

Pilot tunnel: small diameter tunnel driven using a full face TBM, before full tunnel excavation commences, located near or along its axis with the primary purpose of performing an in-depth survey of the geomechanical and structural characteristics of the ground concerned (see also *RS-Method*), but sometimes also to perform advance ground improvement which may be necessary for the construction of the full tunnel. If it is driven outside the cross section of the full tunnel to be driven, it may also serve as a permanent service and safety tunnel (ventilation, drainage, emergency exits and first aid access, etc.) in addition to its other temporary purposes.

Plasticised ring: zone of ground with plastic behaviour which is formed around a tunnel when the stresses induced by excavation exceed the peak strength of the ground which is tunnelled. The thickness of the zone depends on the peak and residual strength parameters of the ground, the natural stress state and, when tunnel advance takes place under the water table in hydrodynamic conditions, on the hydraulic load and the permeability of the material.

Preconfinement: stabilisation intervention which acts ahead of the face on the ground around the future cavity to either prevent the principal minor stress from falling or even to increase it. It uses the advance core to achieve this, by protecting it from excess stress and/or by conserving or even by improving its rigidity.

Preliminary lining: general term which includes all those structural stabilisation instruments employed ahead of the face and inside the tunnel, to eliminate or contain the deformation response of the medium to the action of excavation in the long and short term.

Presupport: a stabilisation instrument placed ahead of the face, but which does not produce any arch effect ahead of the face. Presupports (e.g. forepoles) can sometimes be employed in fractured rocky ground when tunnelling under elastic conditions to protect against concentrated falls of rock and to prevent the localised fall of material.

Radial convergence: radial movement in relation to the axis of a tunnel of a generic point located on the perimeter of the excavation or inside the mass of the ground which occurs as a consequence of the deformation response of the ground to the action of excavation. Radial convergence can develop in time either instantaneously or in a deferred manner, depending on whether the deformation response develops in the elastic or the elasto-plastic range. Surface or depth (radial) convergence is spoken of according to whether it refers to a point on the walls of the tunnel or a point inside the mass of the ground.

Radius of influence of the face R_f: the size of the disturbed zone surrounding the face. It depends on the radius of the tunnel, on the geomechanical characteristics of the medium, on the rate and means of excavation and on the stabilisation intervention implemented.

Relaxation: decrease or disappearance of the principal minor stress σ_3 with a consequent decrease in the shear strength of the medium.

Rigidity (of the advance core, of the core-face): a characteristic of the advance core or the core-face, which determines its greater or lesser tendency to extrude when stressed in the elasto-plastic range, or the magnitude of the cavity preconfinement action that it is able to exert.

Round of shots: simultaneous or predetermined sequence of explosive detonations specifically placed to give a determined result in terms of face excavation.

Rock mass: this term has been widely used in the book to translate the Italian word *ammasso,* which really means just the “mass” of the ground generally without specifying whether it is rock or soil. As a consequence the term “rock mass” has often been used generically to refer to the ground whether as a rock or soil mass.

RS Method: survey instrument which uses a TBM as a huge penetrometer, measuring its operating parameters by means of special sensors (thrust, advance velocity, energy consumed, etc.). It is thus possible to determine the specific energy employed per m^3 of material excavated and, by means of appropriate correlations, also the strength of the rock mass.

Scaling: see *Barring down*.

Section with uniform stress-strain behaviour: portion of the underground alignment in which it is reasonable to hypothesise that the deformation response, understood as the reaction of the ground to the action of excavation, will be qualitatively constant for the whole of its length.

Solid load: mass of ground potentially unstable due to the effect of gravity, which, if not adequately treated, will produce instability sooner or later or which would weigh, as a dead weight, on the lining of the tunnel.

Spoil: material detached and demolished by the blast of explosions or by any other procedure for excavating the earth.

Survey phase: that phase in the *design stage*, during the which the design engineer, proceeds to characterise the terrain, or the medium, through which the tunnel will pass in terms of rock or soil mechanics. It is indispensable for analysing the pre-existing natural equilibriums and for proper conduct of the subsequent diagnosis phase.

The advance core: the volume of ground which lies ahead of the face, practically cylindrical in shape with a cross section and length of around the size of one tunnel diameter.

The monitoring phase: that phase of *the construction stage* in which deformation phenomena (which constitute the response of the medium to the action of deformation) are read and interpreted during the construction of a tunnel to monitor the accuracy of the predictions made during the diagnosis and therapy phases in order to perfect the design by balancing stabilisation instruments between the face and the cavity. The monitoring phase does not finish when a tunnel is completed, but continues for the whole course of its life in order to constantly monitor its safety in service.

The operational phase: that phase in the *construction stage* during which stabilisation intervention is implemented in compliance with the design specifications. It is adapted, in terms of preconfinement and confinement, to fit the real deformation response of the ground and monitored according to pre-established quality control plans.

The therapy phase: that phase in the *design stage* during which the design engineer decides the action to exert (*preconfinement* or simple *confinement*) and the intervention required to achieve the complete stabilisation of the tunnel in the long and short term, on the basis of the predictions made in the diagnosis phase. He then decides the composition of the longitudinal and cross section types and their dimensions and tests their effectiveness using mathematical tools, specifying the criteria for their application and possible variations to be made as a function of the stress-strain behaviour of the ground that will actually be observed during excavation. He then finally draws up the monitoring plan.

Tunnel: a civil engineering work designed to provide a continuous path for communication or for a watercourse through a mass of earth which it is not possible or not worthwhile creating by other means. It is complete from a structural point of view but does not include the finishings characteristic of the particular use to which it is destined.

Variability: a possible variation in a given tunnel section type of the intensity and/or geometry of stabilisation intervention, to be applied when statistically probable conditions, clearly described in the design, occur, but for which the precise location could not be predicted on the basis of the data available at the design stage.

Zero reading: this is the first reading taken after a measurement instrument is installed, against which all the subsequent readings performed on the same instrument are compared.

Bibliography

[1] KÁRMÁN T., “Festigkeitsversuche unter allseitigem Druck”, Berlin 1912

[2] BIENIAWSKI Z.T., “Rock Mechanics design in mining and tunneling”, Balkema 1984

[3] BARTON N.R., LUNDE J., “Engineering classification of rock masses for the design of tunnel support”, International Journal of Rock Mechanics and Mining Sciences & Geomech. Abst. 25, No. 13–13, 1974

[4] BIENIAWSKI Z.T., “Rock mass classification as a design aid in tunnelling”, Tunnels & Tunnelling, July 1988

[5] RABCEWICZ L., “The New Austrian Tunnelling Method”, Water Power, November 1964, December 1964, January 1965

[6] THE INSTITUTION OF CIVIL ENGINEERS, “Sprayed concrete linings (NATM) for tunnels in soft ground”, Telford 1996

[7] KOVÁRI K., “On the Existence of NATM, Erroneous Concepts behind NATM”, Tunnel, No. 1, 1994 (English and German), or Gallerie e Grandi Opere Sotterranee, No. 44, December 1994 (Italian and English)

[8] KOVÁRI K., “Il controllo della “risposta dell’ammasso” in galleria: pietre miliari fino agli anni 1960”, Quarry and Construction, July–August 2001

[9] LUNARDI P., “Applicazions de la Mecanique des Roches aux Tunnels Autoroutiers. Example des tunnels du Frèjus (côte Italie) et du Gran Sasso”, Revue Francaise de Geotechnique No. 12, 1979

[10] KASTNER H., “Statik des Tunnel- und Stollenbaues”, Sprinter, Berlin-Göttingen-Heidelberg, 1962

[11] PANET M., “Le calcul des tunnels par la méthode convergence-confinement”, Ponts et chaussées, 1995

[12] LOMBARDI G., AMBERG W.A., „Une méthode de calcul élasto-plastique de l‘état de tension et de déformation autour d‘une cavité souterraine“, Congresso Internazionale ISRM, Denver, 1974

[13] LUNARDI P., “La costruzione della galleria S. Elia lungo l’autostrada Messina-Palermo”, Costruzioni, December 1988

[14] LUNARDI P., “Aspetti progettuali e costruttivi nella realizzazione di gallerie in situazioni difficili: interventi di precontenimento del cavo”, International Convention on «Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo» – Milan 18–20 March 1991

[15] LUNARDI P., FOCARACCI A., “Aspetti progettuali e costruttivi della galleria Vasto”, Quarry and Construction, August 1997

[16] GOLINELLI G., LUNARDI P., PERELLI CIPPO A., "La prima applicazione del jet-grouting in orizzontale come precontenimento delle scavo di gallerie in terreni incoerenti", International Convention on "Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo" – Milan 18–20 March 1991
[17] ARSENA F.P., FOCARACCI A., LUNARDI P., VOLPE A., "La prima applicazione in Italia del pretaglio meccanico", International Convention on «Il consolidamento del suolo e delle rocce nelle realizzazioni in sotterraneo» – Milan 18–20 March 1991
[18] LUNARDI P., BINDI R., "The evolution of reinforcement of the advance core using fibre glass elements for short and long term stability of tunnels under difficult stress-strain conditions: design, technologies and operating methods", Proceedings of the International Congress on "Progress in Tunnelling after 2000", Milan, 10–13 June 2001
[19] LUNARDI P., BINDI R., FOCARACCI A., "Nouvelles orientation pour le projet et la construction des tunnels dans des terrains meubles. Études et expériences sur le préconfinement de la cavité et la préconsolidation du noyau au front", Colloque International "Tunnels et micro-tunnels en terrain meuble", Paris 7–10 February 1989
[20] LUNARDI P. et al., "Tunnel face reinforcement in soft ground design and controls during excavation", International Convention on "Towards New Worlds in Tunnelling" – Acapulco 16–20 May 1992
[21] LUNARDI P., "Fibre-glass tubes to stabilize the face of tunnels in difficult cohesive soils SAIE: Seminar on "The application of fiber Reinforced Plastics (FRP) in civil structural engineering" – Bologna, 22 October 1993
[22] LUNARDI P., "La stabilité du front de taille dans les ouvrages souterraines en terrain meuble: etudes et experiences sur le renforcement du noyau d'avancement", Symposium international "Renforcement des sols: experimentations en vraie grandeur des annes 80", Paris, 18 November 1993
[23] LUNARDI P., "Avanza la galleria meccanica" – Le Strade, May 1996
[24] LUNARDI P., "The influence of the rigidity of the advance core on the safety of tunnel excavation", Gallerie e grandi opere sotterranee, No. 52, 1997 (Italian and English)
[25] BROMS B.B., BENNERMARK H., "Stability of clay at vertical openings", Journal of the Soil Mechanics and Foundation Division, 1967
[26] ATTEWELL P.B., BODEN J.B., "Development of the stability ratios for tunnels driven in clay", Tunnels and Tunnelling, 1971
[27] PANET M., GUENOT A., "Analysis of convergence behind the face of a tunnel", Tunnelling '82, The Institute of Mining and Metallurgy, London 1982, pages 197–204

[28] TAMEZ E., "Estabilidad de tuneles exavados en suelos", Work presented upon joining the Mexican Engineering Academy, Mexico 1984
[29] ELLSTEIN A.R., "Heading failure of lined tunnels in soft soils", Tunnel and Tunnelling 1986
[30] CORNEJO L., "El fenómeno de la instabilidad del frente de excavación y su repercusión en la construcción de túneles", International Congress on "Tunnels and Water", Madrid 1988
[31] PANET M, LECA E ., "Application du calcul à la rupture à la stabilité du front de taille d'un tunnel", Geotecnique 1988
[32] CHAMBON P., CORTÉ J.F., "Stabilité du front de faille d'un tunnel faiblement enterré: modélisation en centrifugeuse", Colloque International "Tunnels et micro-tunnels en terrain meuble", Paris, 7–10 February 1989
[33] CHAMBON P., CORTÉ J.F., "Stabilité du front de faille d'un tunnel dans milieu frottant. Approche cinematique en calcul à la rupture", Revue Française de Geotechnique 1990
[34] LOMBARDI G., "Dimensioning of tunnel linings", Tunnels and Tunnelling 1973, pages 340–351
[35] LOMBARDI G., "Nuovi concetti sulla statica delle gallerie", La Rivista della Strada, 406, 1975, pages 465–482
[36] LUNARDI P., "Convergence-confinement ou extrusion-préconfinement?", Colloque "Mécanique et Géotechnique " (Giubileo scientifico di Pierre Habib), Laboratoire de Mécanuque des Solides, École Polytechnique, Paris 19 May 1998
[37] GALFO R., "Studio della stabilità del fronte di scavo e metodi di stabilizzazione", Tesi di laurea, Università degli Studi di Firenze, Facoltà di Ingegneria Civile, 1990
[38] FUKUSHIMA S., MOCHIZUCHI Y., KAGAWA K., YOKOYAMA A., "Model study of pre-reinforcement method by bolts for shallow tunnel in sandy ground", International Congress on "Progress and Innovation in Tunnelling", Toronto 9–14 September 1989
[39] ANTHOINE A., "Une méthode pour le dimensionnement à la rupture des ouvrages en sols renforcés", Revue Française de Geotechnique, 50, 1990
[40] LUNARDI P., "Conception et exécution des tunnels d'après l'analyse des déformations contrôlées dans les roches et dans les sols", (Italian and French), Article in three parts: Quarry and Construction, March 1994, March 1995, April 1996 or Revue Française de Geotechnique, No. 80, 1997, No. 84, 1998, No. 86, 1999
[41] LUNARDI P., "Design & constructing tunnels – ADECO-RS approach", Tunnels and Tunnelling International, special supplement, May 2000
[42] LUNARDI P., Progetto e costruzione di gallerie – Approccio ADECO-RS", Quarry and Construction, supplement, May 2001

[43] LUNARDI P., “La stabilite du front de taille dans les ouvrages souterraines en terrain meuble: etudes et experiences sur le renforcement du noyau d’avancement”, Symposium international “Renforcement des sols: experimentations en vraie grandeur des annes 80”, Paris, 18 November 1993

[44] LUNARDI P., “Lo scavo delle gallerie mediante cunicolo pilota”, Politecnico di Torino, Primo ciclo di conferenze di meccanica e ingegneria delle rocce – Turin, 25–26 November 1986

[45] CAMPANA M., LUNARDI P., PAPINI M., “Dealing with unexpected geological conditions in underground construction: the pilot tunnel technique”, Acts of 6th European Forum on “Cost Engineering” – Università Bocconi, Milan, 13–14 May 1993, Vol. 1

[46] GOODMAN R.E., SHI GEN-HUA, “Block theory and its application to rock engineering”, Prentice Hall, New Jersey, 1985

[47] FAZIO A.L., RIBACCHI R., “Stato di sforzo e di deformazione intorno a una galleria sotto falda”, Gallerie e grandi opere sotterranee, 17, 1983

[48] HUERGO P.J., NAKHLE A., “L’influence de l’eau sur la stabilité d’un tunnel profond”, International Congress on “Tunnels and Water”, Madrid 1988

[49] LUNARDI P., BINDI R., La teoria dell’Area Indice nel dimensionamento dell’interasse tra cavità adiacenti. Applicazioni al caso del tunnel sotto La Manica”, International Congress on “Grandi Opere Sotterranee”, Florence 8–11 June 1986

[50] LUNARDI P., FROLDI P., “Influence des conditions géostructurelles et géomécaniques sur les phénomènes de rupture dans la calotte du tunnel, VII International IAEG Congress – Lisbon 5–9 September 1994 – vol. VI

[51] HOEK E., BRAY J.W., “Rock slope engineering”, Institution of Mining and Metallurgy, London 1981

[52] LUNARDI P., “Il progetto dello scavo nella realizzazione di opere in sotterraneo”, Atti della 4° sessione delle Conferenze Permanenti Alta Velocità su “Lo scavo: metodi, tecniche ed attrezzature nella progettazione e costruzione della tratta Bologna-Florence”, Scarperia (FI) 31 March 2000

[53] Recommendations and Guidelines for Tunnel Boring Machines (TBM), International Tunnelling Association, Working Group No. 14, September 2000

[54] MAIDL B., HERRENKNECHT M., ANHEUSER L., “Mechanised shield tunnelling”, Ernst & Sohn, Berlin 1996

[55] ARNOLDI L., Lo scavo meccanizzato, nella nostra esperienza, con TBM del tipo aperto”, Convention on “Scavo meccanizzato integrale di gallerie: esperienze ed esigenze di committenti, fabbricanti di macchine, imprese e progettisti”, Rome 13–15 May 1997

[56] PANCIERA A., PICCOLO G., “TBM scudate. Blocco in rocce spingenti”, Convention on “Scavo meccanizzato integrale di gallerie:

esperienze ed esigenze di committenti, fabbricanti di macchine, imprese e progettisti", Rome 13–15 May 1997
[57] LUNARDI P. et al., "Soft ground tunnelling in the Milan Metro and Milan Railway Link. Case histories", Soft Ground Tunnelling Course – Institution of Civil Engineers – London, 10–12 July 1990
[58] LUNARDI P., "Evolution des technologies d'escavation en souterrain dans des terrains meubles", Comité Marocain des Grands Barrages" - Rabat, 30 September 1993
[59] LUNARDI P., "The construction of large-span stations for underground railways", Tunnels, 8, December 2000
[60] LUNARDI P:, FOCARACCI A., RICCI D., VALENTE A., "Una soluzione innovativa per la realizzazione di gallerie naturali senza copertura", Quarry and Construction, May 1997
[61] LUNARDI P. et al., "Quality Assurance in the Design and Construction of Underground Works", International Congress on "Underground Construction in Modern Infrastructure", Stockholm 7–9 June 1998
[62] TERZAGHI K., "Mécanique théorique des sols", Dunod, Paris 1951
[63] FALCHI DELITALA G., "Calcolo dei rivestimenti delle gallerie", Vitali e Ghianda, Genoa 1971
[64] COLONNA P., "Su alcuni problemi di stabilità dei fronti di scavo in galleria, con particolare riferimento agli ammassi interposti tra due fronti in progressivo avvicinamento", Quarry and Construction, January 1997
[65] CUNDALL P.A., "Numerical modeling of jointed and faulted rock", Mechanics of Jointed and Faulted Rock, A. A. Balkema, Rotterdam 1990, pages 11–18
[66] BERRY P., DANTINI E.M., LUNARDI P., "Pressioni in aria e sismi indotti in opere sotterranee da volate in galleria", Strade & Autostrade, 5, 1999
[67] BERTA G., "L'esplosivo strumento di lavoro", Italesplosivi, Milan 1989
[68] LUNARDI P., "Pretunnel advance system", Tunnels and Tunnelling International, October 1997
[69] ANDRE D., DARDARD B., BOUVARD A., CARMES J., "La traversée des argiles du tunnel de Tartaiguille", Tunnels et ouvrages souterrains, No. 153, May–June 1999
[70] LUNARDI. P., "The "Tartaiguille" tunnel, or the use of the ADECO.-RS approach in the costruction of an "impossible" tunnel", Gallerie e grandi opere sotterranee, No. 58, August 1999 (Italian and English)
[71] MARTEL J., ROUJON M., MICHEL D., "TGV Méditerranèe – Tunnel de Tartaiguille: méthode pleine section", Proceedings of the International Conference on "Underground works: ambitions and realities", Paris, 25th–28th October 1999

[72] "Florence to Bologna at high speed", Tunnels and Tunnelling International, April 1999

[73] "Progress report on the underground works on the Bologna to Florence high-speed railway line", Tunnel, No. 8, 2000

[74] LUNARDI P., BINDI R., CASSANI G., "Prime evidenze e risultati dell'impiego dell'approccio ADECO-RS per la realizzazione di oltre 73 Km di gallerie di linea", Strade e Autostrade, No.1 2006

[75] LUNARDI P., "Tunnelling under the Via Appia Antica in Rome", Tunnels & Tunnelling International, April 2000

[76] LUNARDI P., "Une methode de construction innovante pour elargir les tunnels routiers, autoroutiers et ferroviaires sans interrompre la circulation; son application au tunnel de Nazzano sur l'autoroute A1 Milan-Naples", Proceedings of the Conference on "Instandsetzung von Tunneln", Olten, 21 October 1999

Contents of the special focus boxes

Chapter 1

Chapter 2

Chapter 7

Chapter 8

Chapter 9

Chapter 10

HIGH SPEED RAILWAY SYSTEM • MILAN TO NAPLES LINE • BOLOGNA TO FLORENCE SECTION