Survey phase

The area through which the Dolonne tunnel passes is in the upper part of the Valle D'Aosta between Courmayeur and Dolonne at the point where the valley divides into the Val Ferret towards the north east, and the Val Veny towards the south west. It is one of the most geologically complex parts of the Alpine range, lying on the border zone between the Pennine systems which prevail in the central and western Alps and the Elvetico-Delfinese system that extends across the outer sector of the western Alps. The two systems are separated by the Pennine Front, a very large thrust fault of a regional character which in the area in question runs north east-south west with the dip to the south east (Figure 1).

The front runs roughly along the line of the bed of the River Dolonne. As a result the ground through which the tunnel passes is divided into two parts with very different characteristics:

- the Helvetic system consisting of granites and porphyroids, schists and clayey schists, limestones and liassic arenaceous limestones and;

Reliability rules at Dolonne

Final design specification of the Dolonne Tunnel on Italy's Aosta-Mont Blanc motorway was prepared to the principles of the analysis of controlled deformation in rock and soils (ADECO-RS). Prof. Eng. Pietro Lunardi, Lunardi's Geoengineering Design Office and Dott. Eng. Giovana Cassani, Rocksoil SpA give an outline of the project.

Winding its way from Morgex to Entrèves, the second section of the Aosta-Mont Blanc motorway runs for most of its 17km length underground, emerging briefly onto four short viaducts and a few embankments. Three twin-tube tunnels – Morgex, Pré Saint Didier and Dolonne – together run 12km, with a maximum ruling gradient of 3.2%, the result of a commitment by the design engineers to respect the environment. Siting the route underground also reduces maintenance problems, which are particularly severe in winter because of the very low temperatures and heavy snowfalls.

The adjacent tubes of the Dolonne tunnel, each with a cross section of $100m^2$, run along the right bank of the Dora Baltea river for 2,900m through extremely heterogeneous lithological formations with overburdens varying between 10m and 340m.

In drawing up a final detailed design, including connecting passageways, shelter recesses and vehicle stopping points, the motorway designer Spea Ingegneria Europea (SIE) had to comply with the recently introduced Merloni Law promoting transparency and reliability in contracting of public works, especially on forecasts of construction times and costs.

SIE brought in consulting engineer Rocksoil of Milan to revise the final design using the "Analysis of Controlled Deformation in Rocks and Soils" (ADECO-RS) approach, which has proven its reliability in predicting construction times and costs even under very difficult conditions (Lunardi, 2000).

- the lowermost portion of the Pennine system, characterised by the elevation of M Brisè consisting of black schists, chalks and cellular dolomites (Upper Trias), dolomites and dolomitic limestones (Middle Trias).

The geological survey to reconstruct the lithostratigraphy, the tectonic structure and hydrogeology of the area in detail, involved continuous core drilling and RQD evaluations on the samples, SPT tests, seismic refraction, and piezometric and permeability surveys. It also included a study of aerial photographs and numerous field investigations. It found:

- detritus nappes of varying thickness, resulting from glacial and post-glacial phenomena at the southern portal on the Mont Blanc side;
- Helvetic units consisting of schists and clayey schists of the Aaleniano;
- M Chetif units consisting of granites and pre-Triassic porphyoids;
- Helvetic units consisting of metamorphic crystalline limestones, schists and clayey schists of the Upper Lias;
- a large tectonic alignment (the Pennine thrust fault), mainly moraine detritus nappes where the tunnel crosses the River Dolonne, cellular dolomites, limestone schists and black graphite schists;
- nappes of detritus and denudation of varying thickness at the portal on the Aosta side.

Study of the geomechanics and the geological structure was carried out to characterise the materials and describe the main characteristics of the discontinuities (type, attitude, spacing, shape, opening, roughness, filling, strength, water conditions). A summary of the geotechnical and geomechanical characteristics of the different types of ground is given in Table 1.

The hydrogeology is characterised by a number of springs of varying size and quality scattered above the tunnel alignment along the River Dolonne valleys the right bank of the River Dora Baltea. Given the structural characteristics and permeability of the ground, a constant presence of water is probable during excavation. In the Helvetic area this should just show as small drips of water, but in the detritus ground at the portal on the Mont Blanc side and in the lowermost Pennine system there could be diffused and concentrated inflow.

Diagnosis phase

In the diagnosis phase the geological, geomechanical and hydrogeological information and predictions made using simple analytical methods of calculation (characteristic lines) were used to divide the tunnel alignment into sections with uniform stress-strain behaviour determining the stability of the face. These were:

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

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

- the stress state of the ground at the face and around the cavity would not have exceeded the natural strength characteristics 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, have an immediate effect and be in the order of a few millimetres;
- as a consequence, the face as a whole would have remained stable.

This is the case of the granite and porphyroid sections, which have excellent strength characteristics compared with the stresses brought into play by excavation under the overburden in question.

Category B 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 elastic-plastic range with the effect deferred in time and in the order of centimetres;
- as a consequence, the face would have remained stable in the short term at normal tunnel advance rhythms, with contained extrusion of the core at the face observed, but not enough 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 sections to be driven through Ultra-Helvetic ground (the schists and clayey schists of Aaeliano and of the Upper Lias, metamorphic crystalline limestones) as long as the overburdens and the hydrogeological conditions permit an arch effect to be created naturally.

Category C included the sections of tunnel where:

- 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 not have formed naturally either at the face or around the tunnel since the ground would not have possessed sufficient residual strength;
- deformation phenomena would have developed in the failure range, with the effect deferred in time and of the order of decimeters, giving rise to

Opposite: Full face excavation using the ADECO method for ground prediction, enabling a suitable support system to be applied

Below: Table 1

GROUND	g [kN/m³] (unit weight)	j [°] (angle of friction)	c [MPa] (cohesion)	S_{gd} [MPa] (uniaxial strength of the rock mass)	E [GPa] Modulus of deformability)	n Poisson's ratio	k [m/s] Permeability
Detritus nappes, portal M. Blanc side	19	25	0.05	0.2	0.15	0.3	$10^{-4} \div 10^{-5}$
Aaleniano schists and clayey schists	26	20 ÷ 30	0.24 ÷ 0.4	0.68 ÷ 1.39	1.9 ÷ 5.3	0.3	
Granites and porphyroids	27	27 ÷ 40	0.6 ÷ 1.0	1.94 ÷ 4.3	5 ÷ 19	0.27	
crystalline limestones	26	20 ÷ 34	0.14 ÷ 0.28	0.4 ÷ 1.1	3 ÷ 15	0.3	
Upper Lias schists and clayey schists	26	17 ÷ 28	0.12 ÷ 0.22	0.3 ÷ 0.7	2.2 ÷ 7.5	0.3	
Alluvial nappes and moraine deposits	20	33	0	/	0.12	0.3	$10^{-4} \div 10^{-5}$
Cellular dolomites	25	24	0.18	0.6	4.7	0.3	$10^{-4} \div 10^{-6}$
Grey limestone schists	26	22 ÷ 30	0.20 ÷ 0.25	0.6 ÷ 0.87	3	0.3	$10^{-4} \div 10^{-6}$
Black graphite schists	26	22 ÷ 30	0.20 ÷ 0.25	0.6 ÷ 0.87	3	0.3	$10^{-4} \div 10^{-6}$
Detritus nappes, portal Aosta side	20	33	0	/	0.12	0.3	$10^{-4} \div 10^{-5}$

serious instability, such as the failure of the face and the collapse of the cavity;

- as a result, without appropriate stabilisation the face would be completely unstable.

These conditions will be found at the portals and along most of the section on the Aosta side under shallow overburdens where the only way an arch effect can be created is artificially.

The predictions made for each tube of the tunnel have been summarised in Figure 1.

From the diagnosis it was clear that, apart from the initial sections at the portals, the tunnel could be divided from a stress-strain viewpoint into two sections characterised by very different behaviour: generally stable to the north of the River Dolonne and generally unstable, with the additional problem of water under pressure to the south of the river.

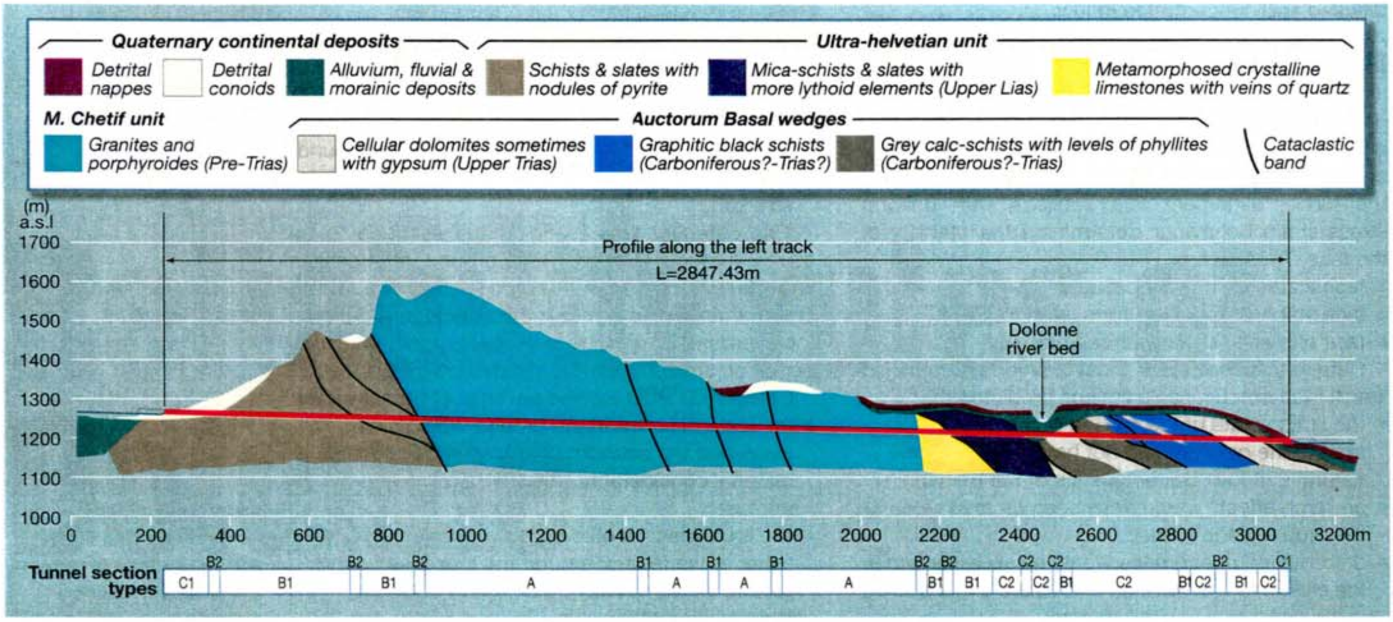

Fig 1. Moving from NW to SE, the tunnel alignment passes through extremely heterogeneous formations. The bottom row shows the tunnel section types

Therapy phase

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

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

- always full-face advance, especially under difficult stress-strain conditions;
- containment of changes to and decompression of the ground around the tunnel by immediate application of effective preconfinement and/or confinement of the cavity (sub-horizontal jet grouting, fibreglass structural elements in the core and/or in advance around the tunnel route and, if necessary, fitted with valves for cement injections, sprayed concrete) of sufficient entity to absorb a significant proportion of the deformation without collapsing, or to anticipate and eliminate the onset of any ground movement;
- placing a final concrete lining, reinforced if necessary, and complete with the casting of the tunnel invert when needed to halt deformation phenomena, in steps at a short distance from the face.

The tunnel section types designed were:

- for category A sections, a type A section consisting of end anchored roof bolts (if necessary), a preliminary 100mm-thick lining of fibre-reinforced sprayed concrete plus a layer of waterproofing consisting of non-woven fabric and PVC, a 400mm-thick final lining in concrete closed by a 500mm tunnel invert;
- for category B, two main tunnel section types were designed.

Type B1 consisted of full-length anchored roof bolts plus a preliminary lining of 200mm thick fibre-reinforced sprayed concrete reinforced with double steel ribs and a layer of waterproofing consisting of non-woven fabric and PVC and a 600mm thick final lining in concrete closed by a 700mm tunnel invert cast at a distance of not less than three tunnel diameters from the face.

Type B2 involved ground improvement of the advance core using fibreglass structural elements and drainage in advance where necessary, a preliminary lining of 200mm-thick fibre-reinforced sprayed concrete reinforced with double steel ribs, a layer of waterproofing consisting of non-woven fabric and PVC, a 600mm-thick final lining in concrete closed by a 700mm tunnel invert cast at a distance of not less than 1.5 tunnel diameters from the face;

- for category C (unstable face) four main tunnel section types were designed.

Type C1 consisted of a 100mm-thick layer of sprayed concrete on the face, either fibre-reinforced or reinforced with wire mesh, plus improvement of the ground around the advance core using sub-horizontal jet grouting, micro columns of improved ground in the advance core itself created using the same technology and reinforced with fibreglass structural elements (length 13m, overlap 4m), drainage in advance where necessary, a preliminary lining of 200mm-thick, fibre-reinforced sprayed concrete reinforced with double steel ribs, a layer of waterproofing consisting of non-woven fabric and PVC, a final lining in concrete in the crown with thickness varying between 400mm to 1300mm and closed by a 900mm tunnel invert cast in steps of between 6m and 12.5m in length at a distance of not less than 1.5 tunnel diameters from the face;

Northern portals of the twin-tube Dolonne tunnel on the Aotsa-Mont Blanc motorway.

- types C2, C2b and C2c consisted of a 100mm thick layer of sprayed concrete on the face, either fibre reinforced or reinforced with wire mesh, and improvement of the ground around the advance core using high pressure injections in advance through valved fibreglass tubes, plus reinforcement of the ground in the advance core itself using grout-filled and cemented fibreglass structural elements (length 13m (16m for sections C2b and C2c), minimum overlap 4m), drainage in advance where necessary, a preliminary lining of 200mm-thick fibre-reinforced sprayed concrete reinforced with double steel ribs and a layer of waterproofing which extends to cover the invert also consisting of non-woven fabric and PVC, an 800mm-thick final lining in concrete (proof against corrosion by sulphates) closed by a tunnel invert and 800mm-thick cast in steps at a distance of not less than 1.5 tunnel diameters from the face.

Precise specifications were made in terms of systems, advance rates and procedures including admissible extrusion and convergence values expected for each geological formation depending on the size of the overburden along the route. Also carefully specified were all factors required by the design engineer in deciding measures for preconfinement and confinement of the cavity and for modulating work during tunnel advance to adapt to the actual geomechanical conditions.

These detailed specifications also made it possible to adopt a quality assurance system conforming to ISO 9000 in the construction phase.

Drainage tunnel

To deal with the expected high levels of flow in the area close to the bed of the Dolonne River on the Aosta side, the design specified a drainage tunnel, 200m long and 50m^2 in cross section be driven in advance of the right hand tube starting at chainage 3+650.

This valved drainage tunnel is to reduce water heads which would otherwise reach 700mm to 800mm above the crown of the tunnel. In the long term it will ensure that groundwater can be captured and channelled back to the aquifers it came from.

Statics calculations

The statics and deformation behaviour of tunnels, both in the construction phases and the final service phase were analysed and verified by calculations on 3D and 2D models in the elastic-plastic range.

For grounds that respond to excavation with stone type behaviour (stable face or face stable in the short term) an important part of the analysis was focused on gravitational instability (wedges of rock isolated from the network of discontinuities) using the software package UNWEDGE 2.3 developed by the Rock Engineering Group of the University of Toronto

For grounds that respond to excavation with cohesive or loose soil type behaviour (face stable in the short term or face unstable) the investigation was performed using finite differences (FLAC 3.3) or finite element (ADINA 6.1.4) analysis. The models were developed to simulate the behaviour of the tunnel-ground system at different phases of tunnel advance as realistically as possible with particular attention paid (in the 3D models) to the effect of preconfinement treatment of the cavity and ground reinforcement in the advance core, to deformation values to be expected during excavation and to stresses on the preliminary and final linings.

Monitoring phase

The adequacy of design hypotheses was verified by geostructural mapping of the face and monitoring of the stress-strain behaviour of the face and the cavity, observed during construction at each of the stages and sequences specified in the design.

To achieve this, a complete monitoring system was devised that included as many as 145 systematic and 10 main stations for each tube.

The systematic monitoring stations (set at 50m intervals in category A sections of tunnel, 27m intervals in category B sections and every 9m in category C sections) involved:

- geological and structural surveys on the face;
- face extrusion measurements (in sections where C2 tunnel section types were used);
- convergence measurements;
- piezometric measurements around the cavity (in aquifers).

The main monitoring stations (placed in critical zones and at least one for each lithological section) were designed above all to measure probable stress-strain interactions between the two tubes and changes in the stress state of final linings. In addition to monitoring performed by the systematic stations they also included:

- three multi-base extensometers (two horizontal at the side walls and one vertical in the crown) for measuring radial deformation even after the placing of the final lining;
- two load cells at the foot of the steel ribs in the preliminary lining where specified;
- two sub-horizontal flat-jacks installed in the side walls of the final lining.

Given the considerable heterogeneity of the ground, systematic sub-horizontal core drilling (either continuous or with destruction of the core) was performed, 30m to 40m long, in advance of the face, on the Aosta side to obtain direct advance knowledge of the material to be excavated in zones with poor geomechanical conditions.

Contracts

Construction of the tunnel was awarded on the basis of the final design specifications as an all-in, lump-sum contract in which the contractor accepts all types of risks including geological risk. Work started a few months ago from both portals and has advanced hundreds of metres. T&T

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