DESIGN ASPECTS OF THE CONSTRUCTION OF THE NEW APENNINES CROSSING ON THE A1 MILAN-NAPLES MOTORWAY: THE BASE TUNNEL

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ABSTRACT : Improvements to Italian infrastructures include upgrading the section of the A1 Milan-Naples motorway between Sasso Marconi and Barberino di Mugello. This operation is particularly significant because it involves a large number of underground projects including the “Base Tunnel” which passes under the Apennines. The tunnel, consisting of two bores, each with a length of 8,500 m. approx., passes through ground belonging to the Monte Cervarola and Monte Modino sandstone formations and to the Scaly Clays formation under overburdens of up to 420 m.. A pilot tunnel driven by TBM for exploration purposes along a long section of the alignment allowed details of the geological and geomechanical picture to be obtained before the construction stage began.
This article describes the assessments made during construction design (employing the ADECO-RS approach) to make the final adjustments to section types. It also illustrates the advance method used, which involves the application of “Guideline” specifications to calibrate construction work on the basis of the results obtained from the monitoring system specially installed for that purpose. More specifically events are described when driving the tunnel close to the two portals, one through sandstones and the other through clayey grounds of poor geotechnical quality, with deep slip surfaces intersecting the excavation.

RESUME’ : Dans le domaine du développement des infrastructures italiennes, l’adaptation du trait entre Sasso Marconi et Barberino di Mugello de l’Autoroute A1, Milan-Naples, est particulièrement significatif pour la présence d’un nombre d’ouvrages en souterrain très élevé entre lesquels il y a la Galerie de Base qui représente le passage des Apennins. Le tunnel, constitué par deux tubes, chacun de longueur de 8500 m env., traverse amas principalement appartenants aux formations des Grés du Mont Cervarola et du Mont Modino, et à la formation des Argiles Ecailleuses en présence de recouvrements jusqu’à 420 m; à cet propos l’exécution par TBM de galeries d’exploration d’une grande traite de tunnel, a permis la définition du cadre géologique-géomécanique avant la phase de réalisation de l’ouvrage.
Dans le présent article, on décrit les évaluations effectuées en siège de projet de détail – en utilisant l’approche ADECO-RS – pour la mise au point des sections type. On illustre en outre la méthodologie d’avancement utilisée, qui prévoit l’emploi de « critères d’application » particuliers pour la mise à point des interventions à mettre en œuvre, en fonction des évidences du système d’auscultation. En particulier on décrit les expériences recueillies en cours d’œuvre dans les deux traites de tunnel à proximité des zones de tête, l’une dans des massif greseux de nature rocheuse, l’autre dans terrains argileux avec mauvaises caractéristiques géotechniques, intéressés par des surfaces d’éboulement profondes qui ont été traversées par les excavations.

1 - INTRODUCTION

Improvements to Italian infrastructures include upgrading the section of the A1 Milan-Naples motorway between Sasso Marconi and Barberino di Mugello. This operation is particularly significant because it involves a large number of underground projects, including the “Base Tunnel” which passes under the Apennines.
The upgrade project involves a total length of approximately 60 km. with an initial section of motorway up to La Quercia, where the existing motorway is expanded to 2 carriageways each with 3 lanes and an emergency lane. This is followed by a second section, named the “Variante di Valico” (new crossing), with a length of approximately 32 km. between La Quercia and Aglio, where a new section of motorway is constructed separate from the existing motorway consisting of two carriageways, each with 2 lanes plus an emergency lane. Finally there is a third section between Aglio and Barberino di Mugello where it is planned to use the two carriage ways of the existing motorway for the north bound traffic and to construct a new route consisting of 3 lanes plus an emergency lane (3 lanes without an emergency lane in tunnels) for south bound traffic towards Florence.
The Base Tunnel passes under the “Valico del Citerna” pass at 726 m. ASL on the “Variante di Valico” section between chainage 1+400 and 9+950 Km.. It has a length of approximately 8,550 m., with longitudinal gradients always less than 0.4% and planimetric radii of curvature of between 3,100 m. and 6,000 m.. It has two bores with the distance between centres varying from a minimum of 30 m. to a maximum of approximately 80 m., each with an inner diameter of 13.8 m. (160 m. approx. of excavation) and with each carriage way consisting of 2 lanes of 3.75 m, 1 emergency lane of 3.75 m plus lateral clearance of 0.25 m. on the right and 0.70 m. on the left. There are then two sidewalks with a width of 0.80 m. (figure 1). The two bores are connected by pedestrian passage ways every 300 m. of tunnel and passage ways for vehicles every 900 m. Safety measures are completed with lay-bays, 80 m. in length and 3.0 m. in width and S.O.S. bays at intervals of 150 m. on the right hand side of the carriage way.

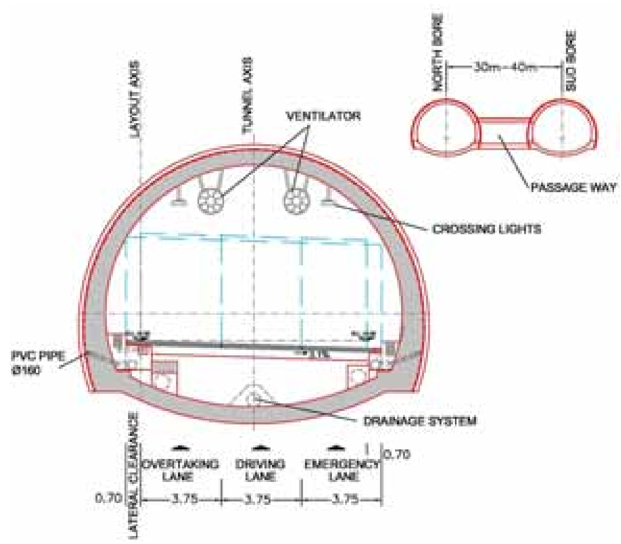

Figure 1 – Base Tunnel section type

An intermediate service tunnel is planned at chainage 5+985 Km (South bore) to speed tunnel advance rates during construction and this will perform safety access and ventilation functions when the tunnel is in service.

Geologically, the tunnel passes through ground belonging to the Monte Cervarola and Monte Modino Sandstone formations and the Scaly Clays formations with overburdens varying from a few metres at the portals to maximum overburdens of 420 m. (250 m, in the Scaly Clays formation). Reconstruction of the lithology and stratigraphy was performed on the basis of surveys and *in situ* investigation and in particular by using the geological and geomechanical data acquired when exploratory tunnels based on "pilot tunnel" principles (Lunardi, 1986) were driven along most of the tunnel alignment before the final design for the project was drawn up.

Construction of the tunnel was contracted out in lots 9, 10 and 11 of the "Variante di Valico" project on the basis of the final design documents, which were used as the basis for making bids.

The "RISALTO" consortium (Rizzani-Salini-Todini) won the contract and the contractor "Todini Costruzioni S.p.A", responsible for the fine detail of the construction work, asked the authors together with MaireEngineering for greater information on the construction methods specified in the design in the light of experience acquired in similar geomechanical contexts – primarily on the railway tunnels of the Bologna-Florence High Capacity Line (Lunardi et al., 2006, 2007) – and above all with regard to construction site organisation and logistics. A modification to excavation procedures was then proposed which optimised the tunnel section design and introduced flexibility to the intensity of intervention to be managed during construction on the basis of data acquired from the monitoring equipment in accordance with the A.DE.CO-RS approach (Lunardi, 2000, 2006). Alongside the design stage, which forms an intrinsic part of the approach, there is also the construction stage in which monitoring and interpretation of deformation phenomena are performed in order to fine tune the design. This approach led to a reduction in the estimated cost.

This article presents the stages that guided the definition of the excavation procedures according to the A.DE.CO-RS approach. The design stage consists of the "survey phase" in which the geological and geomechanical picture of the ground to be tunnelled is constructed, the "diagnosis phase" in which the deformation behaviour at the face without intervention is assessed and the "therapy phase" in which the stabilisation intervention to be performed (pre-confinement and confinement actions) and the sequence of the operational stages are defined. The construction stage involves monitoring deformation phenomena and the appropriateness of the intervention, which consists of specifying the "Guidelines" for calibrating intervention in the operational stages during construction on the basis of monitoring and experience acquired in the field.

2 – GEOLOGICAL AND GEOMECHANICAL PICTURE

The geological and geomechanical picture was drawn up on the basis of boreholes drilled along the tunnel alignment and the relative *in situ* tests and geostructural measurements performed on surface rock outcrops. However the most important source of data was from the "pilot tunnels" driven before 1999, precisely for the purpose acquiring a continuous definition of the predicted geological, geomechanical and hydrogeological profile. A brief summary is given below.

2.1 – Surveys conducted from "pilot tunnels"

The use of pilot tunnels is a very important method of conducting surveys when it comes to reducing risks in tunnel projects like this, characterised by the considerable length and large overburdens which make surface surveys impossible.

It is in fact possible to acquire precise information by conducting a systematic geomechanical survey of the walls of the excavation, to estimate the distribution of ground strength along the tunnel alignment and to perform *in situ* tests and take samples for laboratory tests. Finally by observing the deformation behaviour of the pilot tunnel it is possible to ascertain the type of behaviour that the rock mass will exhibit when widened to the full diameter.

The pilot tunnels were driven from the Badia portal (Bologna side) for approximately 4000 m. and from the Poggiolino portal (Florence side) for approximately 950 m.. No pilot tunnels were driven for the central part of the tunnel because work on them was suspended because of problems with the presence of gas.

The method used to excavate them was that of a rock TBM with a diameter of 3.40-4.00 m. except for short sections through surface deposits at the portals where conventional methods were used with steel ribs buried in shotcrete and the use of steel tubes in the crown. The average advance rates were between 12-14 m./day in the sandstones (maximum value 35 m/day) and 8-10 m./day in Scaly Clays.

Photo 1 – North portal pilot tunnel

Detailed information was acquired and recorded on technical sheets during tunnel advance on: the lithological characteristics and the fracturing of the rock masses; details of the primary and secondary structures present and the degree of fracturing and weathering; groundwater flows and the interception of pockets of gas. Instability of the profile of the excavation was measured precisely in terms of rockfall, the collapse of non cohesive ground and sizeable deformation and the volumes in play were assessed. Details of intervention performed to stabilise the tunnel were also recorded in terms of numbers and type of rock bolts, quantities of steel mesh and shotcrete and liner plates placed along particular sections with squeezing behaviour, particularly in the Scaly Clays formation.

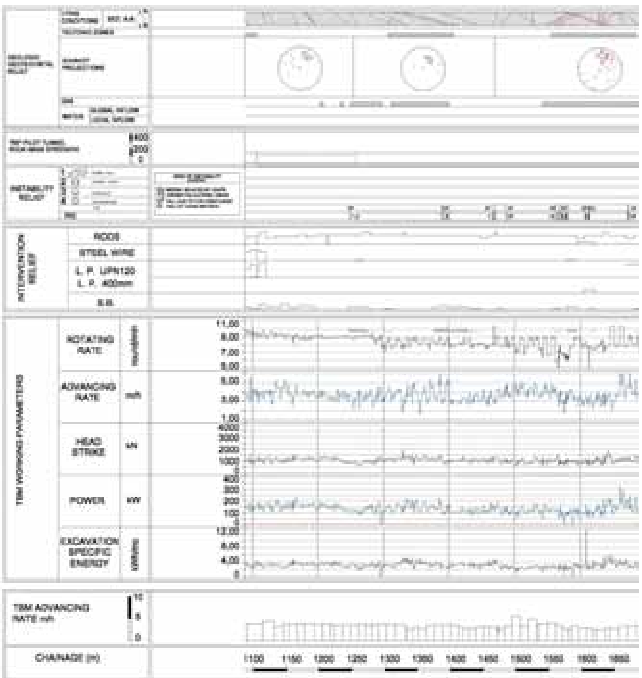

Figure 2 - Example of data acquired and technical sheets

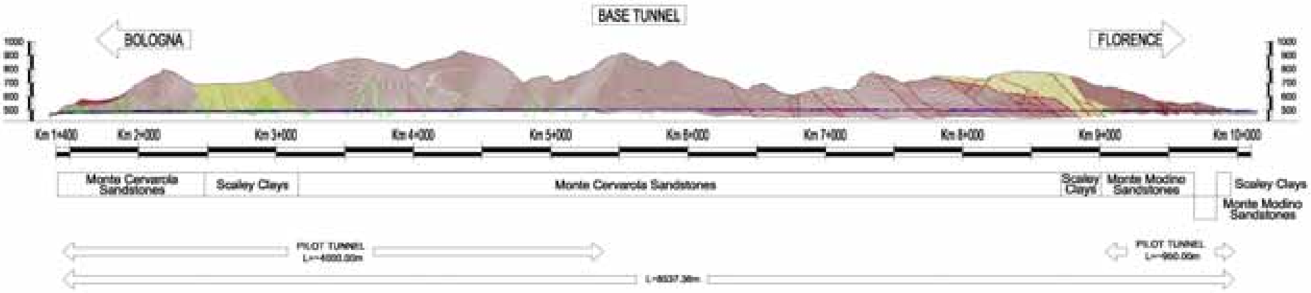

Figure 3 – Geological longitudinal tunnel section

Finally, the operating parameters of the TBM are also of particular interest for indirect assessment of the characteristics of the rock mass, such as the speed of advance, the cutter head thrust, the power consumed and information used to define the "specific energy of excavation".

2.1 – Geology

The alignment of the tunnel passes through the geology of the orogenetic chain of the Northern Apennines with reference to two principle domains: the "Tuscan Domain" to which the Macigno del Mugello belongs (and which comprises in turn the Monte Cervarola and Monte Modino Sandstone formations) and the "External Ligurian Domain" to which the Scaly Clays formation belongs.
The "Monte Cervarola Sandstones" consist of quartz-feldspar sandstones, organised in sedimentary cycles which include sequences of strata ranging from tens of centimetres to a few metres in thickness with alternating siltite and pelite layers from one to ten centimetres in thickness. The "Monte Modino Sandstones" of turbidite origin consist of thick strata of coarse to medium sandstones, with intervening strata, tens of centimetres in thickness, of fine sandstones and marly siltites grouped in banks of around one metre in thickness, which constitute the main characteristic of the unit. Sometimes marls or argillites are present.
Finally the "Scaly Clays" consist of argillites with a scaly structure and interbedding of generally thin (tens of centimetres) marly, calcareous strata. There are frequent discontinuous stone bodies in the unit consisting of flinty limestones, grey limestones (Palombini) and secondarily of ophiolitic green stones. They are very weathered in the zones closer to the surface due to the action of atmospheric agents and of groundwater and sometimes the weathering is accompanied by changes due to gravitational movements in progress or dormant.
Moving along the alignment of the tunnel from the North portal on the Bologna side, the tunnel passes through ground belonging to the Monte Cervarola formation for a length of approximately 1100 m. followed by the lithology of the Scaly Clays (650 m. approx.), and by the Cervarola Sandstones again for a long section until chainage 8+800 where there is an inclusion of the Scaly Clays set between two tectonic features. Continuing towards the South portal on the Florence side, the tunnel intersects the Monte Modino Sandstone formation and the Scaly Clays again in the portal section where the rock mass is of poor geomechanical quality as a result of deep gravitational movements.

2.2 – Hydrogeology

Driving the pilot tunnels also allowed problems relating to hydrogeology to be studied. The pilot tunnels functioned as drainage elements. Even if the tunnel was partially waterproofed, it was intended to have a drainage effect in order to prevent hydrostatic pressure from building up behind the cavity walls which the concrete linings would not be able to withstand.

2.3 – The geomechanics

Study of the strength and deformability of rock masses to be tunnelled is very important for predicting the stress-strain response of a cavity to excavation and this too was also performed in the light of the data acquired on the geomechanical characteristics of the rock (RQD, σc, spacing and orientation of the discontinuities, etc.) in the pilot tunnels. The RMR (Rock Mass Rating) indices (Bieniaswki, 1989) and the GSI (Geological Strength Index) values (Hoek et al., 2002) were defined for each uniform section of tunnel and the strength parameters according to the Mohr-Coulomb failure criterion were also calculated as a function of the size of the overburden or the *in situ* stress state.
Figure 4 shows changes in the value for GSI as a function of the overburden and the lithology. The values for the sandstones fall between 40 and 70, while the values for the argillites are lower in the 30-40 range.

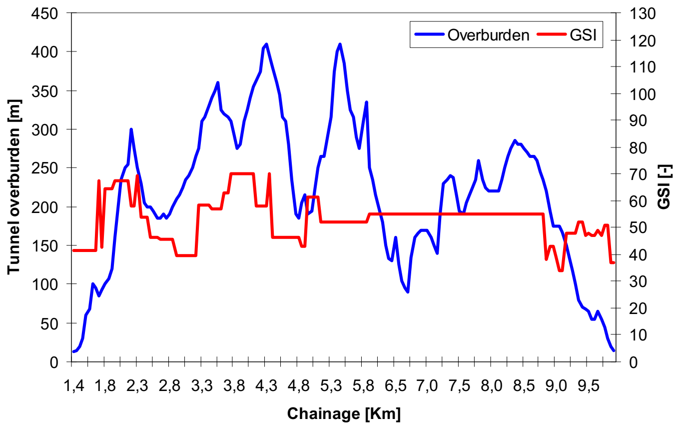

Figure 4 – GSI – Overburden section

The geomechanical picture and the parameters assumed for the design analyses are summarised in tables 1 and 2.

GEOLOGY	GEOMECHANICS			OVERBURDEN		HOEK-BROWN FAILURE CRITERION				
	RMR	GSI	GSI r	γ_n	H (MIN-MAX)	σ_{ci}	m_i	m	s	σ_{cm}
				kN/m³	m	MPa	-	-	-	MPa
M.CERVAROLA SANDSTONES	20-30	40 (35-45)	35	26	50-300	30	10	1.173	0.0013	4.19
	30-40	50 (45-55)	40	26	50-300	30	10	1.677	0.0039	5.23
	40-55	60 (55-65)	46	26	150-450	30	10	2.397	0.0117	6.55
	55-70	70 (65-75)	50	26	150-450	30	10	3.425	0.0357	8.43
SCALY CLAYS		30	*30*	25	170-250	8	5	0.41	0.0004	0.615
		35	*35*	25	170-250	8	5	0.491	0.0007	0.703
		40	*35*	25	170-250	8	5	0.587	0.0013	0.796
		45	*35*	25	170-250	8	5	0.7	0.0022	0.898
M.MODINO SANDSTONES	30-35	35	35	27	50 200	35	10	0.981	0.0007	4.35
	35-40	40	35	27	50 200	35	10	1.173	0.0013	4.889
	40-45	45	38	27	50 200	35	10	1.403	0.0022	5.467

Table 1 – Hoek-Brown failure criterion parameters

GEOLOGY	DEFORMABILITY			MOHR-COULOMB FAILURE CRITERION			
	E peak	E resid	ν	ϕ peak	ϕ resid	c' peak	c' resid
	MPa	MPa	-	°	°	kPa	kPa
M.CERVAROLA SANDSTONES	3080	2300	0,25	34-47	32-45	270-750	240-700
	5400	3080	0,25	37-50	34-47	350-935	270-750
	9700	4790	0,25	36-45	35-49	810-1430	20-850
	13000-17000	5400	0,25	39-47	37-50	1120-1780	350-935
SCALY CLAYS	900	900	0,35	18-20	18-20	200-250	200-250
	1200	1200	0,35	19-22	19-22	220-280	220-280
	1600	1200	0,35	20-23	22	250-310	250
	2100	1200	0,35	21-24	22	250-350	250
M.MODINO SANDSTONES	2500	2500	0,3	36-47	36-47	260-600	260-600
	3300	2500	0,3	38-48	36-47	300-660	260-600
	4400	2500	0,3	39-50	36-47	350-730	260-600

Table 2 – Mohr-Coulomb failure criterion parameters

Table 1 contains details of the RMR and GSI values for each geological formation along with the relative Hoek-Brown failure criterion parameters for them. Table 2 gives the range of the strength parameters according to the Mohr-Coulomb criterion and the deformability characteristics of the rock masses.

3 – DESIGN PROBLEMS AND CONSTRUCTION METHODS

Design study was based on the need to identify methods of tunnel advance which would guarantee the stability of the face and the tunnel profile in the long and short term and also allow the construction process to be industrialised by adopting a flexible excavation system able to handle changes in geological and geomechanical conditions encountered by simply varying the intensity of a few basic operations. The objective was to simplify construction site operations in terms of supplies, equipment and work schedules in order to speed construction times and therefore reduce costs.

In order to decide which stabilisation systems were to be applied, the deformation behaviour of the tunnel at the core-face was calculated (extrusion, pre-convergence, convergence) in the absence of intervention ("diagnosis phase"). This allowed the behaviour category of the "core-face" to be determined: category A "face stable", category B "face stable in the short term", category C "face unstable". The rigidity of the core of ground ahead of the face has a decisive effect on the deformation response of the tunnel system and it determines how an "arch effect" is triggered. The predictions were made by using the Characteristic Line Method (Amberg and Lombardi, 1974), and examining the characteristic line for the face, which takes account of the three dimensional effect of stresses at the face.

The following observations were made: in rock masses belonging to the Monte Cervarola and Monte Modino formations and in the most recurrent geomechanical contexts with GSI in the 50-60 range radial convergence was encountered at the face of between a few millimetres and up to 2-4 cm. where the overburdens were above 300 m.. When a new face is excavated the rock mass remains in the elastic field or with very contained plasticisation, which extends for one tunnel radius. The result is "stable" type "core-face" behaviour (Category A) or "stable in the short term" behaviour (Category B). More accentuated deformation phenomena was only found in fairly fractured rock masses (GSI <40), in the presence of overburdens greater than 250-300 m, with radial convergence of tens of centimetres and the band of plastic ground at the face greater than 2.5-3.0 tunnel radii.

In rock masses belonging to the Scaly Clays formations present along the tunnel alignment and with overburdens varying between 170 m. and 250 m., radial convergence at the face was found in the 5-14 cm. range, associated with plasticised bands of ground of 1.5-3.5 tunnel radii and GSI values in the 40-45 range, while unacceptable radial convergence at the face (greater than 50 cm) and extensive plasticisation was found for GSI values in the 30-35 range. The general result is "core-face" behaviour mainly of the "unstable" type (Category C), with "stable in the short term" behaviour (Category B) for GSI values in the 35-40 range in the presence of overburdens of less than 200 m.

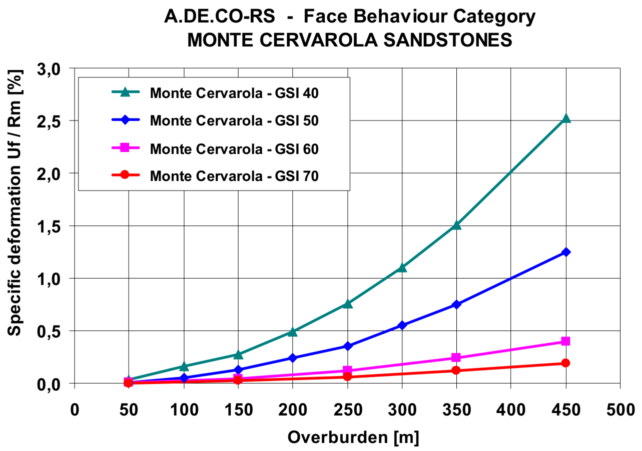

Figure 5 – Monte Cervarola Sandstones – Specific deformation

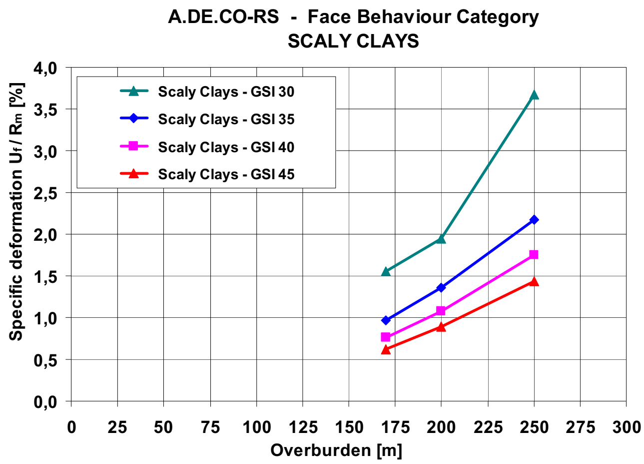

Figure 6 – Scaly Clays – Specific deformation

Figures 5 and 6 show "specific deformation" at the face (defined as the ratio of convergence at the face to the tunnel radius) as a function of overburdens and GSI values. Very contained specific deformation was observed for the Monte Cervarola Sandstones of less than 0.2-0.4% and behaviour was practically linear in relation to the overburden for GSI values of 55-60. Contained deformation was observed for GSI values in the 40-50 range and behaviour was linear in relation to the overburden up to 150-200 m. A non linear relationship with deformation was then observed for the maximum overburdens in the 0.8-1.2% range for GSI values of 50 and 1.4-2.0% for a GSI of 40.

Markedly higher specific deformation in the 0.8-2.2% range for the Scaly Clays was found with the behaviour of the deformation curve deviating widely from the linear for overburdens of greater than 200 m. Maximum specific deformation of 3.6% was reached for GSI values of 30 and overburdens of 250 m.

The operations which constitute the tunnel section advance types were defined in order to guarantee the long and short term stability of the tunnel in the light of predictions of face deformation behaviour. It was performed for each geomechanical context, on the basis of the magnitude of the overburden along the tunnel alignment.

A few basic decisions were made. The first was to adopt “full face” advance even for the most difficult conditions, in order to be able to promptly stabilise the entire core of ground at the face and close the cavity by placing the primary lining and casting the tunnel invert. Fibre glass structural elements 24 m. in length were used for preconfinement of the core and for ground reinforcement around the cavity, if necessary. The length of each advance forward was varied as a function of the extension of the deformation beyond the face. Steel ribs and shotcrete were placed for cavity confinement with the use of Superswellex rockbolts limited to sandstones with good geomechanical characteristics only. Finally one important design factor is the regulation of the distances from the face at which the crown and tunnel invert linings are cast. Groundwater control was performed during construction using micro-slotted drains placed sub-horizontally before the fibre glass elements. The tunnel was fitted with a PVC mantle in the crown and drainage channels to collect water at the foot of the walls.

In detail, where “stable” or “stable in the short term” “core-face” behaviour was identified with contained specific deformation (<0.4-0.6%), confinement of the cavity walls only was specified by using a preliminary lining (shotcrete over steel ribs and/or rock bolts): tunnel advance section types A and B0. For specific deformation values of greater than 0.6-0.8%, preconfinement was also specified using structural fibre glass elements (section types B2, B2V and B0V, the latter with steel tubes in the crown as pre-support for zones with highly fractured rock or for local support for limey-marly inclusions in the argillites). Finally for specific deformation of greater than 1.0-1.5% with unstable “core-face” behaviour, the pre-confinement of the face was accompanied by ground improvement ahead of the face placed around the profile of the excavation, again by means of fibre glass structural elements, injected under pressure or cemented with expansive mixes (section types C2 and C6). The use of expansive mixes with the coercive force induced by the expansion of the mixes allows the stress-strain conditions of the rock mass around of the profile of the excavation to remain undisturbed, thereby limiting the decompression of the materials and the deterioration of the geomechanical characteristics.

MONTE CERVAROLA SANDSTONES		OVERBURDEN							
		<50	50-100	100-150	150-200	200-250	250-300	300-350	350-450
GSI	<45	A	A	A	B0	B0	B0V	B0V	B0V
	45 ÷ 55	A	A	A	B0	B0	B0V	B0V	B0V
	55 ÷ 65	A	A	A	A	A	B0	B0	B0V
	>65	A	A	A	A	A	A	A	A

Table 3 – Monte Cervarola Sandstones – Section type criteria

SCALY CLAYS		OVERBURDEN		
		<170	170-200	200-250
GSI	<30	C2	C2	C2
	30 ÷ 35	C2	C2	C2
	35 ÷ 40	C6	C6	C6
	>40	B2 B2V	B2 B2V	C6

Table 4 – Scaly Clays – Section type criteria

The tables 3 and 4 give the criteria for applying the tunnel section types as a function of the geomechanical context and of the overburdens present along the alignment. Examples are given in figures 7 and 8 of section types B0 and C2 considered the most representative of the two geological formations tunnelled. The use of section types along the alignment was identified in the prediction of the geomechanical profile.

4 – GUIDELINES FOR TUNNEL ADVANCE AND MONITORING SYSTEMS

In order to verify the appropriateness of the tunnel section types specified during construction and to calibrate them to fit the geomechanical context more precisely as it varies, a system of monitoring was drawn up along with a procedural document named the “Guidelines”.

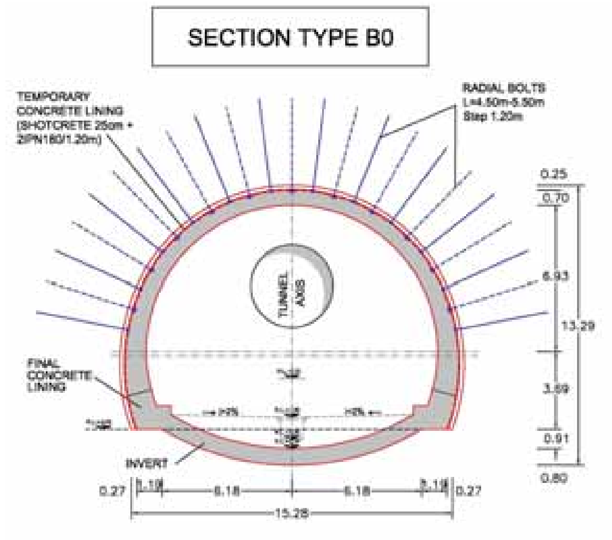

Figure 7 – Section type B0

The monitoring system is designed to systematically acquire information on the geostructural conditions of the face and the deformation response of the tunnel system during excavation. A technical sheet is filled in each day for the face on which lithological, geostructural and geomechanical information is entered along with schematic diagrams and photos. The data acquired can be used to monitor predicted values for RMR and GSI and examine the local stability of the face in the light of the system of discontinuities present. Measurement stations which detect absolute displacement of the preliminary linings are used to monitor the deformation response (convergence, settlement, transverse movements) by taking topographical readings with the frequency dependent on the type of rock mass (1 station every tunnel advance step, 8-12 m, for type C core-face behaviour, 1 station every 25-50 m. for type B and A core-face behaviours respectively). In Scaly Clays rock masses, where the use of preconfinement of the face is specified, face deformation (extrusion) is also monitored using incremental extensometers 36 m. in length. These are fundamental instruments for sections of tunnel where no pilot tunnel is present. Readings are taken on a daily basis for deformation gradients of greater than 1 mm. per day and weekly where stabilisation is in progress.

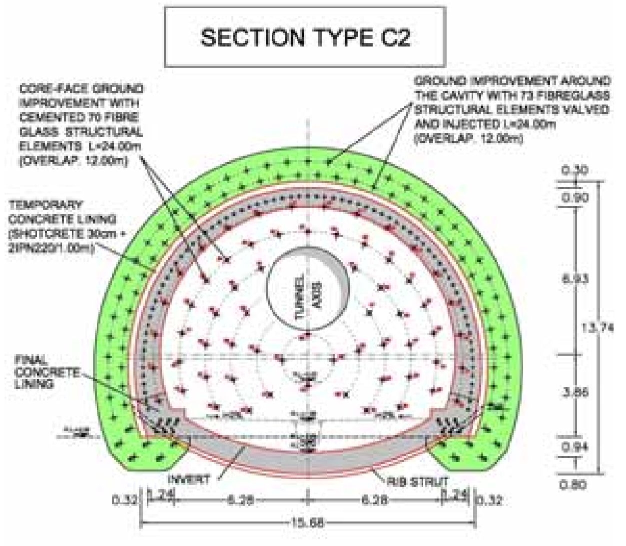

Figure 8 – Section type C2

Finally special stations for monitoring deformation and the plastic band of ground around the cavity (multibase extensometers) and the stress state of the preliminary and primary linings (strain auges installed on steel ribs or inside the concrete lining) are specified for particularly critical sections.

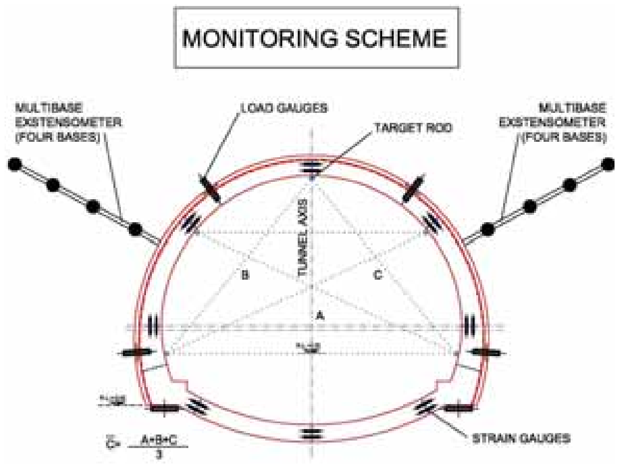

Figure 9 – Monitoring system on a section of tunnel

Once the geomechanical and deformation conditions are known, they can be compared with design predictions and the following decisions can be made:

a) confirm the tunnel section type specified according to the intensity of the intervention and sequence of operational stages given in the design;
b) confirm the tunnel section type and vary the intensity of the intervention or the sequence of the operational stages in order to adapt them to fit the geomechanical conditions encountered during construction more precisely ("transverse variability" of operations, with the tunnel section type constant);
c) select a different tunnel section type from among those specified in the design if the lithological and geomechanical conditions encountered diverge from those forecast for that section of tunnel ("longitudinal variability", with regard to the use of tunnel section types along the tunnel alignment);
d) report the need to design a new tunnel section type with different technical characteristics in cases where completely unforeseen lithological and geomechanical conditions are encountered or unexpected behaviour are encountered.

Driving pilot tunnels as a method of survey along the tunnel alignment does in fact make it possible to limit management of variability to points a) and b) and in isolated cases to points c) and d) where cases c) and d) arises not so much as a result of different lithological conditions, as the possibility of a markedly different response to excavation with the larger scale tunnel than that found with the geometry of the pilot tunnel.

The magnitudes that can be regulated during construction with regard to point b), are principally as follows: the number of fibre glass elements at the face and the length of overlap (i.e. the length of each tunnel advance); the length and geometry of the rock bolts for tunnel section type A and B0; the length of each tunnel advance and the interval between steel ribs; the placement of steel ribs in the tunnel invert; the distance from the face at which the final linings for the tunnel invert and the crown are cast.

The criteria employed to apply the "variability" are contained in the design in the "Guidelines" manual. As already stated, the factors on which judgements are based are those of the geomechanical context and the deformation response which also takes account of the overburden. Numerical analyses conducted using both the Characteristic Line Method and more refined calculation models performed using FLAC calculation software were employed to assess the range of deformation behaviour for each tunnel section type: diametrical convergence and expected extrusion. Table 5 gives the values calculated.

Section Types	Geology	Convergence (cm)	Extrusion (cm)
A	Monte Modino Sandstones	**2-3**	**Negligible**
B0		**3-5**	**Negligible**
B0V		**5-10**	**< 3**
B2	Scaly Clays	**8-12**	**< 6**
B2V		**6-10**	**< 5**
C2		**10-14**	**< 10**
C6		**8-12**	**< 8**

Table 5 – Deformation ranges vs section types

If the deformation values recorded are in the centre of the range predicted, "nominal" intervention is adopted using the design tunnel section types. Otherwise the intensity of intervention can be reduced if convergence and extrusion is near the lower limit of the predicted range (minimum), or the intensity can be increased (maximum) if the values are close to the upper limit ("transverse variability"). If the increase in the intervention is not sufficient to maintain the deformation response within the predicted range, then the tunnel section type specified must be changed ("variability longitudinal").

Moreover, at a geomechanical level and aside from the GSI values, the structural conditions are carefully examined for some types of intervention: e.g. Superswellex rockbolts are only used in the tunnel walls in the presence of banks of some tens of centimetres and tight joints with no clay filling and the absence of frequent joints with landslip/landslide geometry and tectonic disturbance factors (folds, faults).

Table 6 below gives the main "variabilities" for tunnel section types B0 and C2 as an example.

Section Types	Intervention	Variabilities		
		Minimum	*Nominal*	*Maximum*
BO	Steel rib step	1.4 m	1.2 m	1.0 m
	Bolt length	4.5 m	5.0 m	5.5 m
	Bolt mesh (*)	2×1.4 - N	2×1.2	2×1.0

	Invert-face (°)	Not linked	< 5.0∅	< 4.0∅
	Crown-face	Not linked	< 9.0∅	< 6.0∅

(*) No bolts are used for GSI > 50
(°) Invert-face cast distance, with ∅ = tunnel diameter

Section Types	Intervention	Variabilities		
		Minimum	*Nominal*	*Maximum*
C2	Steel rib step	1.2 m	1.0 m	0.8 m
	N°VTR face	50	70	90
	VTR face overl.	10.0 m	12.0 m	14.0 m
	Excavation	14.0 m	12.0 m	10.0 m
	Invert-face (°)	< 2.0∅	< 1.5∅	< 0.5∅
	Crown-face	< 3.0∅	< 5.0∅	< 7.0∅

Table 6 – B0 and C2 section types variabilities

The guidelines are therefore like an "instruction manual" designed to furnish the necessary flexibility in the design during construction with the objective of calibrating intervention to fit local situations while observing safety conditions. They are also a very important instrument for controlling construction times and costs and avoiding waste with a careful distribution of resources.
We consider that, in combination with pilot tunnels, they constituted a valid instrument for managing the risks attaching to an important project like the one considered here.

5 – THE CONSTRUCTION STAGE

Once approval was obtained for the modification to the design, the contractor organised the construction of the tunnel in three principal areas: two faces advancing from the North on the Bologna side (Badia construction site); two faces advancing from the South on the Florence side (Poggiolino construction site) and the face for the service tunnel (Roncobilaccio construction site) designed to serve the four intermediate faces.
Construction work started from the North portal on the Bologna side, where tunnel advance took place by direct excavation of the rock slope through the terrain belonging to the Monte Cervarola Sandstones.
The design predictions were basically confirmed by the progress made with excavation, with prevalent use of tunnel section type B0 and section type A employed for good rock-mass zones or low overburden.

Photo 2 – Monte Cervarola Sandstones – Section type B0

Constant analysis of technical sheets for tunnel section type B0 and of the convergence values recorded allowed the magnitude of cavity confinement action to be varied by modifying the length of tunnel advances and as a consequence the interval between steel ribs.

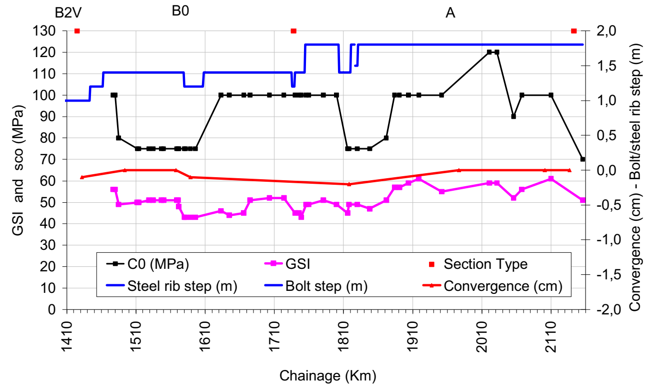

Figure 10 – North portal 1+410–2+160 section - Construction data

While the average design value was 1.20 m., long sections of tunnel were driven with steel ribs placed at intervals of 1.40 m. and some short sections were even driven with ribs placed at intervals of 1.60 m., because the geomechanical characteristics were higher than the range of application for that tunnel section type. Tunnel section type A was employed where the geostructural conditions were favourable. Figure 10 gives a summary of the principal data observed between portal chainage 1+410, and chainage 2+160 on the South bore.
As can be seen the GSI remained within the 40-60 range, with values markedly higher than 50-55 in the section between chainage 1+830 and chainage 2+160 where tunnel section type A was employed. Similarly the mono axial strength of the rock, σ_{co}, measured promptly at the face using a Schmidt hammer was always higher than 75 MPa, with values even reaching 100-120 MPa. Finally negligible values were recorded for convergence, always less than one centimetre, which showed that the behaviour of the rock mass was in the elastic range with immediate deformation measurable in millimetres.

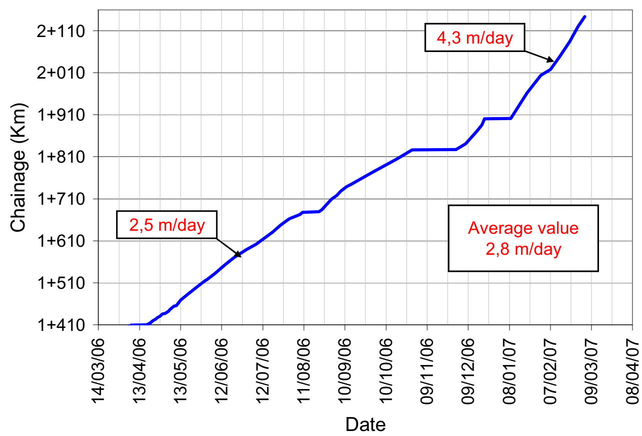

Figure 11 – North portal 1+410–2+160 section – Advance rate

Figure 11 shows the rate of advance. Production was virtually constant between 2.5 m./day and 4.3 m./day (an average of 2.8 m./day, holydays included), but the choice of design was to industrialise construction as much as possible with repetitive work cycles in which adjustments to improve operations were made only on the basis of local conditions. Simultaneous operation on two faces, the North bore and the South bore allowed more functional and efficient management of manpower and equipment resources, to give better control over production times and costs.

Photo 3 – Scaly Clays – Section type C2

Once the portal wall was completed excavation also started from the South on the Florence side. The geological and geomechanical conditions were immediately found to be worse than predicted by the design due to the geomorphological and geotechnical complexity of the area, note completely taken into account by the pilot tunnel investigation. The deformation response to excavation was of much higher magnitude than expected, especially at ground level where the network of topographical datum points and the installation of deep inclinometers identified a deep slip surface which, given the thrusts from the slope had a negative effect on the tunnel. After an initial collapse of the tunnel, the tunnel section type C2 was strengthened by increasing the number of reinforcements specified and above all by also employing repeated high pressure injections in the structural reinforcement elements in the face to improve the strength of the rock mass, especially in the zone where the slip surface was located. This was performed by fitting the fibre glass structural elements with two valves each metre to give average absorption recorded of 50-60 litres/valve (maximum values of even 120 litres/valve) with injection pressures of 8-14 bar.

Photo 4 – Section type C2 – Full face tunnel advance

Some of the more important construction phases are illustrated In photos 3÷7: the reinforcement of the core using fibreglass elements at the tunnel face (3), a full face tunnel advance (4), placing steel ribs(5), reinforcing and casting the tunnel invert (6) casting the final lining (7).

Photo 5 – Section type C2 – Placing steel ribs

In addition to increasing the face preconfinement action, cavity confinement was also increased by reducing the intervals at which steel ribs were placed to 0.80 m. and by casting the tunnel invert closer to the face. These design corrections required the introduction of a new tunnel section type termed C2Vp (Case d) of the guidelines). Careful reading of monitoring instrumentation allowed tunnel advance to be controlled and above all the length of tunnel advances to be calibrated, with excavation interrupted when diametrical convergence reach 6-8 cm during the tunnel advance.
The graph in figure 12 shows the advance of the face, the subsequent casting of the tunnel invert and the convergence values recorded by the relative measurement stations. It can be seen that diametrical convergence increased up to a maximum of 15-16 cm, as the face approached the position of the slip surface.

Photo 6 – Section type C2 – Reinforcing and casting the tunnel invert

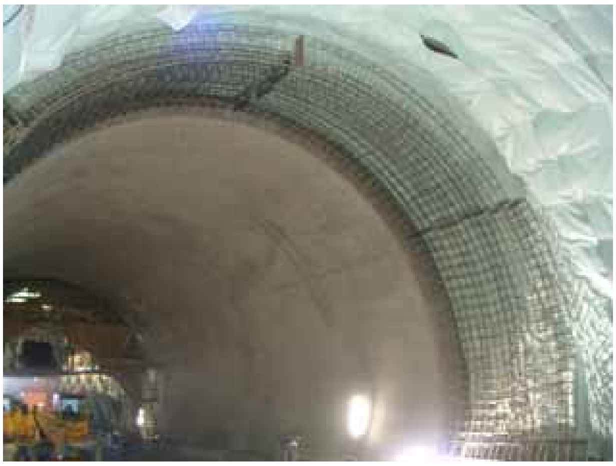

Photo 7 – Section type C2 – Casting the final lining

The favourable stabilisation action exerted by casting the tunnel invert close to the face can also be seen. The instrumentation inside the tunnel was accompanied by instruments installed on the surface in consideration of how important it had become to monitor the deformation response of the slope to excavation.

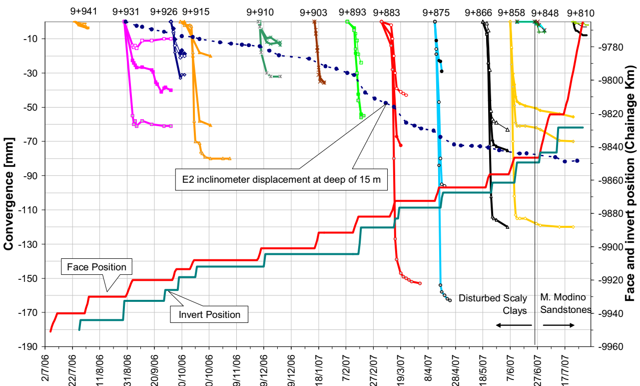

Figure 12 – South portal 9+950–9+765 section – Convergence

Figures 13 and 14 show the settlement of the datum points at ground level and of the deep inclinometers. Very high values of up to 30 cm. were also recorded for surface settlement with a strong reduction in the settlement gradient after the tunnel invert is cast, a gradient which reduces to zero once the final lining of the crown is cast. Note more than 40%-50% of the final values occur before face reaches the targets.

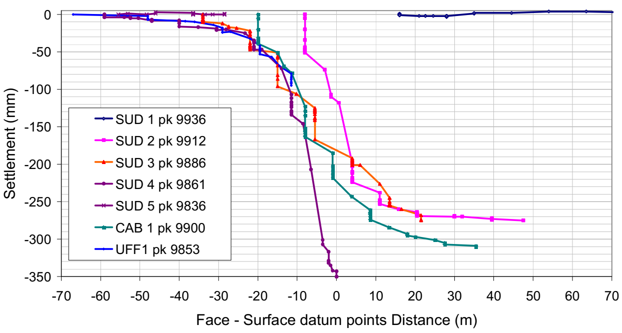

Figure 13 – South portal 9+950–9+765 section – Surface settlement

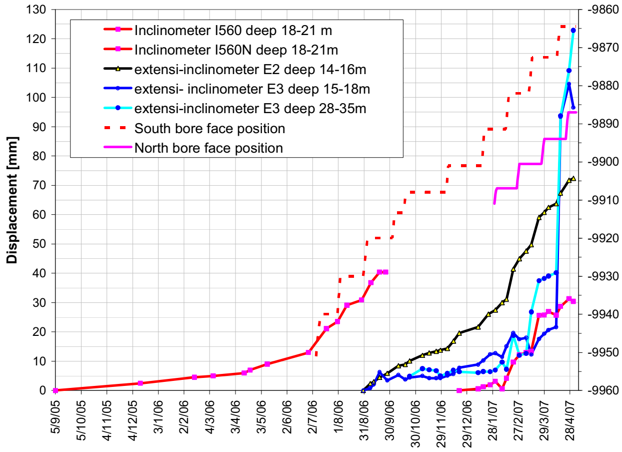

Figure 14 – South portal 9+950–9+765 section – Inclinometer displacements

Photo 8 – Section type C2 – Final lining-face distance

The change of tunnel section type following observations in the field prevented tunnel advance from coming to a halt, although with lower than expected production rates. Once the section affected by the surface slip had been passed, tunnel advance proceeded according to design predictions.

6 - CONCLUSIONS

The base tunnel is currently under construction as part of the works to modernise the new Naples to Milan motorway on the Sasso Marconi and Barberino di Mugello section. The A.DE.CO-RS method was used to develop the construction design for this tunnel by identifying the best methods of tunnel advance for controlling the deformation response to excavation and for guiding advance towards conditions of stability. Examination of the expected behaviour was performed on the basis of the geological and geomechanical data acquired when exploratory tunnels were driven before work to widen them was performed using a “pilot tunnel” approach. This is an extremely valuable geological surveying method because it acquires continuous information along the alignment of a future tunnel which reduces geological uncertainties and allows more accurate predictions of construction times and costs to be made. The advance procedures took account of construction site logistics and organisation requirements and this had two effects. It facilitated “industrialisation” of the excavation process by employing a few basic technologies such as full face advance and the use of fibre glass structural elements as a means of exerting face preconfinement and it also enabled the necessary “flexibility” to calibrate operations during construction to fit the actual real conditions encountered. To achieve this the design involved the preparation of a specific document entitled

“Guidelines” which furnished criteria for the implementation of tunnel section types based on the data acquired by the monitoring system installed during tunnel advance (geomechanical measurements of tunnel faces, tunnel convergence and face extrusion measurement stations). The implementation is varied in terms of the number and overlap of ground reinforcement elements, the distance between steel ribs and the distance from the face at which the tunnel invert is cast. Attentive technical assistance provided during construction ensured proper interpretation of the monitoring data acquired as the instrument for verifying the type of advance adopted and defining new intervention to adopt if necessary. These construction procedures also increase the efficiency of resource use with strong synergies between the design stage and the construction stage to allow more accurate management of construction times and costs.

PROJECT DATA

Client	**Autostrade per l'Italia**
Final design	**Spea Ingegneria Europea**
Contractor	**RISALTO – Todini Costruzioni**
Construction design	**MaireEngineering with RockSoil**
Construction supervision	**Spea Ingegneria Europea**
Technical assistance on site	**RockSoil**

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