The largest TBM–EPB machine in the world, designed to the Appennines. The experience of the Sparvo Tunnel

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Summary

This paper describes the geological and geotechnical assessments performed to select the type of TBM to employ for the excavation of the "Sparvo Tunnel", which has an exceptional outer diameter of 15.62 m. and lies on the A1 Milan-Naples motorway in the section between La Quercia (Bologna) and Barberino di Mugello (Florence). The technical characteristics and operating parameters are identified on which the design of the machine were based, with particular attention paid to the most critical geotechnical context consisting of the *Argille a Palombini* formation. Considerations are also made concerning the management of interference between the excavation and the presence of deposits of underground gas.

Keywords: *TBM-EPB, face chamber-pressure, cylinders' thrust, gas*

1. Introduction

Improvements to Italian infrastructures include upgrading the section of the A1 Milan-Naples motorway between Sasso Marconi (Bologna) and Barberino di Mugello (Florence). This operation is particularly significant because it involves a large number of underground projects including the "Sparvo Tunnel" which, because of its length, the diameter of the excavation and the geological context, is critical for opening the section of motorway between La Quercia (Bologna) and Barberino di Mugello (Florence). The underground alignment is approximately 2,600 m. in length and twin bore. It runs through a slope affected by numerous dormant land slips and active local phenomena – at times very extensive and large such as the *Frana di Sparvo* (Sparvo land slip) – and through geological formations belonging mainly to the *complessi di base liguridi*, such as the *Arenarie dello Scabiazza* and the *Argille a Palombini*.
The difficult geomechanical conditions, especially in the section running through the *Argille a Palombini* and slope deposits, and the consequent uncertainties concerning the speed of tunnel advance led the Contractor, in agreement with the Client, to consider the use of a TBM. It was decided to excavate with an EPB type TBM. Although this decision involves a longer start-up period before construction of the underground tunnel can commence (in terms of design, construction, transport and assembly of the machine), it has the advantage of subsequently allowing much faster tunnel advance rates per day, with a positive overall aggregate result compared to conventional excavation methods. Compared to the 6-8 months required to prepare the portal for conventional excavation methods, the use of a large diameter TBM requires at least 14-16 months for the construction and assembly of the machine at the tunnel portal. However, during excavation it can maintain average advance rates in the range of 10-12 metres per day compared to 1.0-1.2 metres per day in poor quality ground and up to 3-4 metres per day through rock using conventional methods. The use of a shielded TMB has the advantage of heavily reducing risks for the safety of workers who, with this configuration, always work in a protected environment, while it also minimises risks resulting from instability at the face and in the cavity. It must nevertheless be underlined that the most delicate issue considered in the selection of the excavation

method was the study of changes in the stress-strain state of the rock mass during excavation in the *Argille a Palombini* formation. This possesses poor strength and deformation parameters and with overburdens of 100-120 m. along the central part of the tunnel alignment, "squeezing" phenomena are highly probable, which could cause serious problems to the advance of the machine. It was therefore necessary to perform thorough assessments in order to determine the technical characteristics and operating parameters on which to base the dimensions of the machine (geometry, maximum thrust, pressure at the face, etc.). The analyses were conducted using conservative hypotheses to produce dimensions for the TBM which would stand up to very severe conditions, even if – in consideration of aspects that were unpredictable and programmable in the numerical analyses, such as residual stresses in the rock mass due to tectonic stress, strong local anisotropies and the considerable diameter of the excavation in material subject to squeezing – the functioning of the TBM was studied using risk analysis, since it is not possible to eliminate the risk that the design forecast pressures are exceeded during tunnel advance and the TBM becomes blocked as a consequence.
Lastly, one far from negligible aspect for the safety of excavation operations is the management of interference with underground gas deposits, the presence of which is recognised in the rock masses concerned, especially in the interfaces between formations.

2. Project data

To give an idea of the project, the Sparvo Tunnel has a length of approximately 2600 m.,with longitudinal gradients always less than 4% and planimetric radii of curvature of between 1,400 m. and 3,000 m.. It has two bores with the distance between centres varying from a minimum of 30 m. to a maximum of approximately 70 m., each with an inner diameter of 13.6 m. 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 0.60 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 S.O.S. bays at intervals of 150 m. on the right hand side of the carriage way.

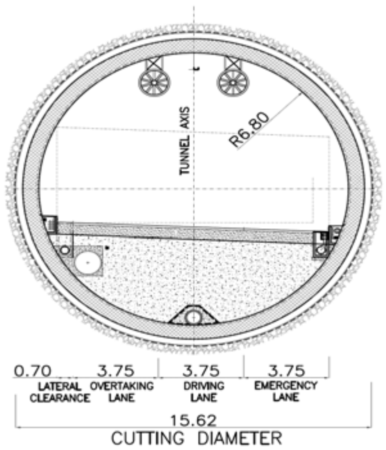

Fig. 1 *Sparvo Tunnel section layout*

Client	**Autostrade per l'Italia**
Contractor	**TOTO Costruzioni Generali**
TBM Supplier	**Herrenknecht AG**
Construction design	**Rocksoil**
Construction supervision	**Spea Ingegneria Europea**
Technical assistance	**Rocksoil**

Table 1 *Project data*

The final lining will consist of rings of prefabricated concrete segments, numbering 9+1 with a maximum weight of 165 KN, a length of 2.0 m. and thickness of 0.70 m., with reinforcing steel bars. The segments are fitted with EPDM gasket along their entire surface and with hydrofiller proofing elements. The concrete segments are connected by fastenings located on the outer sides (steel bolts) and the longitudinal side (steel pins). A guide bar is used to ensure that they are properly aligned. The extrados of the lining will be backfilled with a fast setting two component grout, for an excavation diameter of 15.62 m. and a total excavation area of approximately 190 m. Table 1 shows those involved in the construction of the tunnel; Rocksoil S.p.A. is involved in the detailed design of the tunnel and in the geotechnical analyses of the TBM.

3. Geological and geomechanical context

3.1 Geological conditions along the tunnel alignment

Moving from the south portal on the Florence side, the tunnel first passes through a gentle slope characterised by clayey-silty deposits with a thickness of greater than 10 m. on average. It then enters a substrate consisting of the *Argille a Palombini* (APA) formation until overburdens of 30-35 m. are reached. These are intensely deformed clays and argillites in thin strata with calcareous (Calcareous/Argillite ratio <<1) or sandstone inclusions. The passage into the *Arenarie dello Scabiazza* formation (SCA) occurs through a tectonic contact. This formation consists of sandstones with intervening siltite and argillite strata. A series of tectonic contacts results firstly in the passage into the *Brecce Argillose Poligeniche* formation (BAP), consisting of clays with clasts of varying lithological nature and then back into the *Arenarie dello Scabiazza* and the *Argille a Palombini* formations. Overburdens in this second section of the tunnel where the APA is present reach 120 m. with an abundant presence of ophiolites ranging in size from a few metres to hundreds of metres consisting of basalt breccias, gabbros and serpentinites. Deposits of the Sparvo landslip are also present on the surface in this zone down to a depth of 30-50 m. Finally a fault contact again leads to the *Monte Venere* formation (MOV), consisting of alternating sandstones with intervening strata consisting of argillites, siltites and clayey marls up to within 25 m. from the north portal on the Bologna side, where silty-sandy detrital deposits are present along a medium steep slope. The reconstruction of the stratigraphy (Fig. 2) was performed on the basis of data acquired from various survey campaigns performed since 1985, with a total of 70 boreholes and seismic surveys.

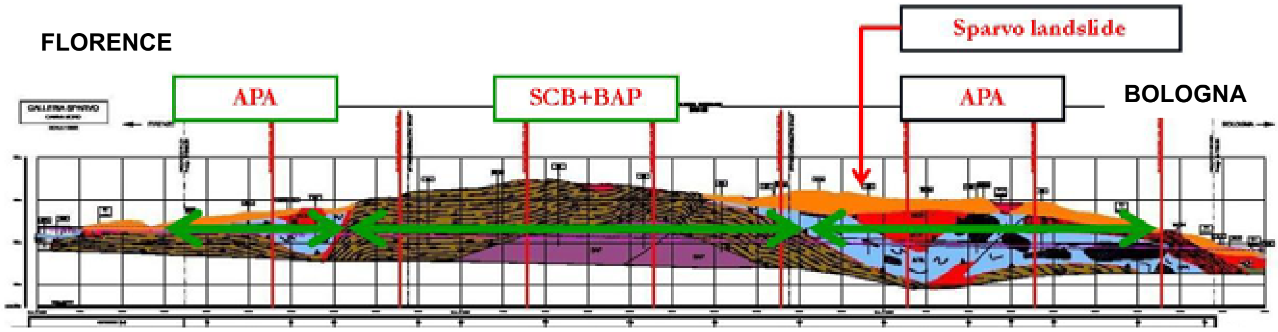

Fig. 2 *Sparvo Tunnel geological longitudinal section*

From a hydro-geological viewpoint, the rock masses present in the area are characterised by secondary permeability, due to fissuring, while the recent deposits and the landslip layers present primary permeability due to porosity, even if the permeability is low. The rocks belonging to the sandy flysch formations (SCA, MOV) have permeability ranging from 10^{-6} to 10^{-8} m/s. Permeability only reaches higher levels in the tectonised bands and underground water flows are present (boreholes found flows of up to 200 L./min. in these zones). On the other hand those formations with a prevalent clayey component (APA, BAP) have very low permeability, with little water flow. No continuous water table was found along the route of the tunnel, even if some samples from the APA had a high water content.

3.2 Geotechnical characteristics

The main lithotypes along the alignment consisted of the SCA (32% of the Length of the tunnel) and the APA (46%), which constituted the two main contexts studied to decide the method of excavation. The *in situ* and laboratory tests performed allowed the geotechnical parameters of strength and deformability given in Table 2 to be established for each lithotype. For the APA, which is without doubt the most delicate geotechnical context, experience acquired in similar contexts was also taken into account such as the *Argille Scagliose* and the *Complesso Caotico* found along the route of the Bologna-Florence high capacity railway line [1] and in other tunnels on the

Bologna-Florence motorway route [2]. Geotechnical parameters that vary with depth and with the degrees of deformation reached during excavation are to be expected for the rock masses belonging to the APA, where the constitutive law is decidedly non linear and of an elastic-plastic nature with "strain softening" behaviour. The equations (1), (2) and (3) give the values for the strength parameters, angle of friction ϕ, cohesion c, and deformability E, as a function of depth (peak and residual):

$$\phi_{peak} = 15+0.07 \cdot z \ [°] \qquad \phi_{res} = 10+0.05 \cdot z \ [°] \qquad (1)$$

$$c_{peak} = 5+1.95 \cdot z \ [kPa] \qquad c_{res} = 1.5 \cdot z \ [kPa] \qquad (2)$$

$$E_{peak} = 13 + 6 \cdot z \ [MPa] \qquad E_{res} = 4 \cdot z \ [MPa] \qquad (3)$$

As a precautionary measure, account was taken when deciding TBM dimensions of the possibility of local zones of rock masses existing with geotechnical parameters poorer by 20% (APA-2). The decay of the strength parameters toward residual values occurs once the failure condition according to the Mohr-Coulomb criterion has been reached, due to plastic deformation in the 0.1-0.6% range. This means that appropriate confinement of the rock mass must be performed during excavation, in order to prevent deformation from occurring which might exert pressures associated with residual strength conditions.

FORMATION	Overburden [m]	GSI	γ [kN/m^3]	k0 [-]	ϕ_p [°]	ϕ_r [°]	c_p [kPa]	c_r [kPa]	E_p [MPa]	E_r [MPa]	v [-]
Deposits	20	-	21	0.53	28	-	20	-	60	-	0.35
MOV	110	30	25	0.59	24	20	150	120	800	640	0.25
BAP	100-130	60	25	0.61	23	18	200	160	4000	3200	0.25
SCB	60-120	40	25	0.50	30	24	250	200	2000	1600	0.25
APA	25	-	22	0.70	16	11	53	37	163	100	0.35
APA-2	25	-	22	0.70	13	10	42	29	130	80	0.35
APA	60-115	-	22	0.70	23	14	230	170	703	460	0.35
APA-2	60-115	-	22	0.70	18	11	180	140	563	368	0.35

***Table 2** Geotechnical parameters of strength and deformability*

Lastly, the main properties of the APA are given: granulometry analyses found the presence of silts and clays in proportions of 40% and 29% respectively (S+C=69%), with percentages for gravels of 12% and sands of 19%. The liquid limit (LL) values were very scattered ranging from 25% to 65%, while values for the plastic limit (PL) were contained within 15% to 25%, with a plastic index of between 10% and 40%. The material is classified as "inorganic clay with medium to high plasticity". The relative consistency index RC, is much higher than one and therefore the rock mass can be defined as "a consistent clay". From a mineralogical viewpoint, 80% of the rock mass consists of clayey minerals and 20%-35% of smectites. Finally the water content is approximately 10%, so the rock mass can be considered "non saturated", even if some samples were found to contain higher levels close to saturation.

4. Design issues

The design of the tunnel excavation operations was conducted in consideration on the one hand of the difficulties of the geotechnical context and on the other of the need to ensure good advance rates in order to reduce construction times. With regard to the geotechnical context, one initial delicate aspect was the alternation along the tunnel alignment of formations which are clearly rocky, such as the sandstones of the SCA and the MOV, with the slope deposits and clayey masses which are poor in quality. This situation requires "flexible" excavation methods both in terms of face excavation and ground improvement and lining operations to be performed at the face and around the cavity. Another critical aspect is the substantial decay of the strength parameters of the

APA, described in section 3.2, as a function of the degree of deformation of the rock mass following excavation (constitutive law with a "strain-softening" branch), a condition which makes it necessary to minimise disturbance during tunnel advance by employing pre-confinement operations in the core ahead of the face. Failure to take that action will result in the progressive closure of the cavity with squeezing phenomena. A third aspect is associated with the presence of dormant, and locally active, landslip deposits near the portals with shallow overburdens and near the village of Sparvo. This condition also requires the use of conservative excavation techniques. Finally the presence of gas deposits within the ground tunnelled is also a very critical factor, especially at the contacts between the various formations. This fact, found in the literature and when boreholes were drilled, means that site organisation, including equipment and operating procedures, must be used which is designed to prevent gas explosions and to guarantee worker safety. The first stage of the design considered both conventional excavation methods, conducted according to the ADECO.RS Approach [3] and TBM excavation and the technical feasibility with the relative advantages and disadvantages was examined for both of them. The final decision to use an EPB TBM was taken jointly by the Client and the Contractor, with account taken of investment strategies linked to future reuse of the machine and, in the belief, as mentioned in the introduction, that this method, in the context of a risk analysis, will provide greater guarantees for completion of the project on schedule. The design of the TBM also constituted a substantial stage in the design. It was conducted by HERRENKNECHT AG and focused on the exceptional diameter of the excavations. Compared to normal excavation diameters of 6-7 m. for underground metro tunnels and a maximum of 10-12 m. for mainline rail and highway tunnels, the Sparvo Tunnel, with a diameter of 15.62 m, is the largest EPB TBM ever constructed. The dimensions of the mechanical parts, especially the cutting-wheel and the shield, therefore needed to be checked and above all the forces required for the thrust cylinders and the cutting torque were defined as a function of the pressure to be maintained in the excavation chamber and the pressure to be exerted on the rock mass around the shield. The whole purpose of this is to prevent the TBM from being blocked when it advances. Details of these aspects are given in the next section. Safety management in the presence of gas also required the introduction of special equipment and work procedures into the design of the TBM which are described in section 6.

5. The geotechnical dimensions of the EPB TBM

As with conventional excavation, it is also essential with a TBM system to analyse the deformation response of the rock mass to excavation when defining the interactive forces between the rock mass and the TBM during tunnel advance. It must be done in terms of "advance core" extrusion and "pre-convergence" at the face and also convergence of the cavity, in accordance with the precise procedures of the ADECO-RS approach [3].

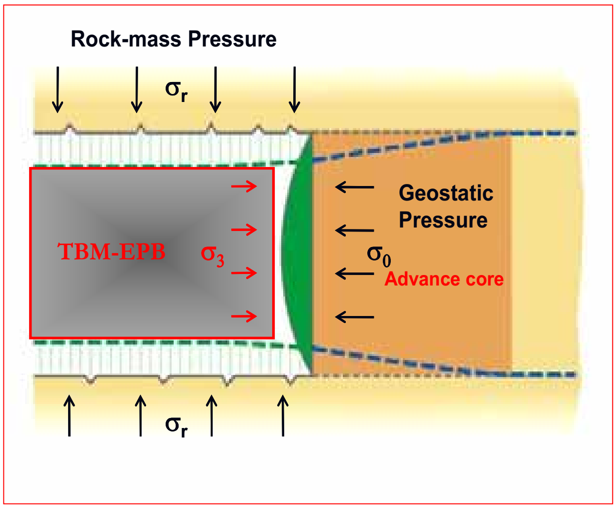

***Fig. 3** – Analysis of the deformation response*

It is important to understand that a close connection exists between "advance core" extrusion and pre-convergence phenomena and convergence of the cavity, and that the control of extrusion, by means of appropriate operations for pre-confinement of the face, allows consequent control of convergence and the pressures associated with it around the cavity [4]. With reference to Fig. 3, once the geostatic pressure σ_0 acting externally on the cutterhead is known, it is essential to accurately define the confinement pressure σ_3 to exert on the excavation chamber, because deformation at the face depends on it and this will condition the stress-strain behaviour of the cavity. This is especially important in grounds such as that of the APA, where the strength characteristics are heavily dependent on the deformation achieved, so that the degree of relaxation of the face basically conditions the size of the plasticised band of the rock mass around the profile of the cavity and the magnitude of the loads acting on the shield and on the final linings. These aspects are addressed in detail in the following sections.

5.1 Core-face behaviour and excavation-chamber pressure

Calculation of the pressure σ_3 to be maintained in the excavation chamber was performed with the objective of maintaining the rock mass at the face in the elastic field and therefore with little deformation. In consideration of the large overburdens of up to 100-120 m. it is in fact impossible to counterbalance the geostatic pressures exerted (σ_v=20-25 bar = 2.0-2.5 MPa, σ_h = 14-18 bar = 1.4-1.8 MPa, assuming $K_o=\sigma_h/\sigma_v=0.7$), as is usually done for TBM excavation in urban contexts, in the presence of shallow overburdens, where the objective is to reduce ground loss to zero and consequently also subsidence at ground level. In this case the generation of plasticisation in the advance-core must be minimised, in order to limit relaxation of the rock mass.
The results of triaxial extrusion tests were particularly useful for calculating the pressure σ_3 according to the criterion just mentioned. These tests simulate tunnel advance on a small scale, by reconstructing the situation that is created in the ground for a cross section ahead of the face when the confinement pressure σ_3 decreases until it is zero. With reference to Fig. 4, a sample of ground (Ø diameter) is inserted into a triaxial cell and the original stress state σ_o of the rock mass is recreated. The pressure of a fluid is then used to also reproduce the stress state ($P_i=\sigma_3$) inside a special cylindrical volume, termed an "extrusion chamber"; the chamber is coaxial to the sample and simulates the face zone of the tunnel. By maintaining the stress state around the sample constant and gradually reducing the pressure P_i of the fluid inside the extrusion chamber, a realistic simulation is obtained of the extrusion at the face ε_x as a function of the confinement pressure P_i. The "extrusion-confinement pressure" graphs are given in Fig. 4 too.

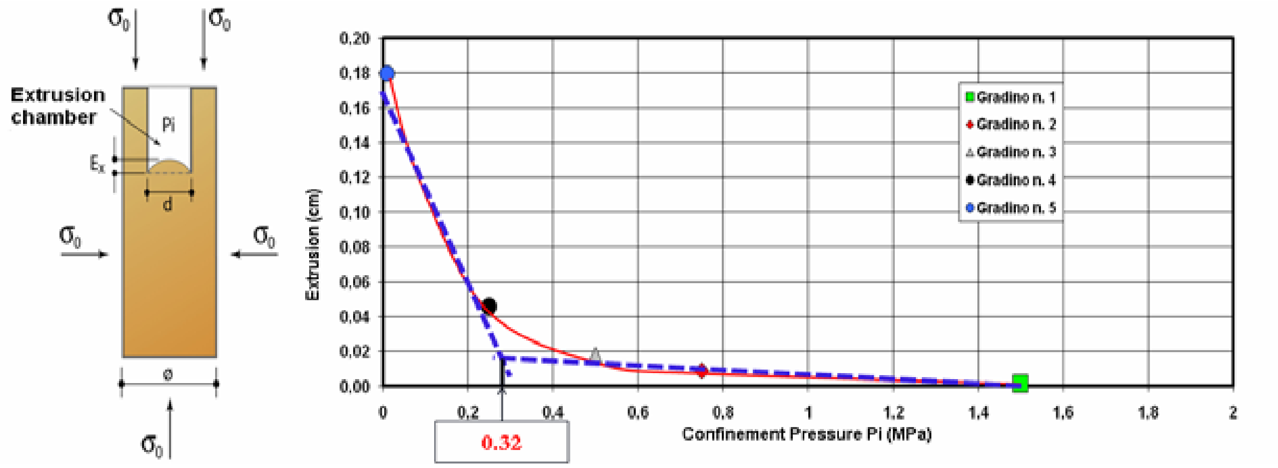

***Fig. 4** – Triaxial extrusion test, Extrusion-Confinement Pressure Curve*

They show the confinement pressure at the limit of the elastic domain (linear section of the curve) which is immediately used, in the design stage, to calculate the pre-confinement pressure required to control deformation phenomena [4]. In the case in question, the tests performed gave confinement pressure values in the range of 2.5-4.0 bar (0.25-0.40 MPa), as a function of the initial stress state simulated (2.0 MPa) and of the geotechnical characteristics of the APA samples employed. Similar confinement pressures were calculated using axially symmetric numerical models. The numerical assessments performed for the SCA formation showed that specific confinement pressures in the excavation chamber were not needed to control face behaviour.

5.2 Frictional forces on the skin of the shield

Calculation of the forces acting on the shield was conducted by analysing the relationship between pressure and convergence of the cavity following the "convergence-confinement" method. Here too, it was unthinkable to work with reference to geostatic pressures, which reach close to 3 MPa with the greatest overburdens. The inevitable convergence of the cavity in the gap set between the diameter of the excavation and the extrados of the shield had necessarily to be considered. The characteristic pressure-convergence curves were obtained by means of 2D numerical analysis which took account of resting coefficients in the 0.6-0.8 range and, in the specific case of the APA formation also of the strain-softening law described in section three. It was therefore possible to calculate the pressures acting for convergence of the cavity of 5 cm, normally manageable by the geometry of the TBM or of 10 cm in the case of overbreak excavation under squeezing rock mass conditions. For example Fig. 5 shows the curves constructed for the APA formation under overburdens of 110 m. employing strength and deformability parameters derived from (1), (2) and (3), the red lines, or considering a decay of 20% in those parameters, the blue lines (APA-2). With convergence of 5 cm. maximum pressures reach 1,250 kPa for the APA-2 and 900 kPa for the APA formation and these pressures fall to approximately 1,000 kPa and 700 kPa for convergence of 10 cm.

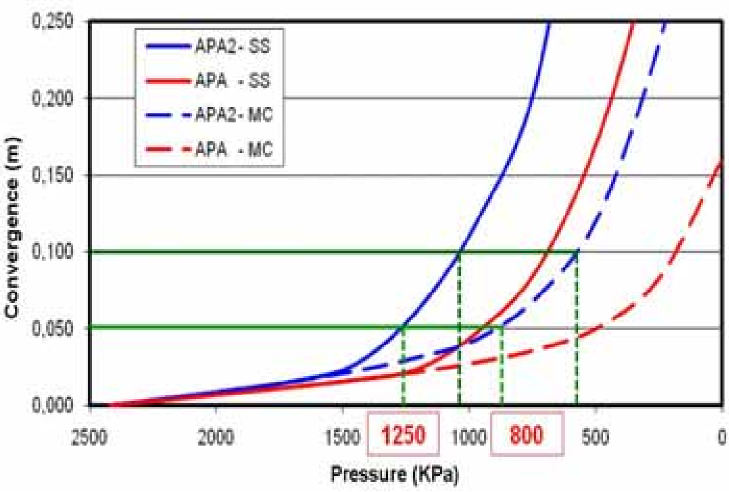

Fig. 5 *- APA (H=110m) - Pressure-convergence curves*

Again in Fig. 5, pressure-convergence curves are given for the use, in the APA and APA-2 formations, of the Mohr-Coulomb elastic-plastic constitutive law, i.e. without inserting the strain-softening branches. The reduction of the consequent pressures, with the same 5-10 cm convergence is important here, showing just how fundamental it is to keep deformation levels low during tunnel advance in order to prevent the material from loosening. The confinement action performed by the pressure in the excavation chamber is crucial in this respect. The pressures of 550 kPa were calculated for the APA formation under low overburdens of 25 m, basically the same as the geostatic loads, which confirms the formation solid loads which propagate up to the surface (gravity action). Finally the pressures calculated for the SCA formation were around 160-200 kPa for convergence of 5 cm and virtually nil for convergence of 10 cm. The results of special 3D analyses confirm the range of the defined pressure acting on the shield.

5.3 Force required of the thrust cylinders (total thrust)

The force associated with the average pressure to be maintained at the face and the force required to overcome the friction between the shield and the rock mass, as a function of the pressures acting on the TBM were certainly the main factors considered in calculating the thrust of the TBM. A series of other factors were also considered, linked to the following: the weight of the machine itself and of the back-up, the forces required to crush the rock mass at the face through the cutters and around the cutterhead, the friction of the system of brushes on the tail of the shield. With reference to what is indicated in [5], the dimensioning of the thrust cylinders (T) is based on the sum of the individual strengths, including an extra safety factor (γ_f). This gives:

$$T = \gamma_f \ \Sigma W \ \text{(kN)} \qquad \Sigma W = W_{sk} + W_{supp} + W_{exc} + W_{sh} \ \text{(kN)} \tag{4}$$

where:

$$W_{sk} = \mu \ [2\pi \cdot R \cdot L \cdot (\sigma_m) + F_w] \ \text{(kN)} \tag{5}$$

is the force due to friction, where R=7.8m and L=12.5m are the radius and the length of the shield respectively and σ_m is the pressure of the rock mass on the extrados of the shield, calculated as discussed in section 5.2. F_w is the weight of the TBM (30000 kN) and of the back-up and μ is the coefficient of friction, for which a value in the range 0.25-0.35 is assumed, based on the lithologies. The use of special bentonite lubricants, injected from the shield mean that a value of 0.15 can be assumed for the coefficient μ;

$$W_{supp} = A_{exc} \cdot P_f \ \ (kN) \tag{6}$$

is the force due to the pressure to be applied in the excavation chamber at the face, where A_{exc} is the area of the excavation and P_f is the confinement pressure σ_3 calculated in section 5.1;

$$W_{exc} = N_{cutter} \cdot P_{cutter} \tag{7}$$

is the force required for excavation, where N_{cutter} is the number of cutters on the cutter head, amounting to 100 and P_{cutter} is the force developed by each cutter, assumed on the basis of experience to be 200 kN per cutter;

$$W_{sh} = 2\pi \cdot R \cdot (p_{sh} + p_b) \tag{8}$$

is the drag force of the tailskin seal where p_b and the cutting edge force, p_{sh}, are assumed to be 10 kN/m on the basis of experience.

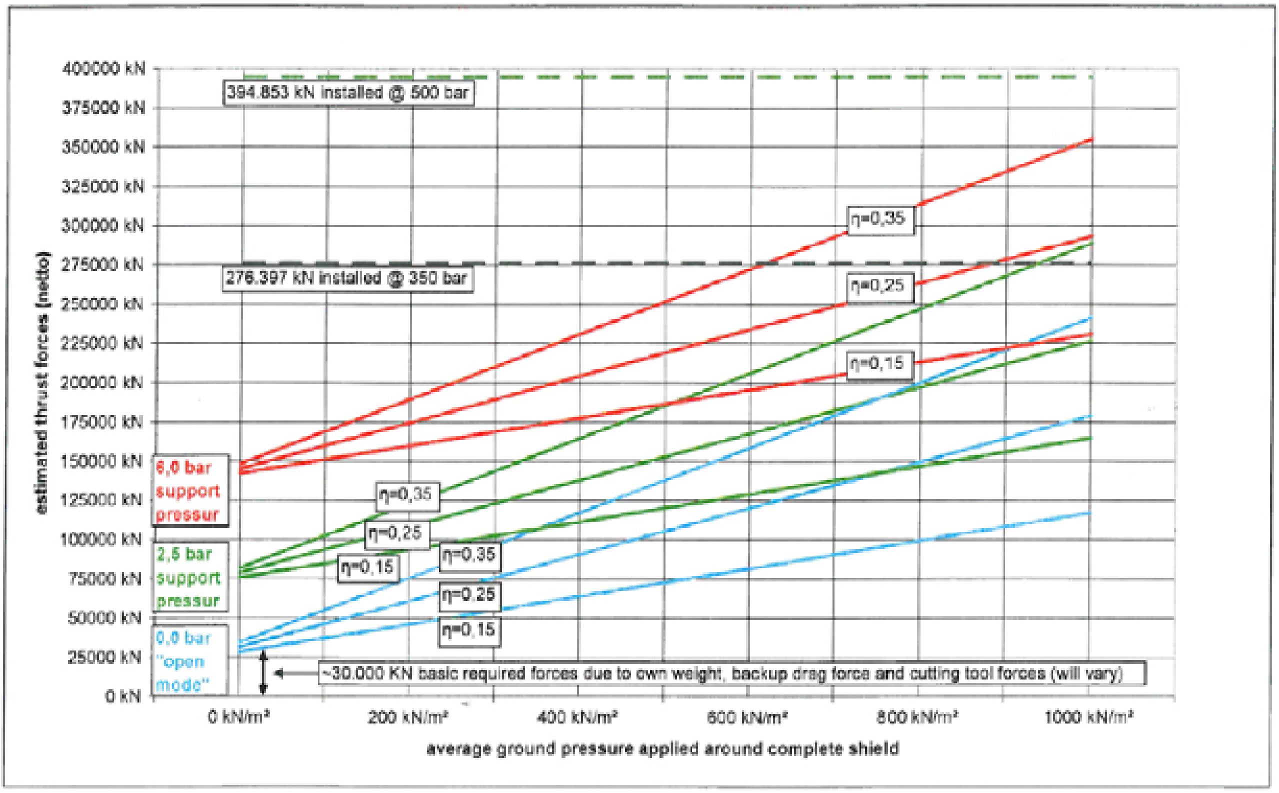

Fig. 6 *– Study of the total thrust of the TBM (Herrenknecht AG)*

The studies conducted led, in co-operation with HERRENKNECTH AG, to the creation of the graph in Fig. 6 where the value for the thrust force is identified as a function of the geotechnical characteristics of the ground and the choice of the coefficient of friction adopted. A minimum thrust value can be observed of 30,000 kN, relating to the weight of the TBM itself, of the back-up and to the force of the cutters. In this case, typical of a non squeezing rock context, the pressure on the shield and in the excavation chamber is considered to be nil. The existence of a pressure in the excavation chamber of 2.5 bar (0.25 MPa) or 6 bar (0.60 MPa) – in the latter case account is also taken of possible hydrostatic pressures in addition to that calculated in section 5.1 – brings the thrust required up to approximately 75,000 kN or 150,000 kN. The thrust values are then derived as a function of the pressure acting on the shield: with a maximum pressure of 1,000 kN/m², or 1,000 kPa, variable thrust values of up to approximately 360,000 kN are obtained depending on the value assumed for the coefficient of friction. The dimensions of the thrust system were designed by employing 57 jacks, which under normal operating conditions exert pressures of 350 bar, to give a total thrust force of approximately 276,000 kN, while under the most severe squeezing conditions greater pressures of up to 500 bar are employed to reach a total thrust of approximately 400,000 kN (safety factor equal to 1.1-1.2).

5.4 Construction details

Detailed design of the TBM highlighted certain construction aspects that are particularly useful in managing interaction between the TBM and the rock mass. Firstly, given the development of convergence in the cavity as a function of distance from the face, it was decided to give the shield a truncated cone shape, with a smaller diameter towards the tail. In detail, the shield is composed of three parts: a front section with a diameter of 15,550 mm., a mid section of 15,510 mm. and a tail section with a diameter of 15,470 mm, with a reduction in the radius with respect to the precious section of 20 mm and a total reduction of 40 mm. between the head and the tail of the shield. A further reduction was also adopted between the cutterhead and the front of the shield of 32.5 mm, therefore bringing the gap on the radius between the profile of the excavation and the extrados of the shield to 72.5 mm. Furthermore, as described in section 5.2, the presence of an extractable cutter in the cutteredge allows an overbreak of a further 50 mm. on the radius, therefore making it easier to manage TBM advance under squeezing conditions. To assist with this the shield was fitted with pressure cells, in order to be able to monitor the pressure exerted by the rock mass on the TBM during tunnel advance. Lastly, again in the shield, special lubricant injectors were fitted to reduce friction on the shield and to reduce the thrust force if the pressures calculated in the design of the dimensions are exceeded.

6. The presence of gas

When conventional excavation methods are employed, the presence of gas in the ground is managed by using anti-explosion equipment and regimes, i.e. using work systems designed to prevent all possibility of explosions. The content of gas in the air is monitored systematically and it is diluted by using appropriate ventilation systems. The issue is more delicate with TBM excavation because it is not possible to render the head of the TBM and the section covered by the shield explosion proof. The approach taken is therefore that of confining the gaseous mixture. TBM advance will be always with a closed shield and the excavation chamber completely full, in order to prevent the possible formation of a "combustion chamber".
This applies also for the SCA and MOV formations where the geomechanical conditions would allow open-type TBM methods of advance. Furthermore, HERRENKNECHT AG also fitted a special gas proof duct to enclose the top of the screw conveyor and the conveyor belt for approximately the first 80 m. in order to prevent any gas present in the muck from spreading into the tunnel, where it might come into contact with components difficult to proof against explosion such as the thrust motors, the head bearings, etc. The duct is kept under pressure in order to prevent gas from escaping. After the first 80 m. of the tunnel anti-explosion conditions are employed, as for conventional tunnelling. A ventilation system and carefully studied safety devices and emergency exit routes complete the measures designed to mitigate the risk from firedamp.

7. Conclusions

This paper describes the analyses conducted to establish the geotechnical dimensions of the EPB TBM that will be employed to drive the "Sparvo tunnel" as part of the works to modernise the A1 Milan-Naples highway in the section between La Quercia (Bologna) and Barberino di Mugello (Florence). The TBM, with the exceptional diameter of 15.62 m, must excavate 2,600 m. of tunnel through a variety of geological conditions ranging from the rocky sandstones of the *Arenarie dello Scabiazza* formation to the geotechnically poor quality clays of the *Argille a Palombini* formation. The latter in particular require excavation to be performed by minimising the deformation response of the rock mass during tunnel advance, both in terms of extrusion and pre-convergence at the face and in terms of convergence of the cavity, in order to prevent the strength parameters from decaying, driven by a strain-softening constitutive law as a function of plastic deformation, and to reduce squeezing pressures acting on the TBM. The magnitude of the confinement pressure in the excavation chamber (in the 2.5-4.0 bar range) was therefore designed to maintain the rock mass in the elastic range, by using, amongst other things, the results of triaxial cell extrusion tests.

Fig. 6 *- Sparvo EPB-TBM (named "Martina") and the portal on the Florence side*

The development of the stress-strain state around the shield was then examined in order to determine the pressures acting on it, which vary as a function of the overburdens and the ground tunnelled in the 250-1,000 kPa range. Calculation of the pressure to be maintained at the face and of the consequent pressures around it, with coefficient of friction values in the 0.25-0.35 range, made it possible to then calculate the thrust to be generated by the jacks for the advance of the TBM. Two working thresholds were identified: the first to generate 275,000 kN of thrust using 57 jacks working at 350 bar each; the second able to reach approximately 400,000 kN under squeezing conditions, bringing the working pressure of the jacks up to 500 bar and using a risk analysis approach. Construction details were designed to prevent the TBM from becoming blocked. These included a gap between the cutterhead and the tail of the shield of 72.5 mm in the radius (shield with a truncated cone shape) and the possibility of a further 50 mm of overbreak on the radius to manage the greatest convergence of the cavity. The shield was also fitted with lubricant injectors to reduce friction between the TBM and the rock mass (μ=0.15). Lastly, special measures for safety management in the tunnel in the presence of firedamp are described.
Construction of the TBM was completed in December 2010 (Fig. 6) and today, in February 2011, it is being transported to the portal on the Florence side, where it will be assembled and commence excavation of the tunnel next June to face a challenge which we hope it will be able to meet.

8. References

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