Problems with TBM & linings

In squeezing ground

To avoid blockages, the TBM must be designed so that its thrust forces can overcome the pressures developed on it by the ground, while time dependent and squeezing phenomena must be considered when the tunnel lining is designed, say Dr Giovanni Lombardi and Andrea Panciera of Lombardi Engineering, Switzerland.

In its April '96 issue, T&T International published a site report on the Transvase Guadiaro-Majaceite 12.2km long, 4.9m diameter water tunnel project in southern Spain. Our article describes the studies undertaken by Lombardi Engineering to develop specifications for the TBM as well as for determination of the short and long term behaviour of the rock mass. The technical data reported are based on information provided by Dragados, a member of the contractor's JV with Fomento Construcciones y Contratas (FCC).

Excavation is based on the use of a double shielded TBM and the installation of a strong lining composed of precast concrete segments. This method is particularly well suited to the excavation of tunnels in heterogeneous poor ground. High pressures tend to develop on the shield which can lead to entrapment of the machine, which in turn leads to expensive downtime. The TBM must therefore be designed to be able to overcome the induced frictional pressures by providing sufficiently high thrust forces.

The most critical sections along the tunnel alignment will be encountered in the Eocene Clay which is expected from much of the length between km7+000 and the northern portal, where there is a relatively high overburden. More than 3km of the tunnel is in this ground. Overburden exceeds 150m (Fig 1). This clay exhibits very low strength and a strong swelling potential due to the high amount of montmorillonite it contains (approximately 30 per cent).

Some assumptions on the rock parameters, as shown below, were based on geological and geotechnical site investigations. They form the basis of the analysis necessary to determine the loads acting in the various situations under review.

The wet, clayey materials develop viscous or creep effects with time and tend to swell. Swelling requires time, the presence of water and reduced confining stresses. To take into consideration the higher strengths in short-term behaviour, the angle of friction and the cohesion have been increased by one degree and 0.06Mpa respectively in terms of the initial project parameters, which can be considered to represent the long term conditions. As a rule, these values were derived from laboratory tests and analyses of similar cases and thus correspond mainly to the characteristics of long term behaviour.

Maximum pressure for the swelling function was assumed to fit the maximum pressures due to the 400m overburden. Two different ‘swelling confining pressure’ relationships were chosen as shown in Fig 2, which refers to volumetric swelling. The assumptions were verified by odometer tests and will be adopted for the present situation. An axial symmetrical model will be used in the computations.

Computational method

The pressures and forces acting along the length of the shield during excavation and after prolonged machine stops, e.g. for maintenance, were computed using a program developed by Lombardi of Switzerland based on a pseudo three-dimensional model and assumes that the rock mass is symmetrically stressed in terms of the tunnel axis. In this model, the frictional forces between contiguous discs, cut perpendicularly to the tunnel axis, are considered.

During iterative calculations, the structural effect produced by the distance of the excavated face (state of tension changing from a 3-D to a 2-D pattern) is taken into account, as well as the rheological effects of the weakening of the rock, from the short to the long periods characteristics. Finally, the influence of the rate of advance on the strain-stress state is also introduced as a function of the time dependent properties of the ground. Supporting pressures can be taken into account behind the face along the length of the excavation (e.g. systematic anchoring, deformable segments with elastoplastic characteristics or supports with predefined rigidity such as steel or precast concrete linings).

It is thus possible to simulate placement of any kind of support during and after excavation. This is particularly important when a TBM is being used, given the possibility of stoppages and their effect on progress. Pressure distribution on the precast concrete seg-

Fig 1. Longitudinal geological cross section of the Transvase Guadiaro-Majaceite tunnel, Spain
ments has been determined using Lombardi Engineering software based on the characteristic lines method. Characteristic lines correspond to stress-deformation relationships at the excavation limit as a function of the geomechanical parameters of the rock mass. The analysis is based on an axis-symmetrical and bi-dimensional model.

Pressure distributions

The materials are assumed to have homogeneous and isotropic properties in accordance with the Mohr-Coulomb model. Three-dimensional considerations are required to evaluate rock mass behaviour near the excavation face. The model has been used to determine short and long term pressure distributions on the concrete lining segments in consideration of the ground behaviour near the TBM and its development to the final state. The time dependent as well as the swelling properties of the rock mass result in increase of pressure on the precast concrete segments.

Case analysis shows that the following conditions are the most critical and that they determine the machine thrust required and the force on the shield. The machine provided for the excavation of the 12.2km tunnel is a double shielded, 11m long TBM. It was designed to accommodate a radial gap of 50mm (overboring) between the excavated diameter at the face and the outer surface of the shield (Fig 3).

1. For an estimated advance of 10m/day, deformation develops from the face to the point where the converging rock touches the shield. This happens when a radial deformation of 50mm has occurred. From this time the rigidity of the steel structure resists the inward movement of the ground. The pressures develop to a maximum of 1MPa (100 tonne/m²) at the rear edge of the 11m long shield (Fig 3 b).

2. The integration of these pressures along the length and circumference of the shield leads to a total compressive force of about 115 000 kN (11 500 tonne). This force, together with the weight of the machine, assuming that the friction factor which will apply along the shield surface (taken by the manufacturers to be 0.25) leads to a thrust value of the order of 30 000 kN (3000 tonne). The thrust required on the face itself has to be added to this force.

3. During a prolonged downtime period, the changes in the stress-strain field will continue. The contact will now be displaced to a point immediately behind the face (Fig 3c). Also, higher pressures will come into play because the ground near the shield will soon reach the lower boundary of its resistance, which is characteristic of long term behaviour.

Total acting compression in this case will be higher than 130 000 kN, requiring a thrust of 34 000 kN (3400 tonne) to restart the machine. In the worst case scenario - when the face becomes unstable and has to be supported by the cutterhead - the thrust on the face must also be taken into consideration.

Summing up, for an advance rate of 10m/day: total compression on the shield is approx 110 000 kN; machine weight approx 4800 kN; thrust on face approx 3000 kN and total pushing force required approx 33 000 kN. Values when standing would be (approx) 130 000 kN, 4800 kN, 3000 kN and 37 000 kN respectively.

Precast concrete linings

Only undisturbed advance rates are considered here. An analysis of the tunnel excavation and concrete linings within the Eocene Clays is presented in Fig 4 and the table below:

1. Point A represents the balance at the face, assuming a 3000 kN machine thrust during excavation. Behind this point, rock deformation may attain 50mm because of the overburden, leading to a compressive strength on the shield increasing from initial values of about 200-300 kN/m² up to 900 kN/m² (points B and C). The values correspond to analytical results (Fig 4).

2. During placement of the precast linings, the shield occupies the space available between the rock and the outside of the concrete segments. Despite the fact that this space is filled with pea gravel after the shield has moved forward, the poor properties of the excavated clays result in additional ground con-

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vergence. The short term equilibrium on the lining elements leads to deformations of 50mm, corresponding to point D in Fig 4.

In the long term, reduction of geomechanical parameters (creep) and swelling effects leads to an increase of the pressures on the confined annular lining. Points E, E1 and E2 in Fig 4 correspond to the equilibrium situation without swelling and with swelling according to assumption 1 and assumption 2 respectively (Fig 2).

Based on the geological conditions and the construction method adopted, concrete strength values (50-80MPa) are sufficient to ensure the long term stability of the excavation, with a maximum hydrostatic pressure of 1350-1700kN/m². The induced compressive stresses on the concrete lining reach approximately 22MPa due to stress concentrations on the contract's surfaces between the precast elements.

Conclusions
The analysis presented above indicates that a minimum design thrust in the order of 40-50MN will be necessary to restart the machine after a stoppage. The project specification therefore required a nominal thrust of 40MN. Tunnelling has shown that such a force has been enough to overcome the difficulties encountered during excavation and to avoid complete stoppage or entrapment of the TBM. Such a stoppage might well have required construction of a by-pass adit.

In similar tunnel projects where this type of evaluation was not carried out and the maximum forces were based on empirical observations, some stoppages did occur. This resulted in several months of additional work and in extreme cases the TBM had to be abandoned.

When the margin between the design thrust and the value computed by means of the analysis described is small (approximately 1.1), it is worth providing the shielded TBM with a lubrication system using either bentonite or synthetic foam to ease the shield's passage through the squeezing ground.

Tunnel excavation has been extremely successful. The double shielded TBM was stopped on five occasions but was easily released by excavation of a small adit above the shield. At most, ten per cent of the shield surface had to be freed. This means that the margin of 1.1 mentioned above was increased to approximately 1.25 due to the actual surface in contact, so a margin of 1.4 to 1.5 on the thrust value at the design stage is considered more appropriate.