Underground Construction in Soft Ground, Fujita & Kusakabe (eds) © 1995 Balkema, Rotterdam. ISBN 90 5410 536 4

Design of tunnel linings being constructed in soft watered ground

N.N. Fotieva & A.S. Sammal
Tula Polytechnical Institute, Russia

SYNOPSIS: Methods of designing tunnel linings being constructed with the application of grouting (cementation) in soft watered ground are given. The methods allow the lining stressed state appearing due to actions of the ground own weight and the external water pressure including cases of water filtration through the grouted soil zone and lining to be determined.

INTRODUCTION

Design of tunnel linings being constructed with grouting the surrounding soils is based on investigating the interaction of the structure, the strengthened ground zone and the rest of the soil massif as elements of the integrated deformable system. The methods propoused are based upon the analytical solutions of corresponding elastic theory plane contact problems for infinite linearly deformable medium simulating the soil massif with one or several openings supported by rings simulating the linings or double-layers rings whose second layer simulates the strengthened soil zone. Application of a linearly deformable soil mass model for lining design is possible because displacements of lining are small and the curve of non-linear dependence between stresses and deformations of soil on the part from the moment of the lining being brough into contact with soil mass till stabilisation of deformations can be considered as linear.

The soil creep may be taken into account on the base of linear hereditory creep theory with the application of the variable modulii method.

1 DESIGN OF TUNNEL LININGS OF AN ARBITRARY CROSS-SECTION SHAPE

The design method developed is based upon the solution of the plane contact problem for a double-layer non-circular ring in the linearly-deformable medium. The design scheme is given in Figure 1.

The S_1 medium simulating soil massif is characterised by the E_1 deformation modulus, ν_1 Poisson ratio and K_1 filtration coefficient. The S_2 outer ring layer of the Δ_1 thickness the material of which has an E_2 deformation modulus, ν_2 Poisson ratio and K_2 filtration coefficient simulates the soil zone strengthened by grouting. The S_3 inner ring layer of the Δ_2 thickness with E_3, ν_3, K_3 corresponding characteristics simulates the tunnel lining.

Ring layers and medium undergo deformation together so that conditions of continuity of displacements and total stresses are satisfied on the L_i ($i = 1, 2$) boundaries. The L_3 internal boundary is not loaded.

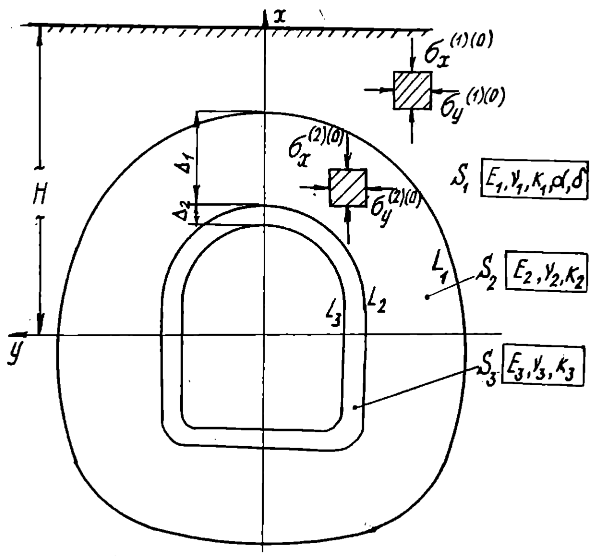

Figure 1. Design scheme

In the S_1 and S_2 fields there are initial stresses coused by the soil own weight or by the underground water pressure. In the last case the initial stresses are equal to the residual water heads and are determined by formulae (Manual in designing ...., 1983):

$$\sigma_x^{(1)(0)} = \sigma_y^{(1)(0)} = -\gamma_w H_e \frac{\frac{1}{K_3} \ln \frac{R_2}{R_3} + \frac{1}{K_2} \ln \frac{R_1}{R_2}}{\frac{1}{K_3} \ln \frac{R_2}{R_3} + \frac{1}{K_2} \ln \frac{R_1}{R_2} + \frac{1}{K_1} \ln \frac{R_1}{R_t}}, \qquad (1)$$

$$\sigma_x^{(2)(0)} = \sigma_y^{(2)(0)} = -\gamma_w H_e \frac{\frac{1}{K_3} \ln \frac{R_2}{R_3}}{\frac{1}{K_3} \ln \frac{R_2}{R_3} + \frac{1}{K_2} \ln \frac{R_1}{R_2} + \frac{1}{K_1} \ln \frac{R_1}{R_t}}, \qquad (2)$$

where γ_w is the water specific weight, H_e is the underground water level, being counted out from the centre of coordinates, R_i ($i = 1, 2, 3$) are average radii of the L_i ($i = 1, 2, 3$) outlines correspondingly, R_t is an agreed radius of feeding (Manual in designing ...,1983)

If the action of the soil own weight is considered then two cases are distinguished: when preliminary soil grouting is made from ground surface or from the opening face and when the grouting is made through the already constructed lining. In the first case the initial stresses are determined by formulae

$$\sigma_x^{(1)(0)} = \sigma_x^{(2)(0)} = -\gamma H \alpha^*, \quad \sigma_y^{(1)(0)} = \sigma_y^{(2)(0)} = -\lambda \gamma H \alpha^*, \tag{3}$$

where γ is the unit weight of ground, H is the tunnel depth, λ is the coefficient of lateral pressure in natural soil massif, α^* is the correcting factor introduced for the lagging of the lining behind the face of the underground opening to be taken into consideration.

In order to determine the α^* value for designing the tunnel linings located in ground not subjected to creep pressure upon lining is considered as reaction of the lining on the advancement of the opening face. The scheme of the opening surface displacements is shown in Figure 2.

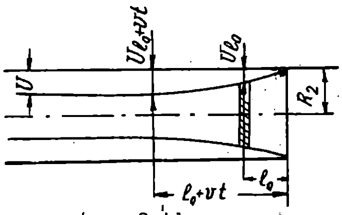

Figure 2. Displacements of the opening surface

If the lining is being installed at the ℓ_0 distance from the face of the opening (Figure 2) a part U_{ℓ_0} of displacements has time to be developed before the lining is put into operation consequently during further advancement of the face as far as certain $\ell = \ell_0 + vt$ distance from lining (v is the mean rate of advance, t is time), the displacements being transferred upon the lining from the soil massif have the form

$$U_{\ell_0+vt} - U_{\ell_0} = U\left(\frac{U_{\ell_0+vt}}{U} - \frac{U_{\ell_0}}{U}\right) = U[f(\ell_0+vt) - f(\ell_0)], \tag{4}$$

where $f(\ell_0+vt) = \frac{U_{\ell_0+vt}}{U}$, $f(\ell_0) = \frac{U_{\ell_0}}{U}$ are relations of displacements in spatial and plane problems, which can be calculated by empherical formula

$$f(\ell) = 1 - e^{-1.3\frac{\ell}{R_2}}. \tag{5}$$

The displacements (4) can be obtained if initial stresses are determined using formulae (3), in which the correcting factor is defined from the following expression

$$\alpha^* = f(\ell_0+vt) - f(\ell_0) = e^{-1.3\frac{\ell_0}{R_2}} - e^{-1.3\frac{\ell_0+vt}{R_2}} = e^{-1.3\frac{\ell_0}{R_2}}\left(1 - e^{-1.3\frac{vt}{R_2}}\right). \tag{6}$$

To calculate stresses in the lining when the opening face advances on a considerable distance from the lining, one may assume $e^{-1.3vt/R_2} \to 0$. In that case formula (6) changes to

$$\alpha^* = e^{-1.3\frac{\ell_0}{R_2}}. \tag{7}$$

In the second case when strengthening by grouting the soil mass being fulfilled through the lining at the $\ell_1 > \ell_0$ distance from the opening face the stresses in the lining are the sum of those caused by the advancement of the face up to the ℓ_1 distance (when there is no strengthened zone yet) and those caused by advancement of the face after strengthening. That is why in case of grouting through the lining at the ℓ_1 distance from the face the calculation is carried out twice. At first stresses and forces in the lining before strengthening are defined from the solution of the problem design scheme of which is shown in Figure 1, when the S_2 layer is absent ($E_2 = E_1$, $\nu_2 = \nu_1$) and correcting factor is

$$\alpha^* = f(\ell_1) - f(\ell_0) = e^{-1.3\ell_0/R_2} - e^{-1.3\ell_1/R_2}. \tag{8}$$

Then additional stresses and forces appearing in the lining after grouting are to be found from the solution of the same problem (Figure 1) with the presence of the S_2 layer having E_2, ν_2 characteristics an at

$$\alpha^* = e^{-1.3\ell_1/R_2}. \tag{9}$$

The obtained results are summed up.

The elasticity theory plane contact problem described above is solved with the application of the complex variables analytic functions theory (Muskhelishvili, 1966), the apparatus of conformal transformations and the complex series. A complete algorithm has been formed and a computer programme has been elaborated.

The design method developed allows the influence of soil creep to be taken into account. Consideration of visco-elastic deformation of soil mass is carried out on the base of linear hereditary creep using method of variable modulii, according to which deformation characteristics of ground in a solution of the elasticity theory problem are presented as functions of time. With this aim the following formulae (Amusin & Linkov, 1974) can be used

$$E_1(t) = \frac{E_1}{1+\varphi(t)}, \quad \nu_1(t) = 0.5 - \frac{0.5-\nu_1}{1+\varphi(t)}, \tag{10}$$

in which $\varphi(t)$ is the creep function determined as

$$\varphi(t) = \frac{\delta t^{1-\alpha}}{1-\alpha} \tag{11}$$

where δ , α are creep parametres, t is time counted out from the moment of putting the lining into operation.

The method of determining the linear hereditary creep parametres from pressuremeter experiments has been described in paper by V.N.Denisov & N.S. Chetyrkin (1982) according to which the function $E_1(t)$ is determined as

$$E_1(t) = \frac{E_1}{1+\theta E_1(1-e^{-\lambda^* t})}. \tag{12}$$

The $E_{1\infty}$ long-term deformation modulus when $t \to 0$ is

$$\frac{1}{E_{1\infty}} = \frac{1}{E_1} + \theta \tag{13}$$

The θ and λ^* creep parametres can be determined from pressuremeter experiments.

For designing tunnel linings with registration of the soil creep it is necessary to take into consideration that a U_{ℓ_0} part of displacements depend not only on the ℓ_0 distance, but also on t_0 time interval between drifting the part of the underground opening and installing the lining. The technique of lining design with consideration of the soil creep is described in the paper by N.N.Fotieva, A.S.Sammal, N.S.Chetyrkin (1988).

2 DESIGN OF THE LININGS OF MUTUALLY INFLUENCING PARALLEL CIRCULAR TUNNELS

For designing linings of circular parallel tunnels the solution of contact problem for the medium, weakened by a finite number of openings, supported by multilayer rings, simulating the linings are applied. These may be for example linings from concrete with a steel envelope, the fitting layers in ferroconcrete linings, the tubbings back edges and ribs may be considered as separate layers. The external layer may simulate the strengthened soil zone.

The analytic solution of contact problem is described in the book by N.N.Fotieva and A.N.Kozlov (1992). The soil creep and also the sequence of the openings being driven and the linings layers being constructed may be taken into account in designing. With this aim a series of problems at the corresponding design schemes (an additional layers or an additional openings are added) with different α^* factors at each time moment are considered (Fotieva & Kozlov, 1992). For example, if the lining has S_i (i = 1,..., n) layers and each of them is being constructed at a corresponding ℓ_{i-1} (i = 1,2, ..., n) distance from the opening face, then stresses in an i -th layer of the lining are estimated by formula

$$\sigma^{(i)} = \sum_{j=i}^{n} \sigma^{(i)(j)} \alpha_j^* \qquad (i = 1,\dots,n) \tag{14}$$

where $\sigma^{(i)(j)}$ are the stresses in the i -th layer of the construction having j layers, and

$$\alpha_j^* = f(\ell_j) - f(\ell_{j-1}). \tag{15}$$

Here

$$f(\ell_j) = 1 - e^{-1.3\ell_j/R_1} (j = 1,\dots,n-1); \; f(\ell_n) = 1 \tag{16}$$

3 DESIGN OF ROUND TUNNELS LININGS IN A HETEROGENEOUS GROUND

The method of designing tunnel linings intersected at a diameter by the border of the grounds with different deformation characteristics (that may be, for example, the interface between soft soil and hard rock) is described below. The method developed is based upon an analytic solution of elasticity theory contact problem for piecewise-homogeneous plane weakened by a round hole symmetrically arranged and supported by ring with R_0 and R_1 external and internal radii correspondingly (Figure 3).

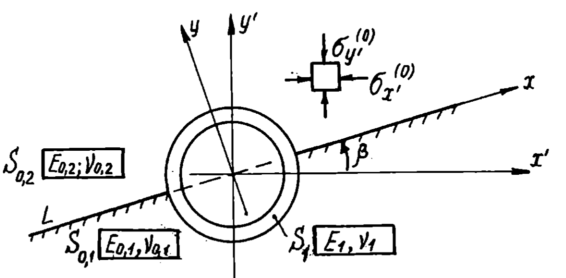

Fig. 3. Design scheme

The S_1 ring whose material has E_1 deformation modulus and ν_1 Poisson ratio simulates the tunnel lining. The plane consists of two linearly deformable $S_{0,1}$ and $S_{0,2}$ medii simulating the grounds with different characteristics, namely the $E_{0,i}$ (i = 1, 2) deformation modulii and the $\nu_{0,i}$ (i = 1, 2) Poisson ratioes correspondingly. Conditions of displacements and total stresses continuity are fulfilled on the L border between the $S_{0,1}$ and $S_{0,2}$ medii and on the $L_0 = L_{01} + L_{02}$ border. The L border may be inclined at a β arbitrary angle to the horizontal; the Ox axis is directed along the L border.

Action of the ground own weight is simulated by initial stresses

$$\sigma_{x'}^{(0)} = -\lambda\gamma H\alpha^*, \quad \sigma_{y'}^{(0)} = -\gamma H\alpha^* \tag{17}$$

For the purpose of simplification initial stresses in S_{01} and S_{02} medii are assumed to be equal.

Plane contact problem described is solved by N.N.Fotieva and O.V.Afanasova (1991).

4 EXAMPLES OF DESIGN

Below there are results of stress analysis for tunnel lining with a 11.2 m unsupported span and a 10.5 m height. The mean opening radius is R_2 = 5.7 m. The design was fulfilled upon the action of soil own weight, input data being the following: $\Delta_1/R_2 = 0.8$, $\Delta_2/R_2 = 0.1$; $E_1 : E_2 : E_3 = 1 : 4.5 : 40$; $\nu_1 = \nu_2 = 0.3$; $\nu_3 = 0.2$; $\lambda = 0.43$; $\ell_0/R_2 = 0.1$. The cases of preliminary strengthening of soil and grouting through the lining at the $\ell_1/R_2 = 2$ distance from the face are considered. The lining was calculated both without and with the soil creep being taken into consideration, in last case $\alpha = 0.7$, $\delta = 0.006\ s^{-0.3}$, t_0 = 0.5 days - time interval between drifting the part of the underground opening and installing the lining; t_1 = 10 days - time interval after putting the lining into operation till the grouting is carried out- were assumed. The forces acting in the structure for the 60-th days were determined. Figure 4a,b shows distribution of the $M/\gamma H R_2^2$ bending moments and the $N/\gamma H R_2$ longitudinal forces in case of preliminary strengthening (Figure 4a) and the grouting through the lining (Figure 4b) for soil not subjected to creep (solid lines) and when the soil is subjected to creep (dash lines)

Let us consider the results of design obtained for two parallel pilot-tunnels of the Moscow Underground. There are data of measurements fulfilled in those tunnels (Vinogradov, 1959) which may be used for comparison with the design results. The tunnels have tubbing linings and are arranged in clays. The distance between tunnels is about 3 m, the depth of tunnels is 40 m.

The lining of the experimental part was installed in pilot-tunnel of the station hall when the first tunnel had already been constructed. The contact pressures were considerably increased when the operations of the tunnel opening on full section came nearer. In general the maximal increase of pressures upon the tubbing ring was 30 %. Maximal pressures on seporate tubbings reached 0.7...0.8 MPa. Average maximal pressures on two mos loaded rings of an experimental part were 0.55 and 0.54 MPa. The average magnitude of pressures was 0.36 MPa. Results of comparison of radial contact stresses calculated with the experimental data are given in Table 1.

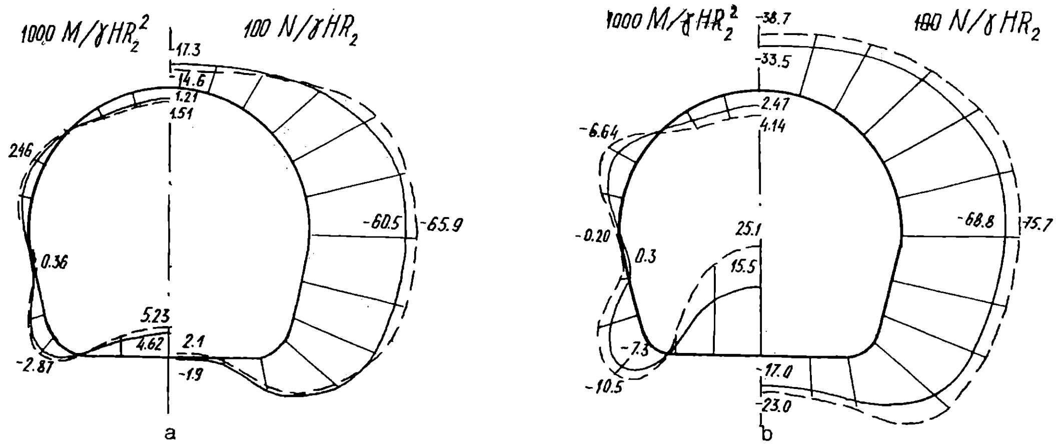

Fig. 4. Distribution of forces in the lining

Table 1. Results of comparison of stresses calculated with the experimental data

Values	Radial contact stresses(MPa) calculated	measured	Differences in %
maximal	0.76	0.7...0.8	9.0
average	0.57	0.54...0.55	5.6
greatest increase of stresses in %	24	30	-

One can see from Table 1 that correspondance is satisfactory.

Further let us consider the results of designing the lining of tunnel intersected by horizontal border of different grounds. Input data are following: R_0 = 5 m, R_1 = 4.6 m, $E_{0,2}$ = 0.5$E_{0,1}$ $\nu_{0,1}$ = $\nu_{0,2}$ = 0.25; λ = 0.33; E_1 = 75 E_0 .

The results are given in Figure 5, which shows diagrams (developed views) of $\sigma_\theta^{in}/\gamma H\alpha^*$ normal tangential stresses upon the internal lining outline obtained by the method descibed (curve 1) and for comparison by numerical-analytic method (curve 2), proposed by V.V.Savitsky (1988).

One can see from Figure 5 that difference of results is insignificant.

In conclusion we can mark that all the methods described above have been programmed for the computers.

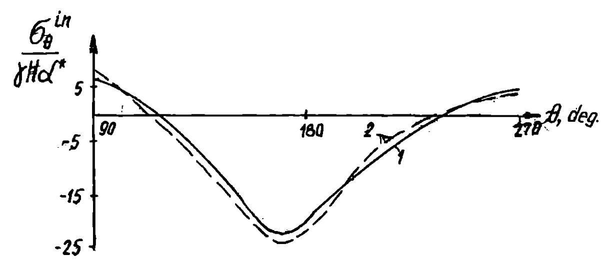

Fig. 5. Distribution of $\sigma_\theta^{in}/\gamma H\alpha^*$ stresses in the lining

REFERENCES

Amusin, B.Z. & Linkov, A.M. (1974).On application of the variable modulus method for solution of one class of linear heridatory creepage problems. Proc. of the USSR Academy of Sciences. Mechanics of a Solid Body. - Vol.6, 162-166.

Fotieva, N.N. & Afanasova, O.V. (1991). Designing the linings of underground structures in a heterogeneous massif upon the action of the rocks own weight action. Podzemnoye i shakhtnoye stroitelstvo. 2, 14-16.

Fotieva, N.N. & Kozlov, A.N. (1992). Design of parallel openings support in seismic regions. Nedra. Moscow.

Fotieva, N.N. & Sammal, A.S. & Chetyrkin, N.S. (1988). Influence of soil creep on stress state of the underground structures being built with strengthening of the surrounding massif. Int. conference on rheology and soil mechanics. Edited by M.I. Keedwell, Coventry Polytechnic, Coventry. UK. Elsevier applied Science London and New York.

Manual in designing underground mining workings and support computation. (1983). VNIMI, VNIIOMShS. Stroyizdat. Moscow.

Muschelishvili, N.I. (1966). Some basic problems of the mathematic elastic theory. Nauka. Moscow.

Savitsky, V.V. (1988). Method of designing stresses in deep round structures in heterogeneous massif. Avtoreferat candidat. dissertacii. Moscow.

Vinogradov, B.N. (1959). Experience of measurements of rock pressure upon the tunnel linings by mezdozes. Proc. by CNIIS. 31. Moscow.