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Interaction of round section assembled lining with soft ground

N.S. Bulychev
Tula State Technical University, Russia

Y.E. Shamrin
Donpromstroyniiproekt, Donetsk, Ukraine

SYNOPSIS: Method of designing pre-fabricated constructions of tunnel linings with articulated joints of blocks is discussed. Advantages of that type of lining for the tunnel in soft ground, its application field and bearing capacity are considered.

1 INTRODUCTION

The problem of designing tunnel lining requires the analysis of their behaviour in ground massif, i.e. consideration of simultaneous deformation of the lining with the ground and interaction thereof. Here we deal with a lining-ground mass system which is subject to external loads and actions (ground own weight, underground water pressure, seismic action action, etc.).

Studying that system is the subject of underground structures mechanics (Bulychev, 1989), the design methods of which are characterized by a fundamental principle, that of contact interaction of the lining and the deformable ground mass under loads and actions.

At present analytic approach to designing the method of elaborating pre-fabricated linning with articulated joints the one corresponding the above principle is too difficult. That is why an approximate design method is proposed in this paper.

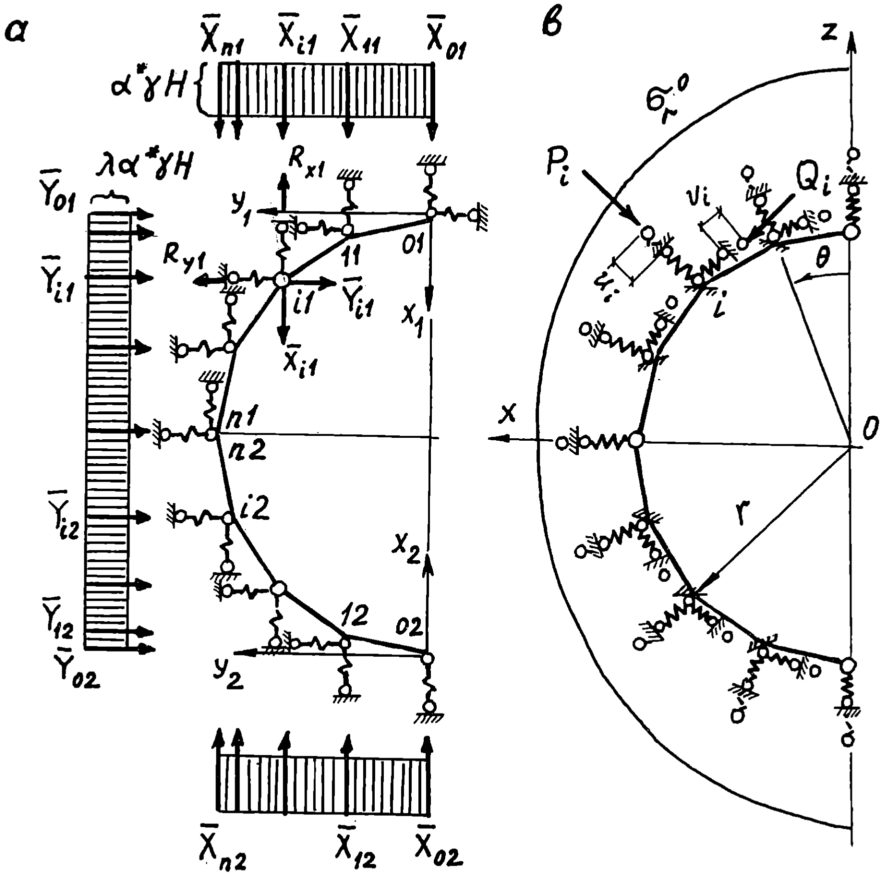

Fig.1 Design scheme of pre-fabricated lining

2 DESIGN METHOD AND ITS RELIABILITY

Design scheme of pre-fabricated lining is shown in Fig.1. The ground massif in that scheme is simulated by a coupled elastic rod system. Coupled rods are in junction centres of a broken line simulating the lining ring. Junction centres may be located on monolith parts of the blocks as well as on the joints of these blocks.

Stiffness factors of the rods are determined according to the stiffness of the ground mass by formula

$$EF = K_{\sigma} \frac{l_i + l_{i+1}}{2} \qquad (1)$$

where is the passive earth pressure coefficient:

$$K_{\sigma} = \frac{E_0}{r(1+\nu_0)} \qquad (2)$$

Here l_i is the spacing between junction centres, E_0 and ν_0 are the deformation modulus and the Poisson Ratio of the ground mass, is the radius of the lining ring.

Loads are connected with initial stresses in the ground massif, namely the γH vertical stress (where γ is the unit of the ground weight and H is the depth) and the $\lambda\gamma H$ horizontal stress (where λ is the lateral pressure coefficient). The design initial stresses are equal correspondingly to $\alpha^*\gamma H$ and $\lambda\alpha^*\gamma H$, where α^* is the correcting coefficient introduced for the distance of lining lagging behind the opening face being taken into account.

Design initial stresses are removed from the opening surface due to ground excavation hence in the design scheme the removed stresses act on the lining-ground system (Bulychev, 1989).

The concentrated forces corresponding the removed stresses may be applied to the junction centres of the lining design scheme in two ways. The first variant is shown in Fig.1, a. The internal forces: the **M** bending moments and the **N** longitudinal forces may be obtained due to computation of the rod system shown.

The radial and shear stresses on the lining-ground contact are determined as sum of the initial and removed contact stresses. It is required necessary mark that the removed contact stresses are determined by the R_x and R_y reaction arising in the coupled bearing rod.

The second variant of loading the design scheme is in preliminary compression of the coupled elastic rods according to the initial ground stresses (Bulychev, Fotieva et al.,1988). The radial and tangential directions of elastic bearing rods are shown in Fig.1,b. In that case the inintial contact stresses distribution is determined from

$$\sigma_r^0 = \alpha^*\gamma H \left(\frac{1+\lambda}{2} + \frac{1-\lambda}{2} \times \cos 2\theta \right) \qquad (3)$$

$$\tau_{r\theta}^0 = \alpha^*\gamma H \frac{1-\lambda}{2} \sin 2\theta \qquad (4)$$

The interaction of the compressed rods (springs) simulating the ground massif and the streak bar simulating the tunnel lining allows the **M** and **N** required internal forces to be obtained.

Reliability of the design method described above is proved by comparison with the accurate solution of the elasticity theory plane contact problem for the ring supporting round opening in linearly deformable medium. Results of comparison are shown in Fig.2 (input data are $\alpha^*\gamma H$ = 0.25MPa, E_1/E_0 = 100, ν_0 = 0.35, λ= =0.54, r = 1.87 m).

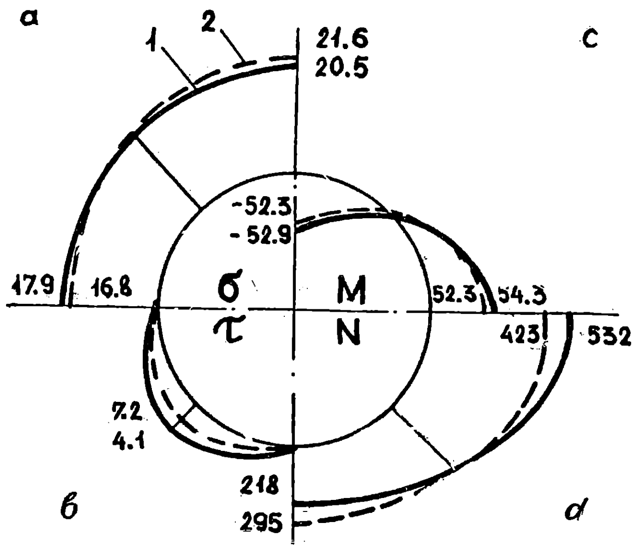

Fig.2 Comparison of design values (2) the σ and τ contact stresses ($1\cdot10^2$MPa), the M(kN·m) and N(kN) internal forces with those obtained from the accurate solution of elasticity theory problem (1).

Comparison of full-scale measurement of the radial contact stress (around pressure) on the subway tunnel lining in St.-Petersburg and the designed one is given in Fig.3. Input data are the following:

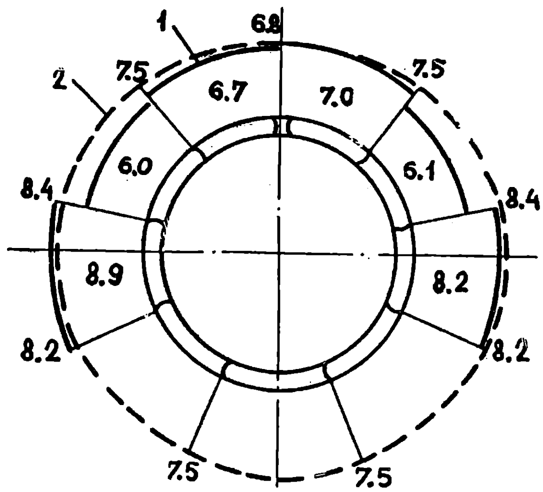

Fig.3 Comparison of the measured (1) and the designed (2) contact stresses ($1\cdot10^2$MPa) on subway tunnel lining

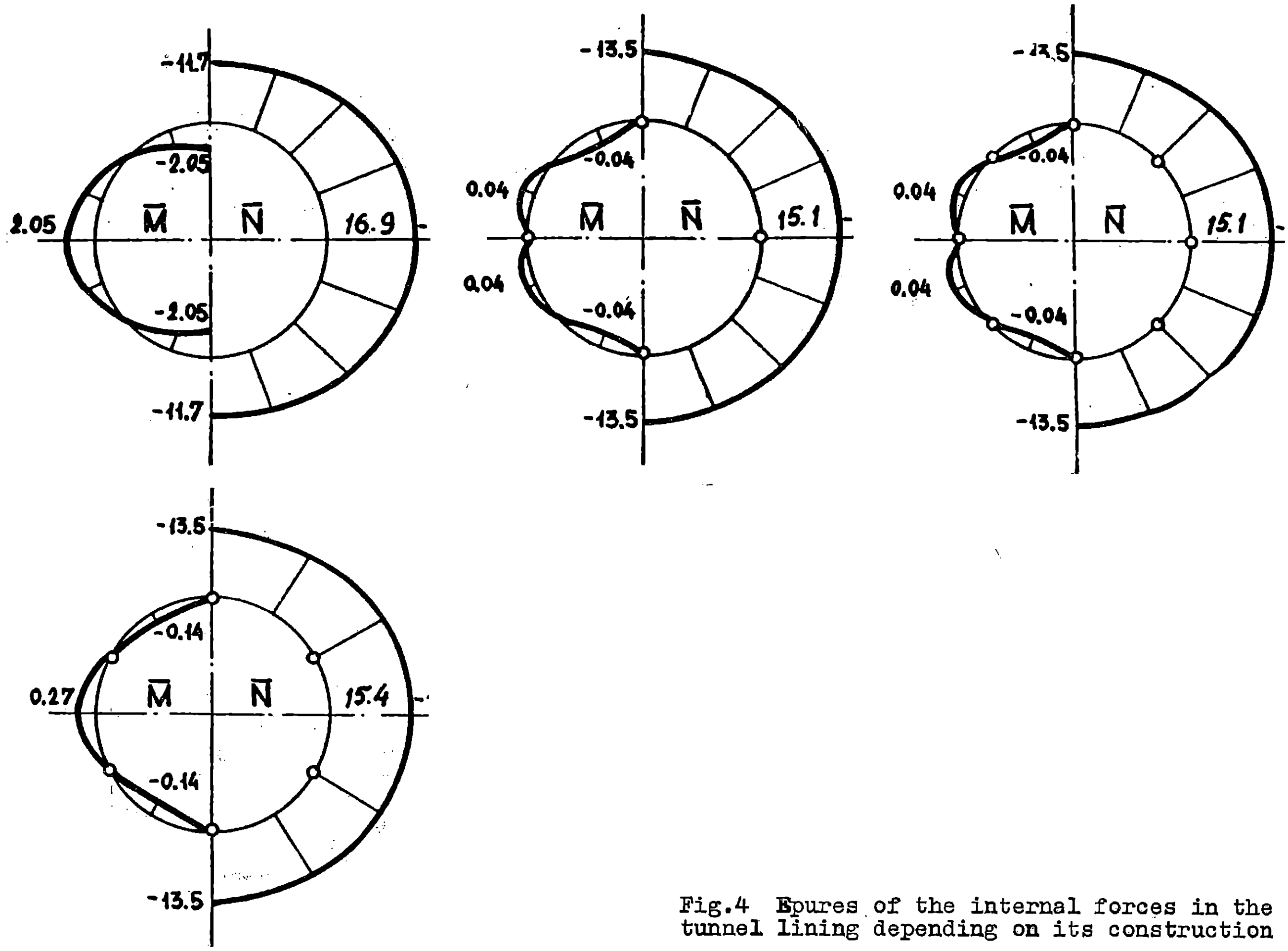

Fig.4 Epures of the internal forces in the tunnel lining depending on its construction

$\alpha^{*}\gamma H = 7.9\cdot10^{-2}$ MPa, $E_0 = 250$ MPa, $\nu_0 = 0.35$, $K_\sigma = 400$ MN/m^3, $r = 2.64$ m, $t = 0.15$ m, $E_1 = 34\cdot10^3$ MPa.

3 COMPUTATION AND RESULTS

Investigation of the pre-fabricated tunnel lining stress-strain state with articulated joints of the blocks allows a dependence of a lining bearing capacity from the number and places of the articulated joints on the tunnel cross-section to be fixed.

Epures of the internal forces $\bar{M} = M/\gamma H$ ($1\cdot10^2$MN·m/MPa) and $\bar{N} = N/\gamma H$(MN/MPa) of several lining construction are shown in Fig.4. For comparison the monolithic lining is represented in picture a. The input data for computation are the following: $r = 2.0$ m, $E_1/E_0 = 1000$, $\nu_0 = 0.35$, $\lambda = 0.54$, $t_1 = 0.2$ m.

The lining bearing capacity is characterized by the $(\alpha^{*}\gamma H)_{lim}$ limited magnitude in confirmation to the altimate strength of the lining material. In fig.5 the results of the lining bearing capacity computation are represented.

The prefabricated lining consists of four, six or eight reinforced concrete blocks with articulated joints. For comparison the monolithic reinforced concrete lining (1) is also

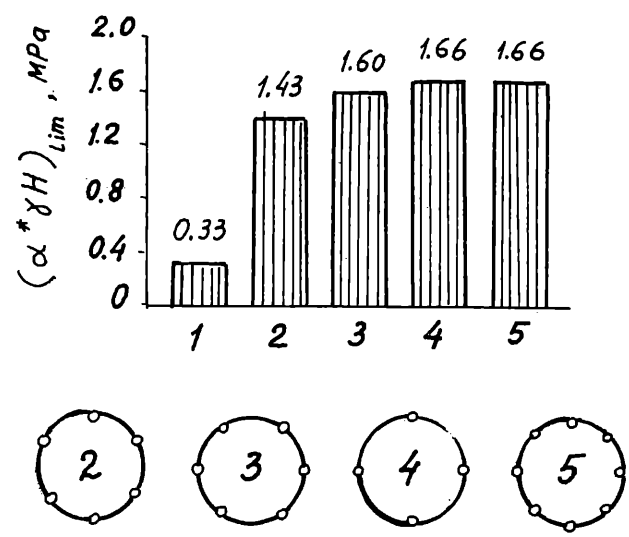

Fig.5 Bearing capacity of the tunnel lining depending on its construction

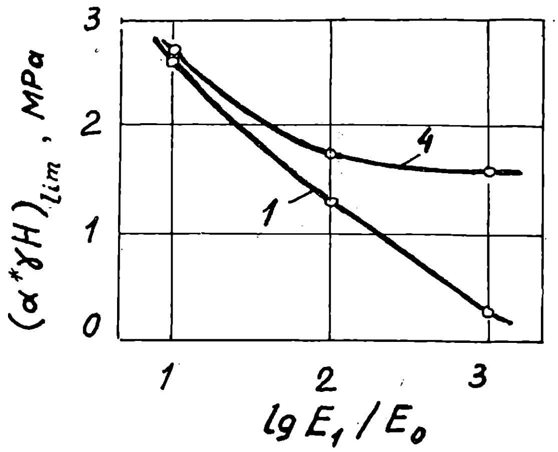

Fig.6 Correlation between bearing capacity of the monolithic (1) and prefabricated linings (2) and the E_1/E_2 Ratio of the E_1 reinforced concrete and E_2 ground deformation modulae

considered. Characteristics of all the lining constructions are the following: r = 2 m, E_1/E_0 = 1000, ν_0 = 0.35. Double effective reinforcement consists of six 12 mm diameter bars ($A_s = A'_s$ = 6.78 cm^2 per m).
The soft ground are effective field of prefabricated lining application. Dependences of the lining bearing capacity on the ground deformation modulus are shown in Fig.6.

On the base of comparison bearing capacity of the monolithic tunnel lining and the prefabricated one (like 4 in Fig.5) with the same characteristic (see Fig.6) one can come to the conclusion, that the most effective application of the prefabricated lining with articulated joints takes place if $E_1/E_0 > 10$.

4 CONCLUSIONS

Several conclusions can be made from this study.

The discrete bar system may be used for simulating the ground massif if there is no analytic solution of the problem. At the same time the tunnel lining design scheme must simulate a ground-lining interaction. Schemes of the so called "active" loads are not to be used.

The prefabricated tunnel lining with articulated joints is most effective in soft ground. The E_0 ground deformation modulus may be $E_0 < 2000$ MPa.

The bearing capacity of the prefabricated lining is the greatest if the articulated joints are located on horizontal diameter of the tunnel.

The blocks of tunnel lining must be installed without displacements of joints in neighbouring rings.

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