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Nina N. Fotieva
Tula State University, Russia

Nikolay S. Bulychev
Tula State University, Russia

Andrew S. Sammal
Tula State University, Russia

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Nina N. Fotieva
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Pr. Lenin 92, Tula, Russia

Nikolay S. Bulychev
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Andrew S. Sammal
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ABSTRACT

The original approach to the problem of designing circular tunnel linings located near slopes upon seismic effects consisting in the determination of the most unfavourable stress state in the every lining radial section at different combinations and any directions of long longitudinal and shear waves propagating in the plane of the tunnel cross-section is described in the paper presented. The analytical methods of determining the tunnel lining stress state caused by static loads namely by the rock own weight and the vertical load uniformly distributed on the part of the inclined straight boundary simulating the weight of building or structure on the surface are also described.

INTRODUCTION

At the design and construction of underground structures in seismic regions it is necessary to take into account that those structures besides usual static loads may be subjected to Earthquakes effects consisting in spreading long seismic compressive-tensile and shear waves in the rock mass the combinations and directions of which respectively to the underground structure are unknown in advance. That is why the original approach to the problem of designing tunnel linings under the Earthquake effects has been developed at Tula State University [Fotieva 1980].

According to that approach the design consists in determining the most unfavourable lining stress state at any combinations and directions of long longitudinal (compressive-tensile) and shear waves propagating in the plane of the tunnel cross-section. On the base of that approach analytical methods of designing the deep tunnel linings, namely - multi-layer linings of circular tunnels and vertical shafts [Bulychev 1994], linings of an arbitrary cross-section shape [Fotieva 1980] including those constructed with the application of grouting [Fotieva *et al.* 1998], shotcrete linings [Fotieva, Bulychev 1996] including those in combination with anchors [Fotieva *et al.* 1992], multi-layer linings of mutually influencing parallel circular tunnels [Fotieva, Kozlov 1992] with the determination of optimal distances between them [Fotieva, Kozlov 1988], rock-bolting for circular openings [Bulychev, Stepanyan 1992] have been developed. The above approach, the basic principles of designing the deep tunnel linings under seismic effects and the mentioned methods have been included in the Russian standards [Instruction 1982, Guide 1996] and widely applied in projects of transport and power-stations tunnels.

However all the methods described are intended for designing linings of deep tunnels (the depth $H>3B$, where B is the most size of the tunnel cross-section) when the influence of the Earth surface may be neglected.

The aim of the work presented is developing the method for designing linings of circular shallow tunnels taking into account the influence of the Earth surface which may be inclined to the horizontal one and based on the same principles as the above methods.

THE METHOD OF DESIGNING TUNNEL LININGS UNDER SEISMIC EFFECTS

With the aim of designing circular tunnel linings located near slopes under seismic effects the two plane quasi-static problems of the elasticity theory are considered. The design schemes of those problems are shown in Fig 1 a, b.

Here the S_0 semi-infinite linearly deformable medium restricted by a straight boundary $L_0^{'}$ inclined under an arbitrary β angle to the horizontal axis $Ox^{'}$ and circular curve L_0 having the R_0 radius, the mechanical properties of which are characterised by the E_0 deformation modulus and the ν_0 Poisson's ratio, simulates a rock mass.

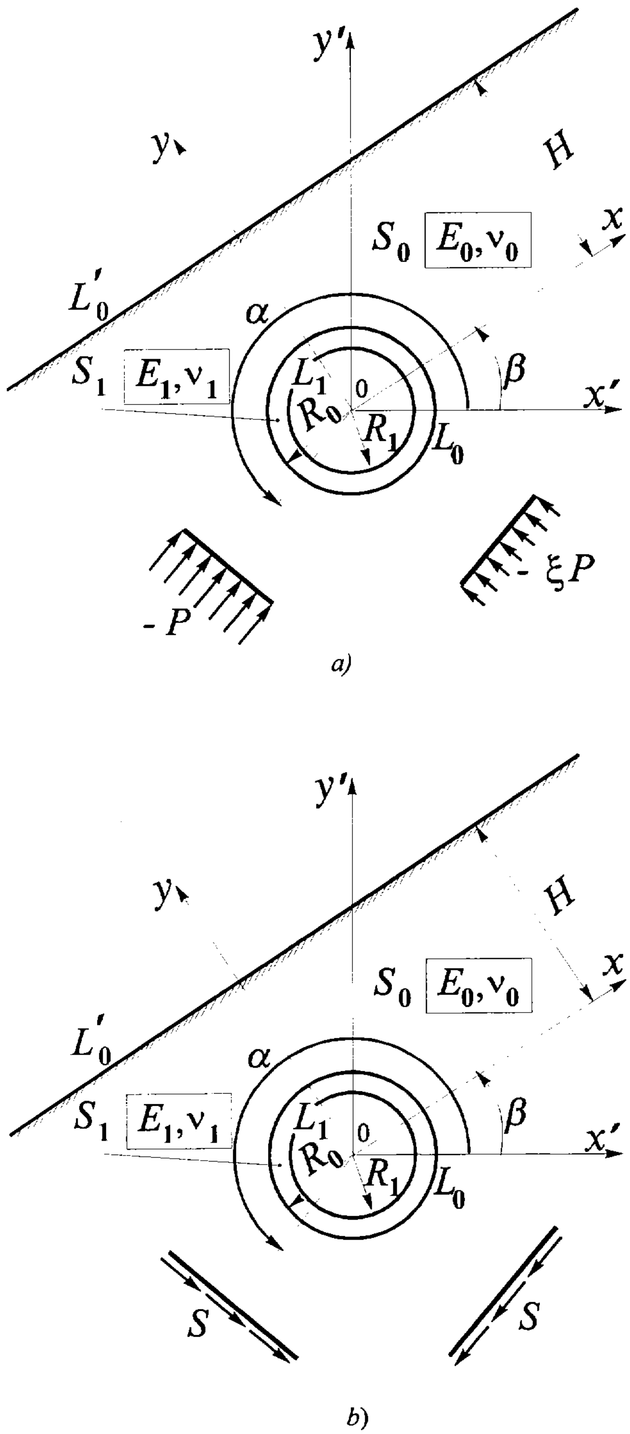

Fig. 1. Schemes for designing tunnel lining upon the action of a long arbitrary directed longitudinal wave (a) and shear wave (b).

The S_1 ring with the L_1 internal outline of the R_1 radius from the material having the E_1 and ν_1 deformation characteristics simulates a tunnel lining constructed on the H distance from the slope surface.

In the first problem (Fig. 1 a) the S_0 medium undergoes on the infinity a two-dimensional compression by stresses P and ξP , directed under an arbitrary α angle with respect to the Ox and Oy axes. Those stresses simulate the action of the long arbitrary directed longitudinal wave and are expressed by formulae [Guide 1996]:

$$P = \frac{1}{2\pi} A K_1 \gamma c_1 T_0, \quad \xi = \frac{\nu_0}{1 - \nu_0} \tag{1}$$

where A is the coefficient corresponding to the Earthquake's intensity, K_1 is the coefficient taking admissible damages into account, γ is the rock specific weight, c_1 is the longitudinal waves velocity, T_0 is the prevailing period of rock particles oscillation.

In the second problem (Fig. 1 b) the S_0 medium is subjected on the infinity to the pure shear under the angle α by the S stresses simulating the action of a long arbitrary directed shear wave and being expressed by formula [Guide 1996]

$$S = \frac{1}{2\pi} A K_1 \gamma c_2 T_0 \tag{2}$$

where c_2 is the velocity of shear waves.

The ring and medium undergo deformation together i.e. conditions of the stresses and displacements vectors continuity are satisfied on the L_0 contact line. The L'_0 boundary and the L_1 internal outline of the ring are free from loads.

The first problem (Fig. 1 a) has been solved with the application of the complex variable analytic functions theory [Muskhelishvili 1966] using the apparatus of complex series, the analytical continuation of the complex potentials characterising the lower semi-plane stress-strain state in the upper semi-plane across the straight boundary [Aramanovich 1955] and the corresponding iteration process [Fotieva *et al.* 1996]. The second problem (Fig. 1 b) has been obtained as the particular case of the first one assuming the $P = S, \quad \xi = -1$ and exchanging the α angle on $\alpha + \pi / 4$ value.

From the solution of the first problem the $\sigma^{(P)}$ stresses (here the σ symbol signifies all components of a stress tensor), appearing in the lining due to the action of a long longitudinal wave falling under an α arbitrary angle are determined; from the solution of the second problem the $\sigma^{(S)}$ stresses called forth by a shear wave are obtained.

Further, the sum and the difference of general expressions for $\sigma_\theta^{(P)}$ and $\sigma_\theta^{(S)}$ normal tangential stresses characterising the lining stress state caused by mutual actions of longitudinal and shear waves passing simultaneously (the worst case) are investigated in every point of the internal lining cross-section outline on the extreme relatively the α angle of the waves falling.

With that aim the following equations are solved

$$\frac{\partial}{\partial\alpha}[\sigma_\theta^{(P)} \pm \sigma_\theta^{(S)}]=0 \tag{3}$$

and for every point such a combination of waves and such an angle of their falling at which normal tangential stresses in the points considered are maximal by their absolute are determined. It allows the envelope diagram of normal tangential stresses on the internal lining outline to be obtained analytically.

The stresses along the external lining outline, longitudinal forces and bending moments in every lining radial section are determined at such a combination and such a direction of waves at which the normal tangential stress in that section has a maximal absolute value.

The stresses and forces obtained that way are assumed to have the signs "plus" and "minus" and summed up with stresses and forces appearing due to the other acting loads in their most unfavourable combinations. After that a sections strength test upon compression and tension is made.

If the lining is not anchored to the rock and is designed with an allowance of the crack formation we assume that the tensile normal loads are not transferred upon the lining. In this case the action of the longitudinal waves in the tension phase is not to be taken into account and the design is made on the base of the two different envelope diagrams of normal tangential stresses, obtained using the maximal absolute values of the compressive (negative) stresses and tensile (positive) ones, called forth by mutual actions of shear waves and longitudinal waves in the compression phase.

Examples of the Design

The shallow tunnel lining has been considered at the following input data: $\beta = 15^o$, $R_0 = 3$ m, $R_1 = 2{,}8$ m, $H = 5$ m, $E_0 = 5750$ MPa, $\nu_0 = 0.3$, $E_1 = 23000$ MPa, $\nu_1 = 0.2$, $\gamma = 0.025$ MN/m³, $AK_1 = 0.1$, $T_0 = 0.5$ s.

Distributions of maximal compressive normal tangential stresses σ_θ^{in}, MPa which may appear on the lining cross-section internal outline and corresponding them σ_θ^{ex}, MPa stresses on the external outline are shown in Fig. 2 by solid lines. The maximal tensile stresses σ_θ^{in}, MPa and corresponding them σ_θ^{ex}, MPa stresses are given by dash lines.

So, the two curves showed both by solid lines and by dash lines are applied for the evaluation of the lining cross-section strength if the lining is designed with the allowance of cracks formation. In the opposite case the stresses given by solid lines are applied assumed to have signs "plus' and "minus". We also can notice that stresses decrease with the increasing of the tunnel depth gradually approaching to the values obtained without taking into account the influence of the Earth surface [Fotieva 1980]. The maximal compressive and tensile stresses determined by that way have the following values: σ_θ^{in} = = - 3.66 MPa and 0.70 MPa; σ_θ^{ex} = -3.0 MPa and 0.23 MPa correspondingly.

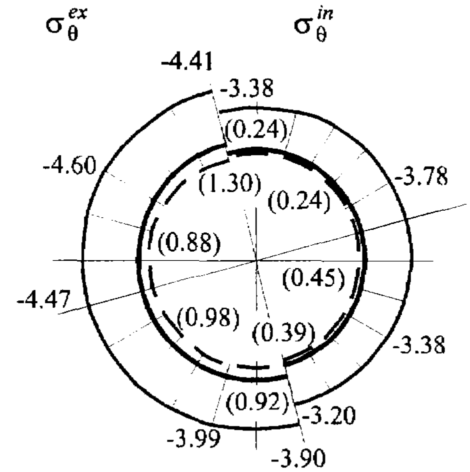

Fig. 2. Distribution of maximal compressive and tensile stresses σ_θ^{in} on the internal outline and corresponding them stresses σ_θ^{ex} on the external outline of the lining cross-section.

The comparison shows that in case considered the influence of the surface results in substantial increasing both compressive and tensile stresses.

So, the possible maximal stress increases on 26 % both on the internal and the external outline of the lining cross-section.

The maximal tensile stress in the lining designed with the allowance of cracks formation increases due to the influence of the Earth surface on 86 % on the lining internal outline and on 96 % on the external outline of the lining.

DESIGNING TUNNEL LININGS UNDER THE ACTION OF STATIC LOADS

Methods of designing shallow tunnel linings located near high slopes are based on the investigation of interacting the tunnel lining and the surrounding rock mass as elements of a united deformable system undergoing the actions of gravitational forces and the weight of buildings or structures located on the surface. The design scheme is given in Fig. 3.

The action of the rock own weight is simulated by a presence of initial stresses in the S_0 area determined by formulae

$$\sigma_x^{(0)} = -\gamma\lambda\alpha^*(H-y)\cos\beta;$$
$$\sigma_y^{(0)} = -\gamma\alpha^*(H-y)\cos\beta; \tag{4}$$
$$\tau_{xy}^{(0)} = -\gamma\alpha^*(H-y)\sin\beta$$

where λ is the lateral pressure coefficient in an intact rock, α^* is the correcting multiplier introduced for an approximate registration of the influence of the l distance between the lining being constructed and the tunnel face which may be determined by empirical formula [Instruction 1983]:

$$\alpha^* = \exp(-1.3l / R_0) \quad (5)$$

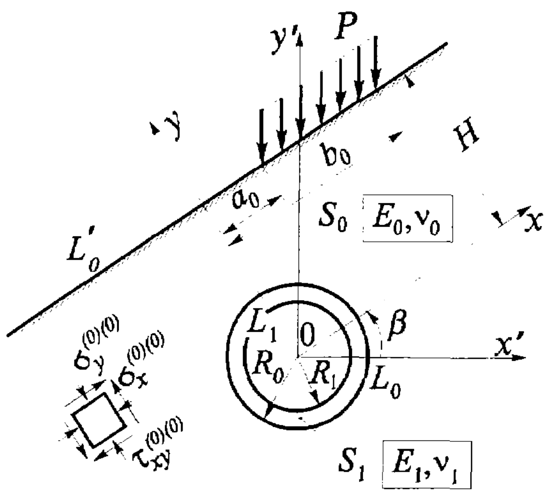

Fig.3. Scheme for designing tunnel linings under actions of static loads.

The weight of a building or structure on the surface is simulated by a vertical load of the P intensity uniformly distributed on the part $a_0 \le x \le b_0$ of the L_0' straight boundary of the semi-plane.

There are two cases into consideration - when the structure on the surface is being built after the tunnel construction or vice-versa when the tunnel is being constructed under the already existing structure. In the latter case initial displacements in the rock mass caused by the P pressure are excluded from the consideration and the above α^* multiplier is introduced into the P value.

The corresponding problems of the elasticity theory have been solved by way mentioned above.

The influence of the rock creep may be taken into account applying the method of variable modules according to which the rock deformation characteristics in a solution of the corresponding elasticity theory problem are represented as time functions [Amusin, Linkov 1974]. In that case the α_t^* correcting multiplier also depends on time and may be determined according to the work [Fotieva 1980].

Examples of Designing Tunnel Linings on Static Loads

The calculations for determining the same tunnel lining stress state caused by gravitational forces have been carried out in shares of the α^* value at the above input data and $\lambda = 0.54$.

Distributions of the normal tangential stresses $\sigma_\theta^{ex} / \alpha^*$, $\sigma_\theta^{in} / \alpha^*$ (in MPa), appearing on the external and internal outlines of the lining cross-section due to the action of the rock own weight are correspondingly shown in Fig. 4 a, b by solid lines. For comparison the same stresses determined at the $\beta = 0^o$ are shown in Fig. 4 a, b by dash lines (values of stresses are given in brackets).

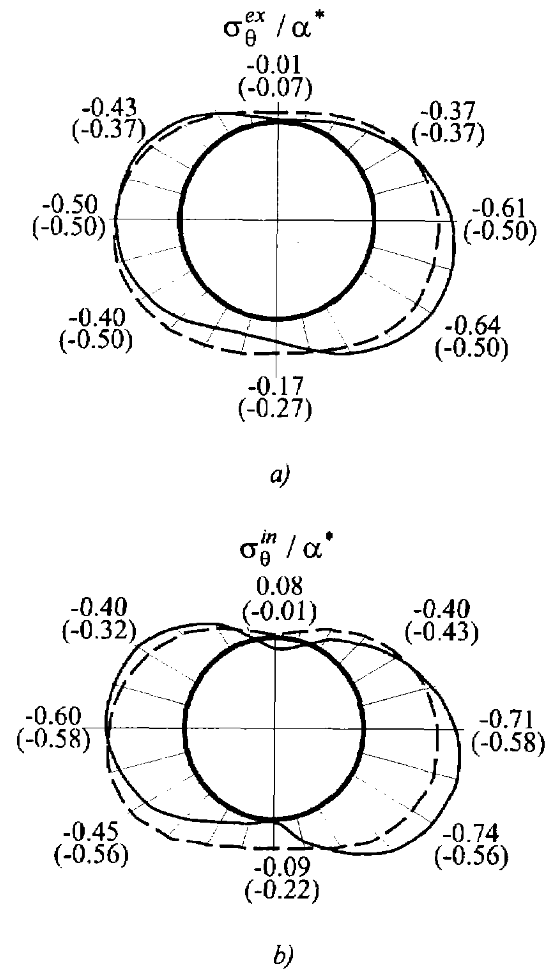

Fig. 4. Distributions of stresses in the lining caused by the rock own weight.

Similar distributions of the $\sigma_\theta^{in} / \alpha^*$ stresses obtained at the $E_0 / E_1 = 0.05,\ 0.1,\ 0.25,\ 0.5$ are shown in Fig. 5 by curves 1, 2, 3, 4, where the circular internal outline is represented as a straight line. Here the θ angle is counted off from the horizontal.

The dependencies of extreme $\sigma_{\theta\,extr}^{in} / \alpha^*$ stresses on the lining cross-section internal outline on the β angle are shown in Fig. 6 (here the β angle changes up to theoretical value

90^o corresponding to the free infinite vertical slope). The dependencies obtained at the relations $E_0 / E_1 = 0.05$, 0.1, 0.25 are shown by curves 1, 2, 3 correspondingly.

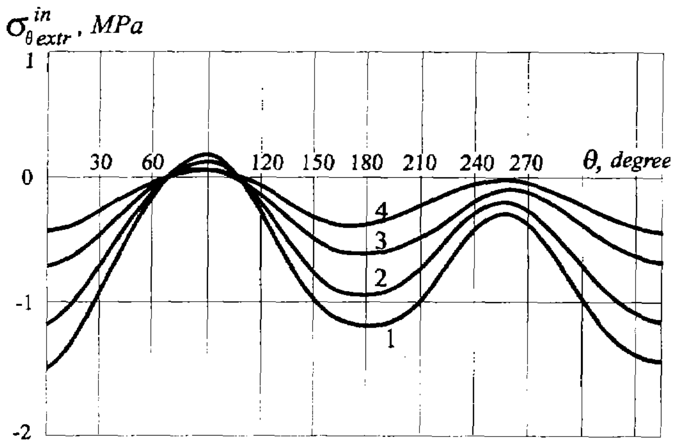

Fig. 5. Distributions of the σ_θ^{in} stresses at different E_0 / E_1 relations.

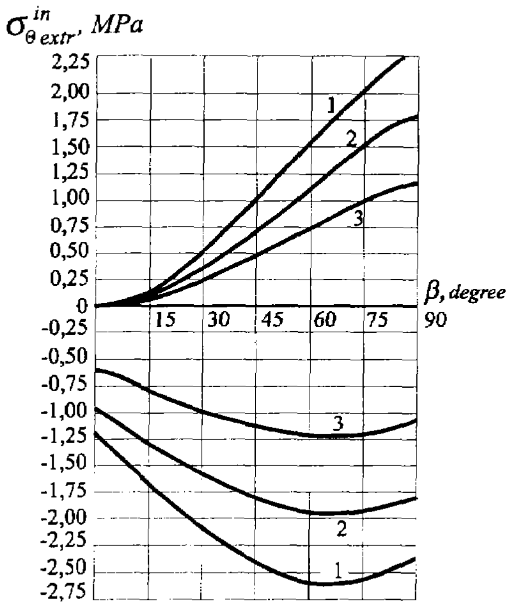

Fig. 6. Dependencies of $\sigma^{in}_{\theta\, extr} / \alpha^*$ stresses caused by the rock own weight on the β angle.

Distributions of the $\sigma_\theta^{ex} / P\alpha^*$, $\sigma_\theta^{in} / P\alpha^*$ stresses appearing in the same lining ($\beta = 15^o$) due to the action of a load of $15\ m$ length ($a_0 = 3\ m$, $b_0 = 18\ m$) existing on the surface before the tunnel construction are shown in Fig. 7 a, b by solid lines; the same stresses at $\beta = 0$ are given by dash lines.

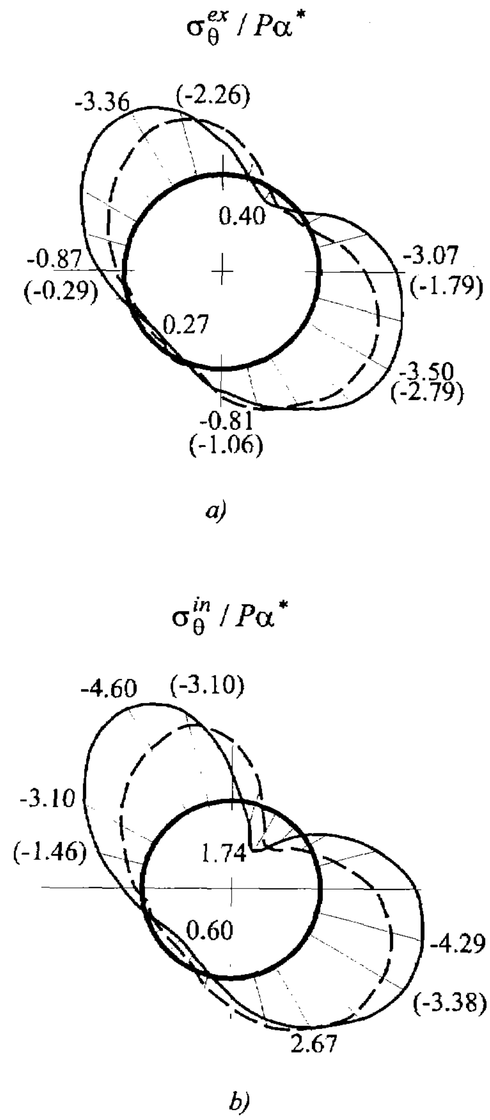

Fig. 7. Distributions of stresses in the lining caused by the surface load..

CONCLUSIONS

As follows from the results obtained the stresses which may appear in the tunnel linings located near slopes during the Earthquakes surpass the ones caused by the rock own weight and are comparable with the ones caused by a surface loads (even at the $\alpha^* = 1$). Besides that the influence of the Earth surface results in increasing the stresses especially when that surface is inclined to the horizontal one.
We can also mark that the method described may be generalised for designing shallow tunnel linings of an arbitrary cross-section shape and for calculation of linings of parallel mutually influencing shallow tunnels.

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