11 Mar 1998, 4:30 pm - 6:30 pm

Case Histories of Designing Tunnel Linings in Seismic Regions

Nina N. Fotieva
Tula State University, Tula, Russia

Nikolay S. Bulychev
Tula State University, Tula, Russia

Proceedings: Fourth International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, March 9–12, 1998.

CASE HISTORIES OF DESIGNING TUNNEL LININGS IN SEISMIC REGIONS

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

Nikolay S.Bulychev
Tula State University
Pr. Lenin 92, Tula, Russia

Paper No. 6.01

ABSTRACT

The original approach to the problem of designing tunnel linings upon the seismic effects consisting in the determination of the most unfavourable stress state in the every lining normal section at different combinations and any directions of long longitudinal and shear waves propagating in the plane of the lining cross-section is described in the paper presented. The analytical methods and corresponding computer programs based on that approach have been developed for the design of tunnel linings of an arbitrary cross-section shape, circular linings including the multi-layer ones of the complex of parallel mutually influencing tunnels etc. Two case histories of applying those methods at the design and construction of the railway tunnel and six parallel vertical turbine shafts of the hydro-electric power- station are given.

KEYWORDS

Design, tunnel, lining, seismic effects, long seismic waves, stresses.

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 upon 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.

METHODS OF THE TUNNEL LININGS DESIGN

With the aim of designing tunnel linings of an arbitrary cross-section shape two plane quasi-static contact problems of the elasticity theory are considered. The design schemes of those problems are shown in Figures 1,a,b.

Here the S_l ring the material of which is characterised by the E_l deformation modulus and the ν_l Poisson's ratio simulates the tunnel lining.

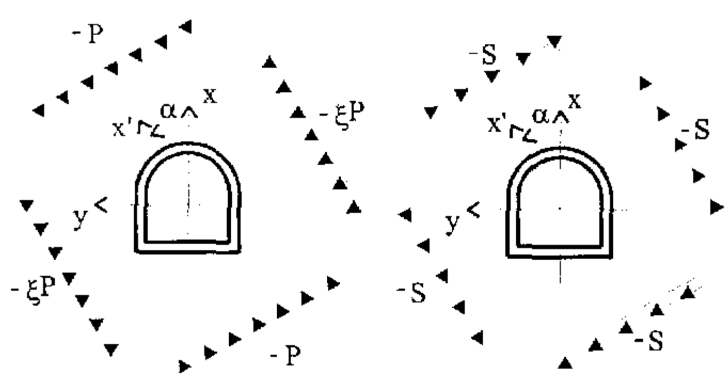

Fig. 1 Schemes for designing tunnel lining upon the action of a long arbitrary directed longitudinal wave (a) and shear wave (b).

The S_0 infinite linearly deformable medium with the E_0, ν_0 deformation characteristics simulates the surrounding rock mass.

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

$$P = \frac{1}{2\pi} AK_1\gamma c_1 T_0, \quad \xi = \frac{\nu_0}{1-\nu_0} \qquad (1)$$

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_l 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

$$S = \frac{1}{2\pi} AK_1\gamma c_2 T_0 \qquad (2)$$

where c_2 is the velocity of shear waves.

The $\sigma^{(P)}$ stresses in the lining (here all the components of the stress tensor are designated by a symbol σ) caused by the longitudinal wave falling under an arbitrary angle α , are determined from the solution of the first problem. The stresses $\sigma^{(S)}$ caused by the shear wave of the same direction are being determined from the solution of the second problem.

Then the sum and difference of analytical expressions for normal tangential stresses $\sigma_\theta^{(P)}$, $\sigma_\theta^{(S)}$ characterising the lining stress state in the worst case of the simultaneous action of longitudinal and shear waves, in every points of the internal lining cross-section outline are investigated on the extremum with respect to the α angle of the waves falling. With that aim the equations

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

are solved. After that for every point such a combination of waves of different kinds and such an angle of their falling at which the normal tangential stress in this point has a maximal absolute value are determined. It allows the surrounding diagram of normal tangential stresses on the internal outline of the lining cross-section to be obtained analytically. The normal tangential stresses on the external outline, bending moments and longitudinal forces in every normal section of the lining are being determined namely at such combination and direction of waves at which the normal tangential stress on the internal outline in that section has a maximal absolute value. The stresses and forces in the lining obtained by way described are taken with signs "plus" and "minus" and summed up with the ones caused by the other loads. After that the strength of lining sections is being controlled on the compression and tension.

The approach described allows all possible multitude of seismic effects in their most unfavourable combination for every normal section of the structure to be taken into account.

If the lining is not anchored to the rock and is designed with the allowance of the cracks formation, the action of the longitudinal waves in the tension phase is excluded from the consideration and the design is fulfilled on the base of two surrounding diagrams of the normal tangential stresses obtained taking into account the maximal compressive and tensile stresses caused by the mutual action of the shear waves and the longitudinal ones in the compressive phase. In that case two different designed diagrams of the stresses and forces are obtained. Those stresses and forces after their being summed up with the ones determined for other kinds of loads are applied for the lining strength control on the compression and tension.

The analytical solutions of contact problems shown in Fig. 1,a,b have been obtained in the work by Fotieva [1980] with the application of the complex variable analytic functions theory using apparatus of the conform mapping and complex series [Muskhelishvili 1966].

The approach mentioned above and the basic principles of designing tunnel linings upon seismic effects have been included in the standard [Instruction 1983] and widely applied in projects of transport tunnels and tunnels of power-stations.

Later on the similar base analytical methods and corresponding computer programs for designing tunnel linings of an arbitrary cross-section shape being constructed with the application of grouting [Klimov 1993], linings and multi-layer linings of the complex of parallel mutually influencing circular tunnels including the determination of minimal safe distances between them [Fotieva, Kozlov 1992] have been created.

CASE HISTORIES

The case histories of the application of the methods mentioned above at designing tunnel linings in seismic regions are described below.

The design of the railway tunnel

The tunnel of the Baikal-Amur railway in Siberia of 7 km length, 9.5 m height and 7.5 m width supported by the monolithic concrete lining of 0.4 m thickness is located on the 50 m depth in the gneisoid granite very jointed and fractured up to druss and debris subjected to degradation;

the seismic activity of the region is 9 degrees according to MSK-64 scale. The lining has been designed with the allowance of fissures appearance.
The input data for designing the certain part of the tunnel were the following: $E_0 = 700$ MPa, $\nu_0 = 0.23$; $E_1 = 35000$ MPa, $\nu_1 = 0.15$, $\gamma = 0.027$ MN/m³, $AK_1 = 0.1$. $T_0 = 0.5$ sec.
The distributions of the designed M bending moments and the N longitudinal forces in the lining corresponding the maximal compressive stresses which are possible at the Earthquake are given in Fig. 2 by solid lines; the same forces corresponding to the maximal tensile stresses are shown by dash lines.

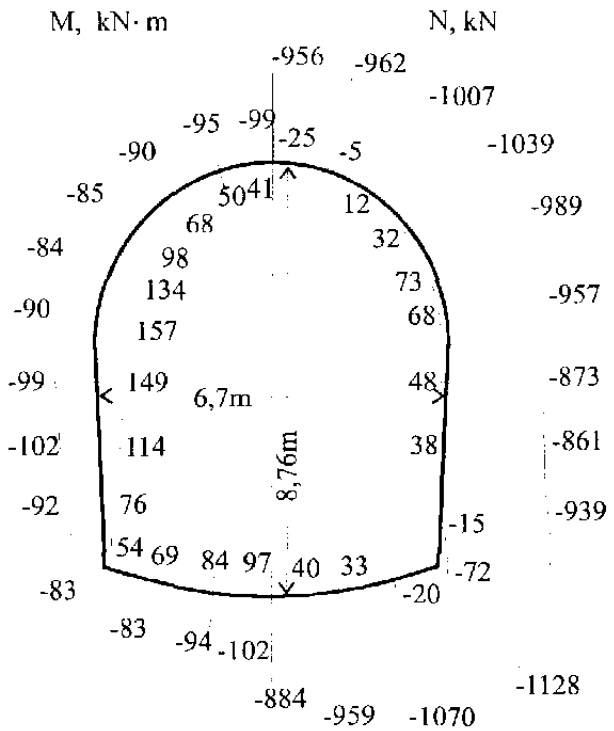

Fig. 2 Distribution of the M bending moments and the N longitudinal forces corresponding to the maximal compressive (solid lines) and tensile (dash lines) stresses.

The similar calculations (more than 2000 variants) together with the ones concerning the action of the rock's own weight have been fulfilled for the linings of four tunnels of the Baikal-Amur railway.

Designing vertical turbine shafts of a power-station

The linings from concrete with internal steel layer of six parallel vertical turbine pressure shafts of the Rogun power-station in Tadjikistan (the external radii of tunnels are 3.93 m, the distances between their centres are 26.3 m) have been designed with the application of the method [Fotieva, Kozlov 1992] mentioned above.
The design scheme of that method is a linearly deformable medium weakened by an arbitrary number of arbitrary located circular holes of different radii supported by multi-layer rings fulfilled from different materials. The medium is loaded on the infinity by the same stresses as in Fig. 1 and two the same problems of the elasticity theory are considered. The most unfavourable stress state of the linings is determined by solving the equations similar to the (3) obtained for internal outlines of the every layer of the every lining.
The design of six parallel vertical shafts of the Rogun power station has been fulfilled taking into account the actions of the water internal head, the rock's own weight and the Earthquake effects.
The general input data are the following: thickness of the concrete layers is 0.4 m; thickness of the steel layers is 0.03 m (internal radii of the steel layers are 3.5 m); the concrete deformation modulus and the Poisson's ratio are correspondingly $E_1 = 24000$ MPa, $\nu_1 = 0.15$; the steel deformation modulus and the Poisson's ratio are correspondingly $E_2 = 200000$ MPa, $\nu_2 = 0.3$; the rock unit weight is $\gamma = 25.5$ kN/m³ ; the lateral pressure coefficient in an intact rock mass is $\lambda = 1$; the internal water pressure is p=2.5MPa; the coefficient corresponding to the Earthquake intensity is A = 0.4; the coefficient taking into account the admissible damages is $K_1 = 0.25$; the prevailing period of the rock particles oscillation is $T_0 = 0.5$ s.
The calculations have been fulfilled for two kinds of the rocks with different E_0 deformation modules, ν_0 Poisson's ratios, initial stresses $\sigma_r^{(0)}$ in the intact rock mass and c_l values of the long elastic waves velocity. Those characteristics are given in the Table 1.

Table 1. The variants of the input data

Parameters	Variants	
	I	II
E_0, MPa	30000	36000
ν_0	0.3	0.33
$\sigma_r^{(0)}$, MPa	14.0	17.0
c_l, m/s	4250	4600

Distributions of the σ_θ normal tangential stresses in the lining layers obtained as the results of calculations at the loads mentioned above are shown in Figures 3,a,b,c, and 4,a,b,c for both variants of the input data correspondingly (in Figures 3,c and 4,c the maximal compressive (negative) and tensile (positive) normal tangential stresses at different combinations of the mutually acting longitudinal and shear waves of any directions in the plane of the tunnels cross-section are given). Taking into account the symmetry the results are represented for three left tunnels; the stresses in the steel and concrete layers are shown in the upper and lower parts of figures correspondingly.

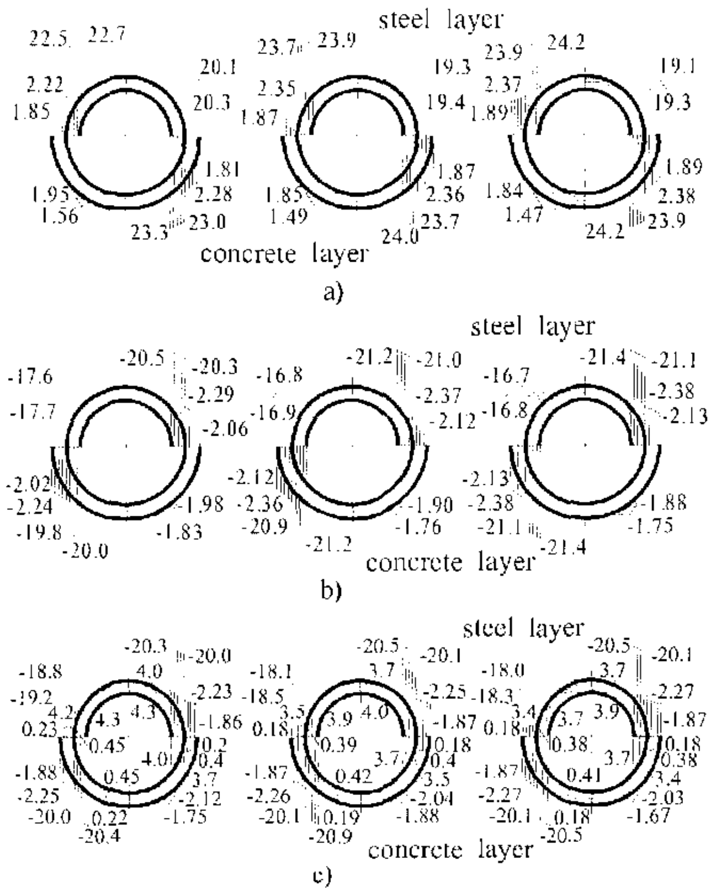

Fig. 3 The σ_θ normal tangential stresses in the steel and concrete linings layers at the action of the internal water head (a), the rocks own weight (b) and the Earthquake effects (c) obtained for the first variant of the input data.

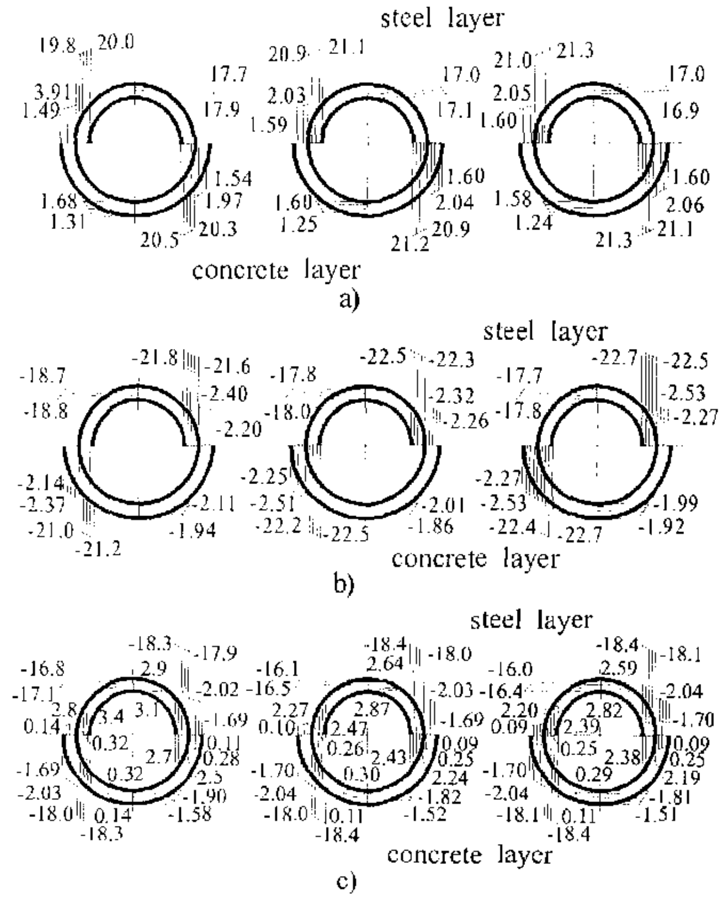

Fig. 4 The σ_θ normal tangential stresses in the steel and concrete linings layers at the action of the internal water head (a), the rocks own weight (b) and the Earthquake effects (c) obtained for the second variant of the inpu data.

As follows from the results obtained for both variants the tensile σ_θ stresses in the concrete layers surpass the designed concrete strength on the tension $R_{bt} = 0.75$ MPa (Fig. 3,a and 4,a). That is why additional calculations have been fulfilled at the value of the E_0 concrete deformation modulus decreased up to $E_0 = 11600$ MPa taking into account the possible crack formation. Besides that taking into account that stresses appearing in steel layers are small their thickness has been decreased up to 0.02 m.

For additional control the calculations have been fulfilled at the steel layers thickness $\Delta_2 = 0.02$ m and a still lower value $E_1 = 7000$ MPa of the concrete deformation modulus. Those calculations also confirmed the possibility of the steel layer thickness to be decreased.

So, with the aid of calculations fulfilled it was be shown that the bearing capacity of the lining of the vertical turbine pressure shafts of the Rogun power-station is being secured at the steel layers thickness of $\Delta_2 = 0.02$ m.

In general the experience of applying the methods developed shows that those methods may be useful and effective at designing tunnel linings in seismic regions.

REFERENCES

Fotieva, N.N. [1980]. " *Design of underground structures support in seismically active regions".* Nedra, Moscow.

Fotieva, N.N., A.N.Kozlov, [1992]. "*Designing linings of parallel openings in seismic regions."* Nedra, Moscow.

"Instruction to designing mining workings and linings calculation ." [1983]. Stroyizdat, Moscow.

Klimov, Y.I. [1991]. "Designing tunnel linings undergoing seismic effects with rock grouting to be taken into account." in *Underground Structures Mechanics,* Tula, Russia.

Muskhelishvili, N.I. [1966]. "*Some basic problems of mathematical elasticity theory."* Nauka, Moscow.