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ABSTRACT

The paper describes the application of analytical methods if tunnel linings design under static, tectonic and seismic effects for the design of the transport tunnel Albors in Iran. The methods are based on solutions of the corresponding plane problems of the elasticity theory simulating the initial stress fields in the rock mass caused by gravitational or tectonic forces and actions of long arbitrary directed longitudinal (P) and shear (S) waves propagating in the plane of the tunnel cross-section. For the design of tunnel linings under seismic effects the original approach has been applied consisting in the determination of maximal compressive and tensile circumferential stresses wich may appear in points of the lining internal outline at any combinations and directions of P- and S- waves.

DESIGN MODELS AND INPUT DATA

Alborz Tunnel lining is subjected to two types of loads caused by initial stress field in the rock mass due to rock dead weight and tectonic forces as well as possible Earthquake activities. Analytical methods of Underground Structure Mechanics as well as contact interaction design schemes of the tunnel lining and the surrounding rock mass have been used for the tunnel lining design.

Rock dead weight and tectonic forces actions

Mechanical properties of the rock mass. Tunnel design model in pre-stressed rock mass shown in Figure 1. Characteristics of mechanical properties of main rock types in the geological section are general deformation modulus E_0 and Poisson's ratio ν_0 are given in Table 1. Since each rock section is featured by interstratifications the design deals with the softest rocks.

Table 1. Design properties of the rock mass

Rock name	Deformation modulus E_0, MPa	Poisson's ratio ν_0	Unit weight γ, MN/m^3
Tuffs	6 000	0.25	0.026
Andesite	8 000	0.22	0.028
Tectonic fracture zone	3 000	0.28	0.025

Note: Design deformation modules of tuff and andesite in the Table 1 are conservative to increase design stress in the lining, i.e. to raise its safety factor.

Mechanical properties of the lining material. Bulk concrete class B30 was picked as a lining material the properties of which were adopted as follow:
- deformation modulus subjected to short-time and long-duration creep E_b=13 800 MPa, Poisson's ratio ν_b=0.2;
- design compressive strength R_b=17 MPa;
- design tensile strength R_{bt} =1.2 MPa.

Calculating a tunnel lining subjected seismic actions the design concrete compressive strength is multiplied by behaviour condition factor m_k=1,2 (influence of short-time seismic loads on strength of materials). Concrete deformation modulus is taken to be equal to initial elasticity modulus (disregarding creep) E_b=32 500 MPa.

In situ stress state of the rock mass. Due to lack of information on principal stresses values and their direction in the rock mass surrounding Alborz Tunnel, estimated initial stress field characteristics were taken from literary sources.

An international scientific team involved over 30 scientists from 18 countries in framework of the International Lithosphere Project has mapped worldwide stress fields in the earth's crust upper part (Zoback *et al* 1989, Markov 1977). Over 7,300 stress orientation entries are included in a digital database. The data included in the World Stress Map are derived mainly from geological observations on earthquake focal mechanism, volcanic alignments and fault slip interpretation. Figure 2 is a generalized version of the World Stress Map in which the orientations of maximal horizontal stress σ_{h1} are plotted on a basis of average topography.

Map analysis brings into light that tectonic stresses are mainly associated with the lithosphere (tectonic) plates interaction boundaries, subduction zones and with direct relation to their motion. Tectonic stresses in the Iranian plate are resulted from its interaction with Turkey, Arabian and Eurasian plates. North of Iran where tunnels are under construction is a part of tectonic zone comprising Caucasus and Middle Asia. Tectonic horizontal stresses are tilted to meridian, so excessive horizontal stresses are expected both along tunnel routes (mainly) and tunnel's cross-sectional planes.

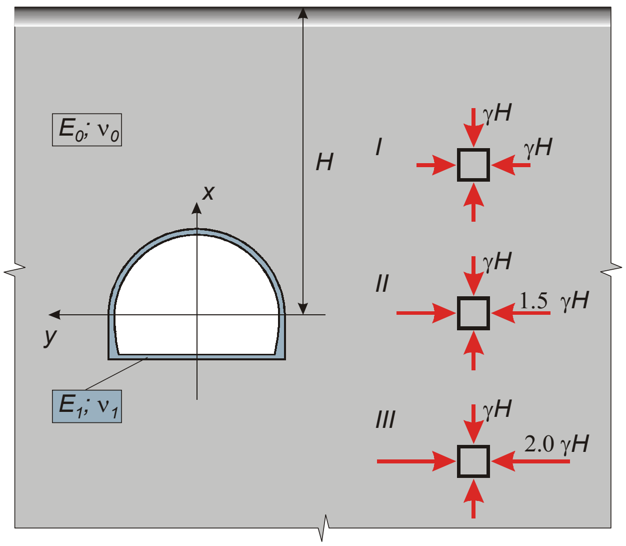


Fig. 1. Design model of tunnel lining contact interaction with tectonically pre-stressed rock mass:
I-III – alternative characteristics of in situ stress state

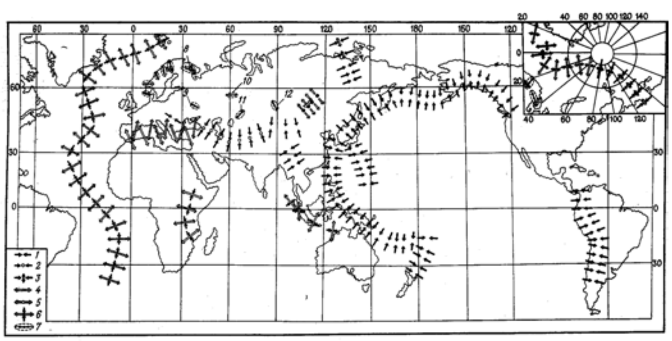


Fig. 2. Generalized world stress map giving mean maximum horizontal stress directions: 1-3 – the most compression; 4-6 – the most tension. Directions of minor principal stress: 1, 4 – are indeterminate; 2, 5 – are vertical; 3, 6 – are horizontal; 7 - directions of the most compressive stress are based upon field measurements in ore mines: 8 – Hibin deposit; 9 – Kursk Magnetic Anomaly; 10 – the Middle Ural; 11 - Dzheskazgan; 12 – Mountain Shoria (redrawn from reference, Markov)

Excessive horizontal compressive stresses in the tunnel construction area are confirmable by rock walls convergence measurement results obtained in the Taloun exploratory-ventilating adit (EVA). Figure 3 illustrates convergence of cross section points NP-19. There EVA crosses geological failure where black tuffs (basalt) contact with andesite and glaucous tuffs.

Measurements originated far from the face (43 m), however the graph shows adit walls are displaced (converged) horizontally for the most part. Also, a considerable difference between C-L and C-R points is recorded.

Evidence of preponderate tilted sub-horizontal initial tectonic rock mass stresses is obtained from field cross-section observations carried out during Taloun drainage-ventilating adit drilling. Rock rupture is concentrated in adit roof mainly somewhat vertically misaligned. Blasting-resulted rock rupture beyond design adit section occurs in compressive stress concentration spots. This is demonstration of above-mentioned preponderate initial rock stresses axially inclined to skyline to some extent.

It is the practice to express in situ compressive rock mass stress as follows:

$$\sigma_v = -\gamma H; \quad \sigma_h = -\lambda\gamma H, \qquad (1)$$

where σ_v is vertical stress, MPa; λ is lateral pressure factor; γ is rock unit weight, MN/m^3; H is depth, m.

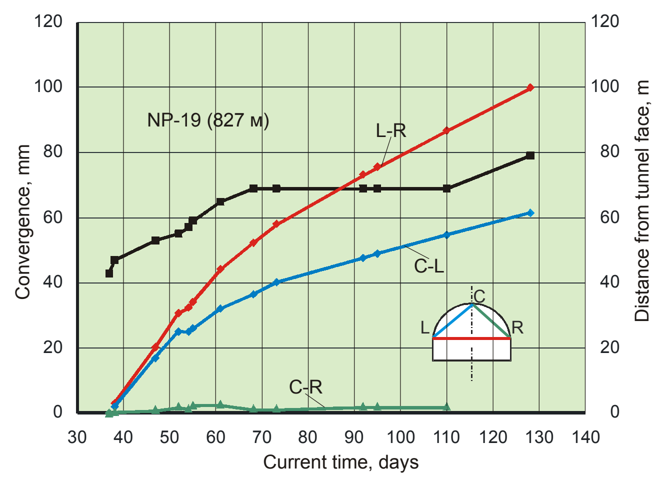


Fig. 3. Convergence of cross section points NP-19 of EVA (pic. 0+827 m from the northern portal) at depth of 125 m and graph of EVA driving

The Middle Asia, which is one of the most seismically active area with complex geology and severe topography, was the base for numerous pilot rock stress studies carried out in mineral deposits. The region is featured by horizontal stress prevalence over vertical ones; maximal compressive stresses are sub-meridian directed for the most. Thereto, the major geological structures (saddles, faults, ridges) are sub-laterally directed there. The Middle Asia is quite similar to the North Iran in geology so this would help to estimate initial stress field related to Alborz Tunnel constructional area in North Iran.

Results of field measurements of in situ rock mass stress as in mineral deposits and subsurface hydroelectric plants construction areas in Caucasus, Kazakhstan, Kirghizia and Tajikistan (Kurlenya 1996) are taken into account.

Grounding on above the following may be concluded on rock mass stress in Alborz Tunnel construction area:

- the rock mass is formed by strong volcanogenic rocks in teconically active area and subjected to tectonic forces;
- maximal principal sub-horizontal compressive stresses are sub-meridian directed;
- maximal rock mass compressive stress can not be too high because the rock mass is near Caspian sea in central part of Iranian plate (see Figure 2)
- tunnel's cross-sectional tectonic stresses correspond to minimum measured sub-horizontal stress σ_{h2} inasmuch as maximum principal stresses were sub-meridian directed.

With above the principal initial stresses are taken for Alborz Tunnel designing as calculated by formulas (1).

The following design (probabilistic) lateral pressure coefficient in tunnel cross section are adopted (see Figure 1):

Table 2. Dependence of lateral pressure coefficient on depth

Depth, m	λ coefficient
50-150	1.0; 1.5; 2.0
200-800	1.0; 1.5

Recommended design lateral pressure factor values are equiprobable, therefore tunnel lining is designed for each λ, and lining strength must be evaluated for the worst conditions.

Stating on the grounds of above, estimation of in situ rock mass stress field in Alborz Tunnel area may be appraised sufficiently probable.

Tunnel lining stress state depends appreciably on tunneling technology, lining erecting lag from the face namely. The α* decreasing factor (to compensate multiplier of initial stress field elements α*γH и α*λγH), calculated by formula is introduced to allow for lining lag from rock exposure with space and time (Bulychev *et al* 2001):

$$\alpha^* = 0.6\exp\left(-1.38\frac{l_0}{r}\right); \quad \alpha^* \geq 0.15\,, \qquad (2)$$

where l_0 is distance between the permanent lining and the tunnel face; r is average tunnel radius.

If lining is far away from the face, α*=0.15.

Seismic effects. The Alborz Tunnel construction area is assumed as a high seismic activity of MM (modified Merkalli scale) intensity VIII, IX and X . All Russian standards for tunnel linings design under seismic effects are MSK-64 scale oriented. Tunnel construction area seismicity by MSK-64 must be found out to design of tunnel linings.

Construction area seismicity. Most (95%) earthquakes happen on lithosphere plate edges (see Figure 2). Few earthquakes happen inside plates. Tunnel under construction is situated within Iranian plate. Iran appears in Alpine quake zone (Mediterranean, Turkey, Iran, North India) where 5 to 6% earthquakes occur. Area seismicity is rated by intensity of from time to time replicated earthquakes.

Characteristics of earthquakes occurred in Taloun and Alborz tunnels constructional area listed in Table 2. Active tectonic faults and earthquake epicenters are shown in Figure 4.

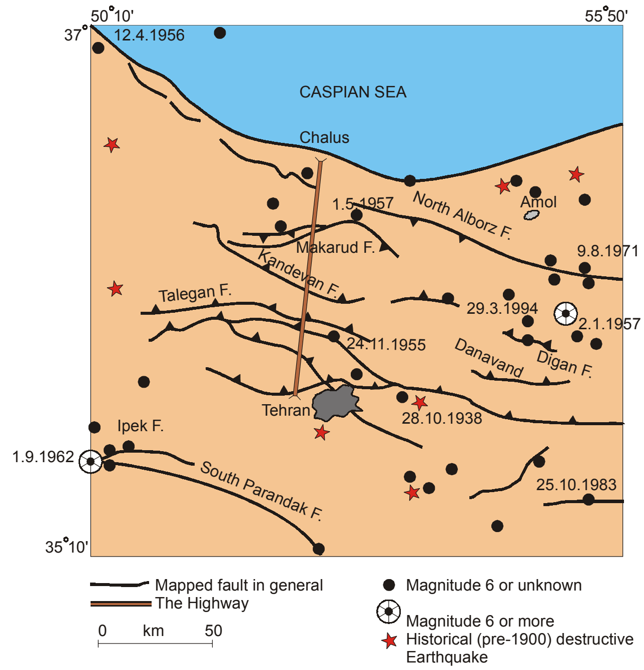


Fig. 4. Disposition of active tectonic fractures and earthquake epicenters.

Table 3. Earthquakes in tunnel site (Figure 4)

Date	Epicenter	Magnitude	Depth
1930 October 22	Mobarak-Abad, VI MM	-	-
1955 November 24	Mosha fault	-	-
1957 July 2	36˚ 14 n.l.	7.2	15 km
1959 May 1	North of Kandovan	5.5	33 km
1971 May 9	Babol-Kanar	5.5	27 km
1983 October 25	Garmsar, V-V I MM	4.6	-
1984 March 29	Kharaz-Larizhan	5.5	33 km
1988-1989	Tehran	4.5	

Data above show that the severest earthquakes happened at Iranian plate edges: northwest, east and south of Iran. Quakes in constructional area cannot be ranked destructive except two events with 7,2 (1957) and 7,5 (1962) of magnitude.

It is known that earthquake intensity depends on magnitude and depth of focus. According that the earthquake in 1957 with magnitude 7,2 and focus depth of 15 km (Table 2) may be considered destructive of MSK-64 scale IX. Epicenters of those only earthquakes are at least 100 km far from tunnels.

In addition, it is noteworthy that MM and MSK-64 scales are very similar (both are 12-score), they to some extent differ in earthquake affect on rock mass evaluation (Gere and Shah 1984, Eiby 1978, Nikonov 1984). It is easy to verify that earthquakes of intensity IX and VIII on MM scale correspond to those of VIII and VII on MSK-64 scale by *rock* mass affecting.

So, stating on above, seismicity of Taloun and Alborz tunnels constructional area may be assigned (with a safety margin) to the VIII intensity on MSK-64 scale.

Tunnel lining design schemes under long arbitrary directed seismic waves are shown in Figure 5.

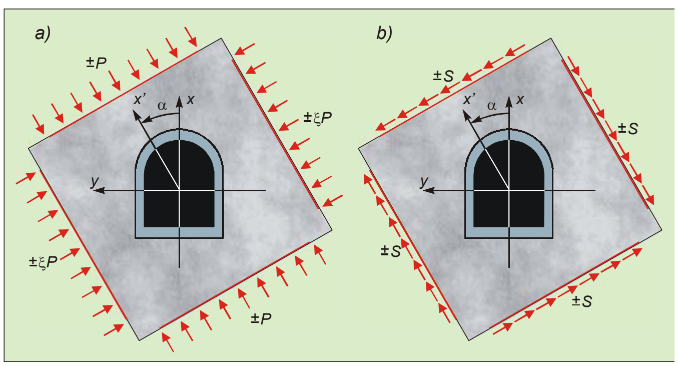


Fig. 5. Design schemes of tunnel lining subjected long arbitrary directed seismic waves: a) longitudinal; b) transverse

Tunnel lining is subjected to longitudinal P (a) and shear S (b) seismic waves generated in hypocenter of the earthquake.

Rock mass seismic extremes under these waves can be determined by formulae according to standards:

$$P = \pm\frac{1}{2\pi} AK_1 \gamma C_P T_0;\ \xi = \frac{\nu_0}{1-\nu_0};\ S = \frac{1}{2\pi} AK_1 \gamma C_S T_0, \quad (3)$$

where A –an earthquake intensity factor by MSK-64, determined by Russian Standard; K –destruction allowance factor; C_p, C_s – velocities of P and S seismic waves propagation correspondingly, m/s; T_0 –dominant rock particles oscillation period ($T_0 = 0.5$ s if data is missing).

As a result of a tunnel lining calculation the maximal compressive and tensile normal tangential stresses on the lining cross-section internal outline and corresponding them stresses on the external lining outline under any direction (dependent on the α angle) of simultaneous P and S seismic waves are assessed.

TUNNEL LINING DESIGN

Main principles.

The design model and alternatives of in situ stress field of the rock mass are shown in Figure 1. All the variants present as equiprobable, therefore each of them has been used in calculations and the worst case has been adopted for the design.

The cross section of the tunnel lining of main tunnels: the west and east ones is shown in Figure 6. Monolithic concrete lining of 40 cm thick was picked.

Analytical design method (Fotieva 1980) of the tunnel lining has been used. The method uses for a non-circular cross-section tunnel lining mathematical manipulations (the conformal mapping method) as a result of which insignificant shape alterations of the cross-section take place. The inner and outer outlines of the lining cross sectional limits with calculation points marked are shown in Figure 7*a*. Coordinates of the points after conformal mapping are listed in Table 3. Calculation of normal tangential stresses has performed in those points of the tunnel lining. Internal forces namely bending moments and longitudinal forces have been calculated in superimposes determining sections onto design cross-section shown in Figure 7b. The most part of computed lining outline (Figure 7b) coincides with the design one (Figure 7a) only a right angle rounds in points 8-10 (8'10'). It needs to be noticed that only the permanent concrete lining has been calculated. Temporary supports have not been taken into account that might consider as reserve of bearing capacity of the permanent lining.

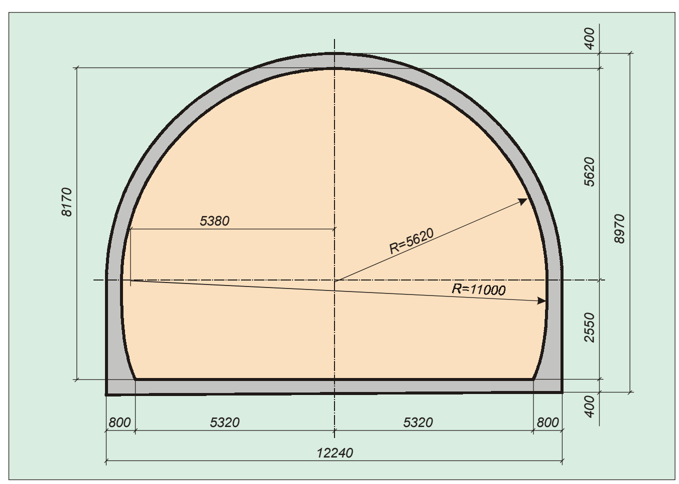
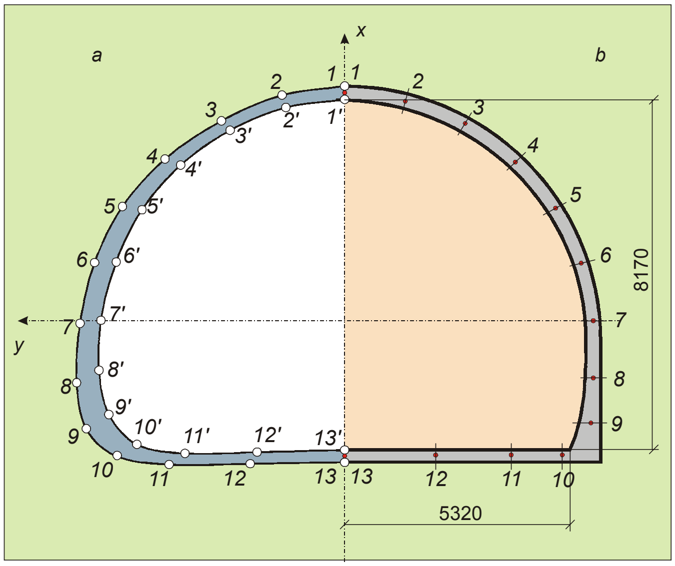


Fig. 6. Cross section of the tunnel lining of the main tunnel of 40 cm thickness



a) b)

Fig. 7. Calculated inner and outer outlines of the lining cross section with the points layout for lining stress state determination (a); layout of cross-sections for internal forces determination (b).

Table 3. Point coordinates of 0.4 m thick lining's cross-sectional outlines and centerline

Point numbers	Outlines point coordinates, m					
	inner		outer		centerline	
	x	y	x	y	x	y
1 (1')	5.22	0.00	5.64	0.00	5.43	0.00
2 (2')	5.03	1.37	5.41	1.48	5.22	1.42
3 (3')	4.50	2.67	4.78	2.93	4.64	2.80
4 (4')	3.69	3.82	3.85	4.23	3.77	4.02
5 (5')	2.65	4.72	2.67	5.24	2.67	4.98
6 (6')	1.43	5.32	1.29	5.90	1.36	5.61
7 (7')	0.11	5.63	-0.24	6.30	-0.06	5.96
8 (8')	-1.16	5.67	-1.66	6.46	-1.41	6.07
9 (9')	-2.20	5.44	-2.70	6.30	-2.45	5.87
10 (10')	-2.82	4.84	-3.25	5.62	-3.04	5.23
11 (11')	-3.03	3.72	-3.40	4.27	-3.22	3.99
12 (12')	-3.00	2.04	-3.37	2.32	-3.18	2.18
13 (13')	-2.95	0.00	-3.33	0.00	-3.14	0.00

Tunnel lining design in tuffs.

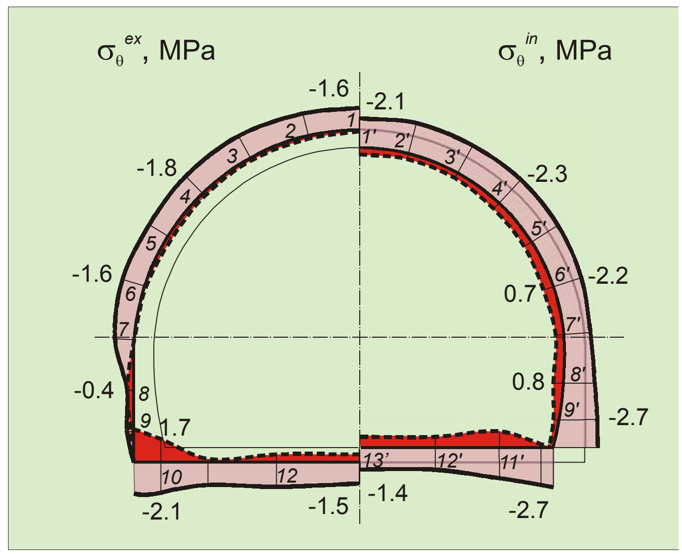
This chapter includes the design of the tunnel lining in tuffs at the depth of 800 m. The tunnel lining of the 40 cm thickness see (Figure 6) has been considered.

Seismic effects analysis. Tunnel lining analysis results upon MSK-64 scale intensity VIII earthquake effect are presented in Figure 8. Remember that the analysis is resulted in the determination of:

- probable maximal compressive (negative) normal tangential stresses in the points of the inner lining cross-section outline and corresponding them stresses along the outer outline and internal forces;
- probable maximal tensile (positive) normal tangential stresses in the points of the inner lining section outline and corresponding them stresses along the outer outline and internal forces;

at any combination and direction of P and S seismic waves obtained in accordance with the design schemes (see Figure 5).

Possible extreme normal tangential stresses that may appear during an earthquake (VIII by MSK-64) are shown in Figure 8. The corresponding internal forces: bending moments (*M*) and longitudinal forces (*N*) are shown in Figure 9.

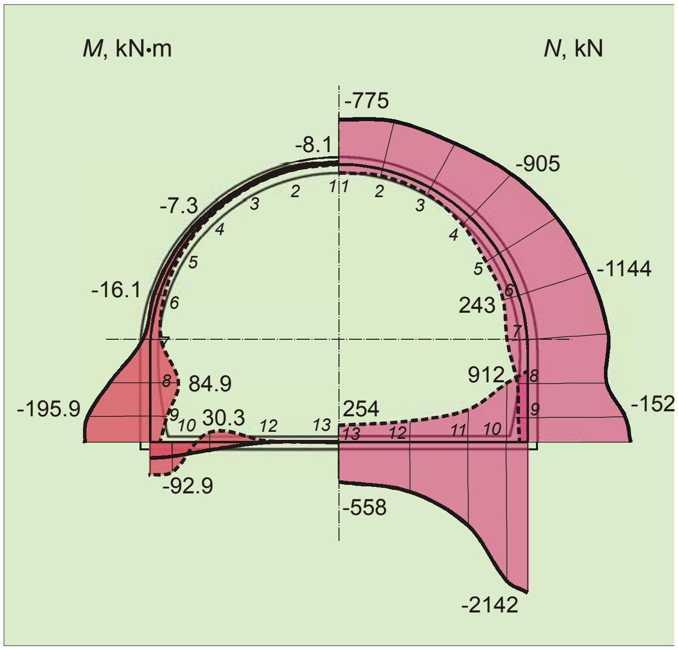


a) b)

Fig. 8. Contours of possible extreme stresses σ_θ^{in} that may appear on the inner outline of the lining located in tuffs during the earthquake and corresponding them stresses σ_θ^{ex} on the outer lining outline

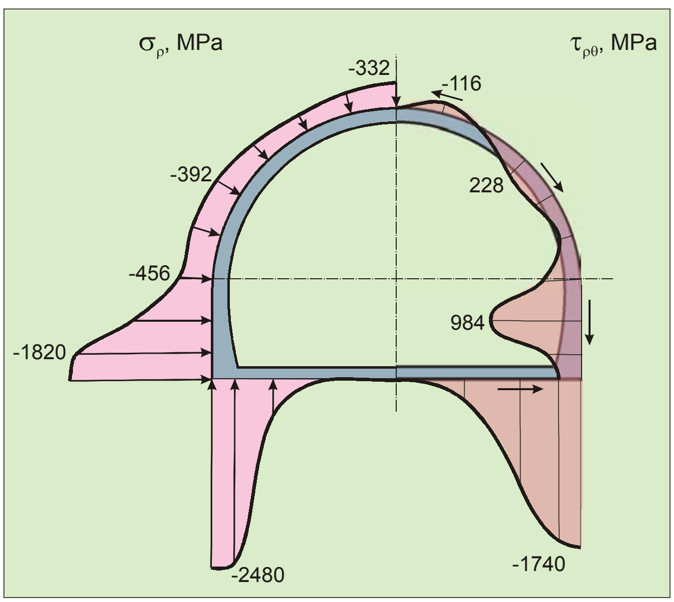
Static loads analysis. Tunnel lining design has performed for rock mass enclosing tuff layers (see Table 1). Calculated contact stresses: normal σ_ρ and shear $\tau_{\rho\theta}$ once that occurred on the interface of the lining and rock mass are presented for two variants of the in situ stresses (see Figure 1). Contours of the contact stresses at lateral pressure factor $\lambda = 1.0$ are represented in Figure 10.

The results of the tunnel lining design at the depth of 800 m in the rock mass with lateral pressure coefficient of in situ stress state $\lambda = 1.5$ are shown in Figures 11, 12 where normal tangential stresses σ_θ (MPa) on inner and outer lining outlines as well



a) *b)*

Fig. 9. Contours of bending moments M and longitudinal forces N corresponding the extreme stresses σ_θ^{in} .



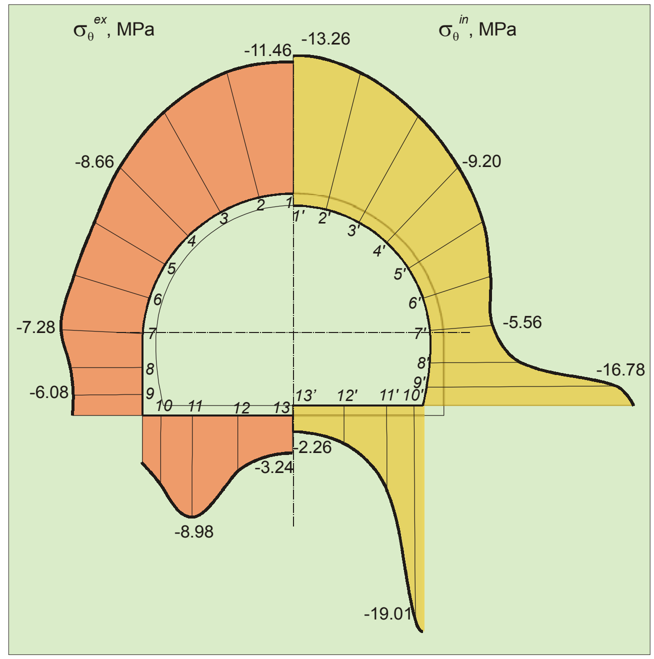
a) *b)*

Fig. 10. Contours of the contact (lining - rock mass) stresses: normal σ_ρ *(a) and shear* $\tau_{\rho\theta}$ *(b) namely at the 800 m depth and the lateral pressure factor of in situ rock mass stress state λ = 1.0*

as internal forces – bending moments *M* (kN·m) and longitudinal forces *N* (kN) are given.

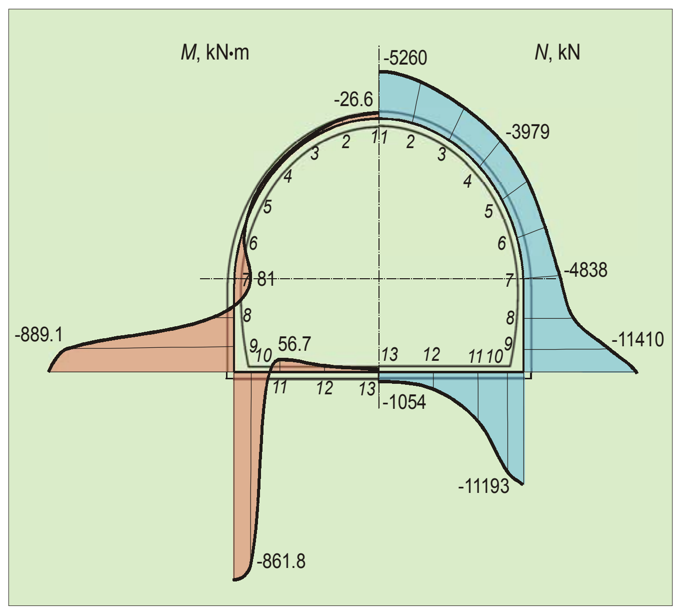
Tunnel lining strength assessment

Results obtained from tunnel lining static calculations prove that in two studied cases of the intact rock mass stress state the lining is subjected to compressive stress only and the stress magnitudes are less than the design (allowable) concrete resistance. So, designed lining from monolithic concrete of B30 class and 40 cm thickness can take static loads with a reserve of strength of strength.



a) *b)*

Fig. 11. Normal tangential stresses σ_θ^{ex} *(a)* σ_θ^{in} *(b) on outer and inner outlines of the lining located at the 800 m depth in tuffs with the lateral pressure factor of in situ rock mass stress state λ = 1.5.*



a) *b)*

Fig. 11. Contours of bending moments M and longitudinal forces N in the lining sections at the 800 m depth and the lateral pressure factor of in situ rock mass stress state λ = 1.5.

At the same time calculations revealed that the tunnel lining may suffer both additional compressive stresses and significant (comparable with allowable concrete resistance) tensile stresses under seismic effects of variously directed and combined seismic waves.

Calculated (maximal by magnitude) stresses in critical points of inner and outer tunnel lining outlines under seismic effects and average compressive strength (taking into account creep and stress relaxation of concrete during durable loading under static loads) at the depth 800 m are given in Table 4.

Table 4. Extreme possible seismic stresses in critical points of tunnel lining's (*outer / inner*) outlines and average static one (MPa)

Point numbers	Seismic		Static (*compression*)	
	tension	compression	H = 700m	H = 800m
8' (8)	0 / 0.8	- (0.4 / 2.2)	-6.9	-7.9
9' (9)	0 / 0.5	- (0.4 / 2.7)	-10.0	-11.4
10'(10)	1.7 / 0.3	- (2.1 / 2.7)	-11.0	-12.5
11'(11)	0.3 / 1.2	- (1.6 /2.2)	-7.2	-8.2
12'(12)	0.6 / 0.8	- (1.7 / 1.6)	-3.6	-4.1

As to tensile stresses, it should be taken into account that at the depth of 800 m the seismic tensile stresses are completely balanced out by the opposite lining compression under static loading.

It may be concluded from the results obtained that the strength of the monolithic lining from concrete of B30 class and 40 cm thickness at the depth 700 and 800 m is provided under static and seismic loading.

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