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ATYPICAL SOLUTION OF A ROAD TUNNEL VALÍK

1. Introduction

The tunnel Valík forms a significant part of the route regarding a by-pass road of the city Plzeň on highway D5 from Prague to Rozvadov.

The tunnel is designed with two tunnel tubing with net sections of approx. 130 m^2 meeting in a central reinforced concrete column located in the area of the central tunnel (Fig. 1); this is an atypical solution in our country which significantly decreases the width of the tunnel complex and therefore also limits the width of the dip basin on the surface. This requirement of opponents regarding a tunnel solution of the by-pass road was a reason for the acceptance of this conception that, on the other hand, increases acquisition costs of the tunnel. Each tunnel tube has an oval shape with a counter-arching. The max. width of the vacant space within the spherical cap is approx. 16 m; the overall height of the vacant place is approx. 11.7 m.
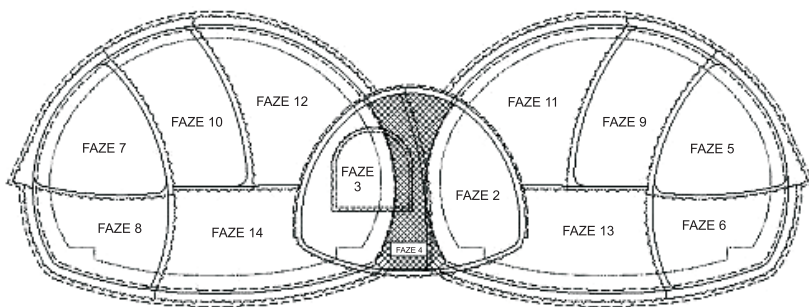

Fig. 1. Phase of tunnel construction

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There was a survey tunnel holed in the right tunnel in 2002–2003 for the purposes of getting specifications for engineering and geological relations and based on the detailed evaluation's results, the rock mass was grouped into the 5a technological class NRTM. This decision resulted in a proposal regarding the fragmentation of a vacant place in both tubing (Fig. 1) and a proposal to construct the column in the so called central tunnel holed within the 1st stage as a temporary piece forming a part of further construction advancing.

The static and stability solution of the tunnel is in very unfavorable underground conditions which had to deal in detail with the static problems of the column regarding the stable status of its upper layer and bottom layer incl. a proposal of stabilization measures implemented during the tunnel holing. It was obvious that holing by means of a combination of vertical and horizontal fragmentation of the tunnel profile would cause different loading conditions in the column; the column dimensions must be adjusted in accordance to such changes (see Fig. 4).

2. Proposal regarding the column construction

The central inter-tunnel column was designed in accordance to Figure 2 as monolithic reinforced concrete construction from the C 30/27-XF3 material, reinforced by self-supporting welded spatial armature φ 20 mm, construed in regards to the status and division of the load within the column (Fig. 3).
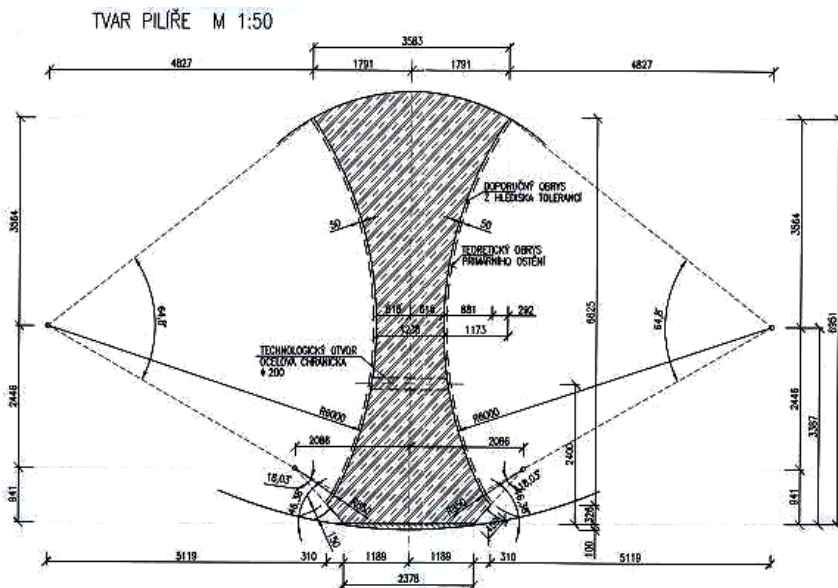

Fig. 2. Shape of the column

In accordance to the results of the preliminary static and stability solutions, a system for assurance of the upper and bottom layers stability was proposed; the stability of the upper and bottom layers in an approved tunnel construction system solved by means of using micropilots in the bottom layer of the column and cross anchoring in the area above the column up to approx. 6 m the reason this conception was approved is the practical non-injectability of the underground environment.

The solutions for individual technologic classes of NRTM show that the reliability and stability of the central column is provided in all loading conditions.

The least favorable loading of the column resulted from variant 5a l/16 m in the phase 8 (Fig. 1) where the column reinforcement was done by means of allowable loading stress. For purposes of evaluation of the column's bearing capacity through the boundary condition (non-reinforced column), the approximate values of internal forces in the least favorable section D–D′ (Fig. 3) could have been used.
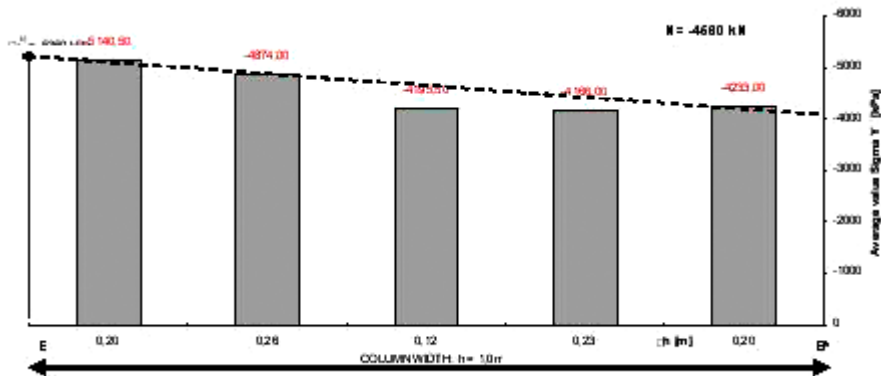

Fig. 3. Cross-section of the column D–D*, normal stresses σ_y

By evaluating the behavior of normal stresses σ_y in the cross-section D–D′, the edge pressure in the following values can be determined:

$\sigma_1 = -5350$ kPa,

$\sigma_2 = -4230$ kPa.

The following normal forces N and shear forces T were determined by a numeric integration:

$N = -4560$ kN,

$T = 133$ kN.

Because a division of stresses within the cross-section corresponds to linear assumption, it is possible to determine for

$$W = \frac{1}{6} \cdot 1 \cdot 1^3 = 0.166 \text{ m}^3$$

size of the bending moment in the cross-section D–D' (central part of the column).

$$M = 0.166 \left(5350 - \frac{5350 + 4230}{2} \right) = 93 \text{ kN} \cdot \text{m}.$$

Eccentricity of normal force

$$e = \frac{93}{4560} = 0.021 \text{ m}$$

for concrete C16/20 (B20) (R_{bd} =11.5 MPa, R_{btd} = 0.9 MPa) there will be a normal force at the failure limit

$$N_{\acute{u}} = 0.8 \{ 11\,500 \cdot (0.5 - 0.021) \cdot 2 \} = 8813 \text{ kN} \gg 4560 \text{ — the column meets the needs.}$$

Evaluation regarding a damage caused by shear force:

$$Q_d = \text{T} = 133 \text{ kN},$$

$$Q_{bn} = \frac{1}{3} \cdot 1 \cdot 1 \cdot 1.25 \cdot 0.8 \cdot 900 = 300 \text{ kN} > 133 \text{ kN — the column meets the needs.}$$

The division of vertical stresses in the cross-sections (E–E′; C–C′) passing through the top and bottom base line (Fig. 4) is, from the viewpoint of stress, significantly more favorable (max. σ_y = 3.8 MPa).

Regarding the evaluation in accordance to an ultimate state, it would be possible to design the column reinforcement with regards to minimum reinforcement degrees according to ČSN 73 1201 and to construct the central column from concrete B20 (C16/20) that additionally complies better with requirements regarding the minimization of hydrating heat formation.

The maximum possible value of the column stressing after an overall collapse of the upper layer can be

$$p_{max} = \gamma \cdot h \cdot 2b = 25 \cdot 16 \cdot 2 \cdot 8 = 6400 \text{ kN}$$

produced stress intensity at column width of 1.00 m

$$\sigma_{max} = \frac{6400}{1.0} = 6400 \text{ kPa} = 6.4 \text{ MPa} < 11 \cdot 5 \cdot 0.8 = 9.2 \text{ MPa} - \text{the column also meets the}$$

needs of this practically unreal loading condition.

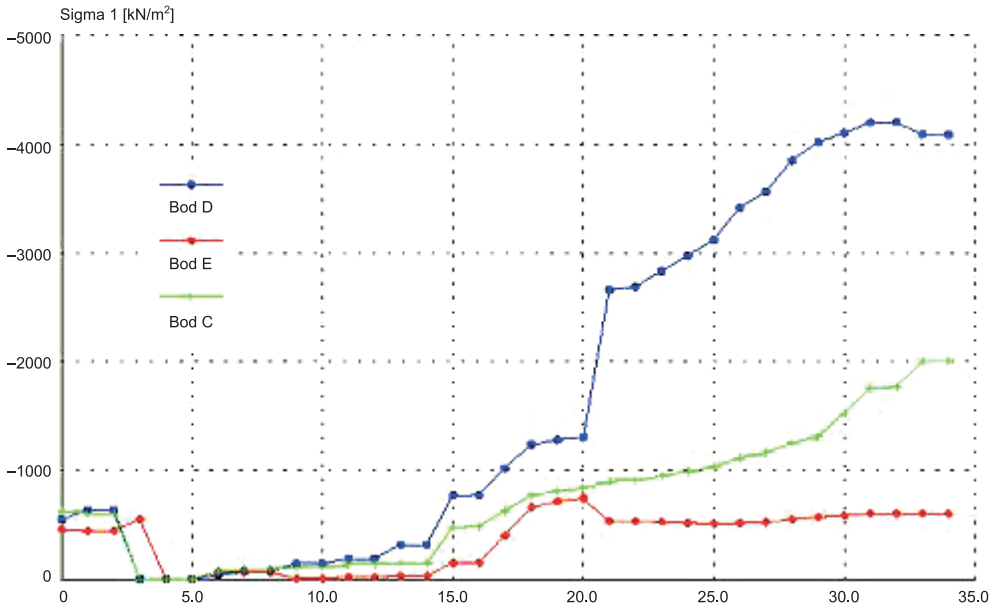Sigma 1 [kN/m²]

Bod D

Bod E

Bod C

Fig. 4. Principal stress in the reinforced concrete column

The extent of this load is higher than the value determined by the calculation — value of σ_{1max} = 5.25 MPa. The results indicate that there is no loading from the entire height of the upper layer transferred onto the column; it is also confirmed by a stability solution without an anchoring impact indicating a formation of a bearing arch in the upper layer of the tunnel. Also the absence of shear surfaces in the upper layer indicates particular self-supporting in the upper layer of the tunnel.

The vertical decrease of the column achieves a max. size of approx. 8–10 mm and is significantly limited by the application of micropilot armatures in the bottom layer applied in the TT 5al; their loading is implied from Figure 5. The stated shift values do not negatively influence the loading of lining or progressiveness of development regarding contamination in the tunnel area.

The stability solution regarding a vacant place, that is a determination of contamination areas, was evaluated for each excavating and reinforcing phase in all NRTM technology classes of the soil.
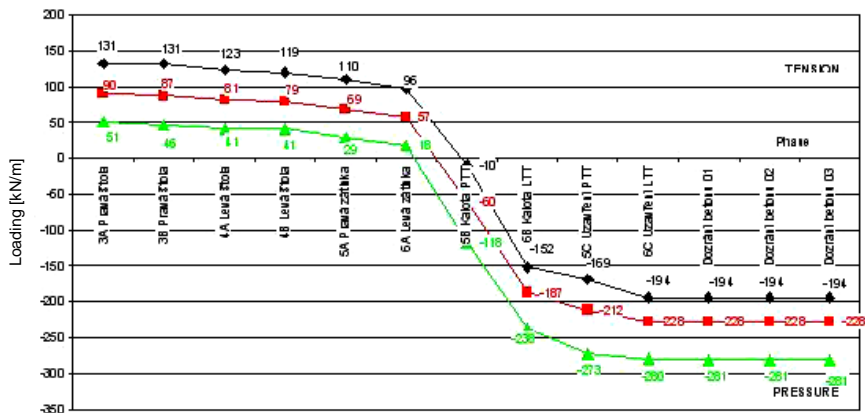
Fig. 5. Loading of micropilots under the central reinforced concrete column

The results showed that there is no basic difference in the character and extent of contamination areas in relation to the technology classes and the main contamination areas are:

— areas of counter-arching transition to points of the central tunnel, to depth of approx. 1 m,
— areas of the central column bottom layer in phases No. 10–14; stability is also provided by a system of micropilots.

The results proved that real loading of the column is least influenced by asymmetric loading conditions and the bearing capacity of the column is also provided for a minimum degree of its reinforcement.

REFERENCES

[1] *Aldorf J., Ďuriš L.*: Statické řešení primárního ostění pilíře středního tunelu a stabilitní řešení tunelu Valík (Static solution of primary lining of the central tunnel column and stability solution regarding the tunnel Valík). Solution for RDS 02/2004