CALCULATION OF CONCRETE TUNNEL LININGS, INTERACTION DIAGRAM, STRENGTH AND DEFORMATION MODULUS OF CONCRETE.

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ABSTRACT: Calculations of tunnel linings can be performed in many ways. The calculation that best captures the actual stress and deformation of the lining is only known after construction begins and the lining/rock mass system is monitored. Every construction project must meet two fundamental contradictory conditions. The structure must be economical and at the same time reliable. During loading, the structure continuously passes through states of stress and strain, which we can express mathematically based on knowledge of structural mechanics. The reliability of a building structure is defined by three subsystems: The load-bearing structure of the tunnel, the loads acting on the structure, the environment surrounding the structure, whose deformation characteristics are represented in a homogeneous isotropic medium by the modulus of elasticity ***E,*** Poisson's ratio ***ν***, and the shear modulus ***G***. The selection of the correct calculation method is necessary to determine accurate costs for construction work.

1. INTRODUCTION

Static calculation determines the load-bearing capacity of tunnel construction with the participation of the environment surrounding its structure. The aim of comparing completely different calculation systems is to establish differences in outputs while maintaining the same input parameters for the load-bearing structure, loading, and the environment surrounding the structure. We will compare calculation systems for tunnel linings that are not time-consuming and are commonly available to the technical public. For testing, a circular metro tunnel with an inner diameter of 5.2m, with a lining made of sprayed concrete, was selected. This geometric shape allows for relatively simple mathematical definition of the contact problem "rock-lining". Determination of the ultimate load-bearing capacity of the lining and subsequently the ultimate deformation in the first 28 days is possible only with knowledge of the increase in strength and deformation modulus of concrete during these days. Data on concrete strength and deformation modulus were collected over a very long period. For the strength of sprayed concrete, two regression curves are fitted; for the deformation modulus, one regression curve is fitted.

2. LOAD AND BEARING CAPACITY FUNCTIONS OF TUNNEL LINING p_L, p_U

The calculation of static quantities of tunnel lining using the finite element method (FEM) assumes of equilibrium and compatibility of deformations at common nodes of elements. The relationship between vectors of deformations and forces at element nodes is determined by the element stiffness matrix.

$$\{X\} = [k]\{u\} \qquad [1]$$

where $\{X\}$ is the vector of forces $\{u\}$ is the vector of deformations $[k]$ is the element stiffness matrix

By FEM calculation we obtain the stress state and deformations of the rock mass and tunnel lining as well as the loading of the tunnel lining p_{Li} at the contact between the rock and the tunnel lining. Concrete tunnel lining and the surrounding rock have rheological properties, and the loading is therefore a function of time (Zapletal, A.; Bucek, M. and Barták 1992)

$$pL = f\{\rho B(t), \rho R(t), \gamma, r, R) \qquad [2]$$

where $\rho_{B(t)}$ are the rheological properties of concrete,

$\rho_{R(t)}$ are the rheological properties of rock

γ is the unit weight of rock

r is the radius of the circular excavation

R is the radius of the surrounding rock ring

For a specific tunnel lining, there exists a limit load of the tunnel lining $p_{Lmax(t)}$ and a limit bearing capacity of the tunnel lining $p_{Umax(t)}$, which is usually achieved at only one point of the lining. We can simplify this relationship by plotting it in the following graph.

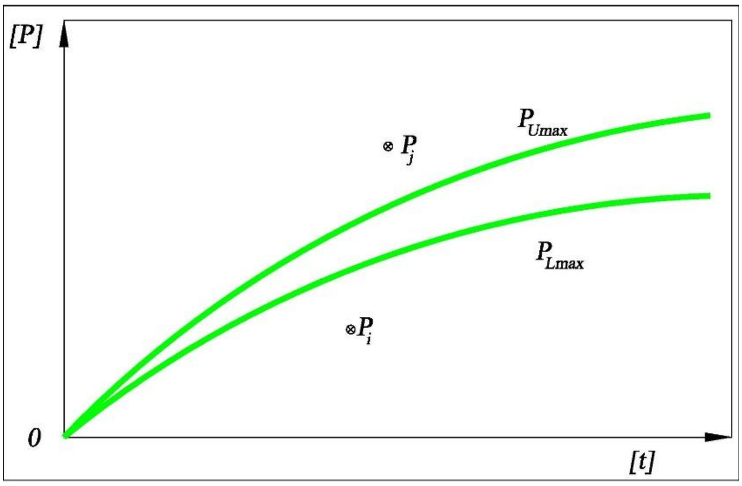

Figure 1. Limit bearing capacity and limit loading of tunnel lining

The consequence of the ambiguity of the load-bearing capacity function is the ambiguity of the deformation limit function. Deformations that in one case cause lining destruction, in another case induce stress that does not nearly exhaust the concrete strength. Under these circumstances, it is necessary to establish criteria according to which – regardless of the multivalence – it would be possible to safely assess the state of deformation.

For an arbitrary point of the tunnel lining, we calculate using FEM the static quantities M_i, N_i, T_i, which correspond to the loading $p_{Li(t)}$, thus

$$p_{Li(t)} = f\,(M_i, N_i, T_i) \qquad [3a]$$

The ultimate load-bearing capacity of tunnel lining is dependent on the cross-sectional dimensions, the degree of reinforcement of the cross-section, the quality of concrete and reinforcement, and simultaneously on the quantities M_{Ui}, N_{Ui}, T_{Ui}.

$$p_{Umax(t)} = f\,(A_{bi}, A_{si}, R_b, R_s) = \quad f\,(M_{Ui}, N_{Ui}, T_{Ui}) \quad [3b]$$

where A_{bi} is the area of effective concrete

A_{si} is the area of effective reinforcement

R_b is the strength of concrete

R_s is the strength of reinforcement

By comparing the values of load $p_{Li(t)}$ and load-bearing capacity $p_{Umax(t)}$, we then evaluate the load-bearing capacity of the tunnel lining. If the load $p_{Li(t)}$ is less than $p_{Umax(t)}$

$$p_{Li(t)} < p_{Umax(t)} \qquad [4]$$

then we conclude that the tunnel lining will satisfy the ultimate limit state.

If the load $p_{Li(t)}$ is greater than $p_{Umax(t)}$

$$p_{Li(t)} > p_{Umax(t)} \qquad [5]$$

then we establish that the limit of strength is exceeded and the structure does not meet the requirements at the ultimate limit state.

We determine the critical location of the tunnel lining capacity by calculating the values si around the entire perimeter of the lining. Where the value s_i is less than s = 1, the structure does not meet the requirements at the ultimate limit state.

In general, this relationship is expressed by the equation

$$\frac{p_{Umax(t)}}{p_{Li(t)}} = s \qquad [6]$$

This assessment, however, usually needs to be supplemented by an assessment of the serviceability limit state (deformation) and the limit state of cracking.

3. EVALUATION OF TUNNEL LINING BEARING CAPACITY USING INTERACTION DIAGRAM

The assessment of load-bearing capacity of concrete tunnel linings is performed using an Interaction Diagram. Assuming full utilization of the load-bearing capacity of concrete and reinforcement at the cross-section failure limit, we calculate the corresponding bending moment value M_u for the normal force N_u. By plotting the values of N_u and M_u on a graph, we obtain a line representing the cross-section failure limit, which we call the "Interaction Diagram of Cross-Section Failure Limit". This line corresponds to the theoretical full utilization of the load-bearing capacity of concrete and steel (Zapletal, A.; Bucek, M. and Barták 1992).

For each specific point of the tunnel lining, the FEM calculates the values M_i, N_i, T_i, and the corresponding values M_{ui}, N_{ui} are determined from the Interaction Diagram.

$$\frac{N_{ui}}{N_i} = s_i\,, \quad \frac{N_{uj}}{N_j} = s_j \qquad [7]$$

Assessment of the safety of the proposed structure according to equation [6] using the Interaction diagram. The magnitudes of the values s_i and s_j determine the safety of the support, and if the values s_i, s_j differ significantly from s = 1, it is recommended to modify the original support design and perform the calculation again.

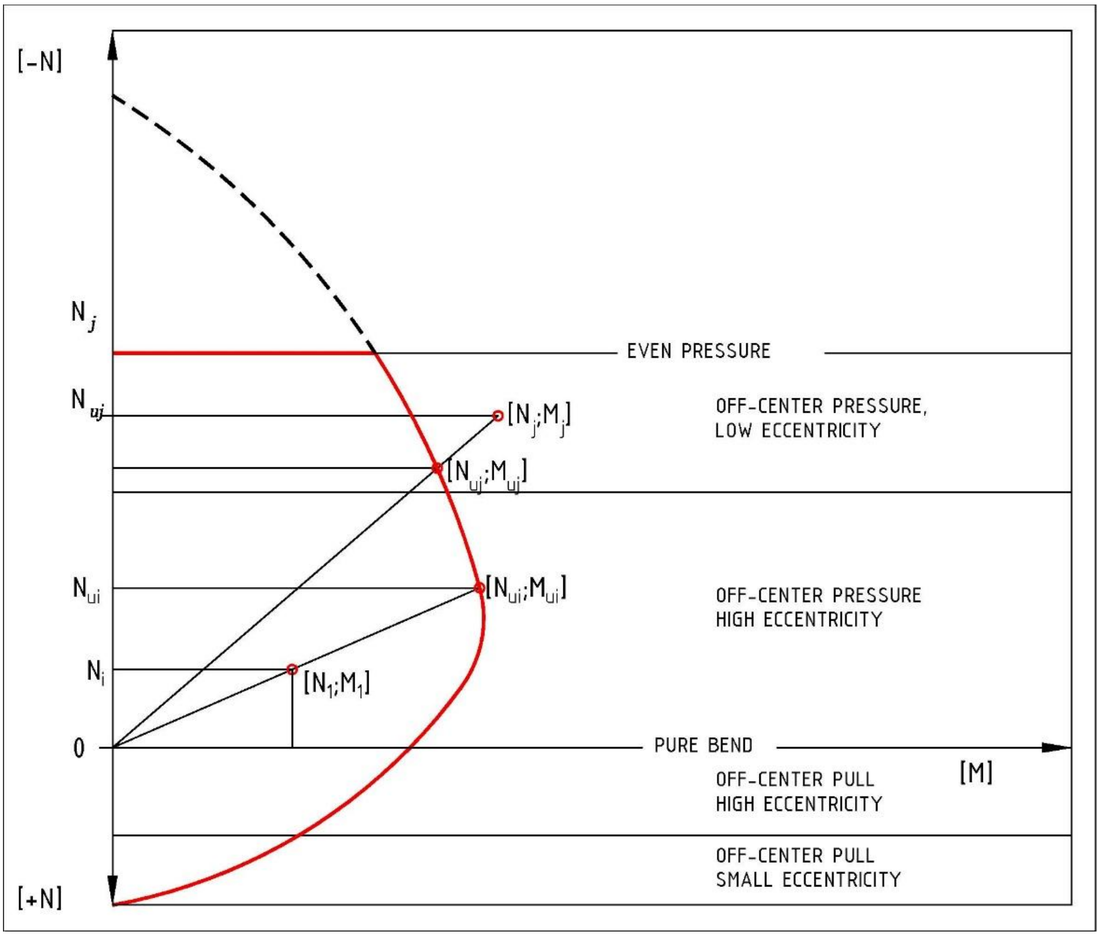

Figure 2. Interaction diagram

$$\lambda = \frac{l_e}{i} \qquad [8]$$

When designing reinforced concrete cross-sections, it is necessary to determine the position and area of reinforcement in such a way as to ensure its full utilization (deformation at the limit of cross-section failure). This condition determines what percentage of the reinforcement area must be sufficiently distant from the neutral axis of the cross-section. A typical economic condition is that a maximum of 25% of the total reinforcement area should be in the middle part of the cross-section (0.5h). When evaluating the load-bearing capacity of tunnel lining, special attention must be paid to concrete lining in segmented excavation technology, which is not supported by the rock mass. Lining placed freely in the profile is classified according to slenderness into thick, medium-slender, and very slender, evaluated by the coefficient λ.

Where l_e is the effective length of the member

i is the radius of gyration of the concrete cross-section

Thick members $\lambda < 20$

Medium-slender members $\lambda < 150$

Very slender members $\lambda > 150$

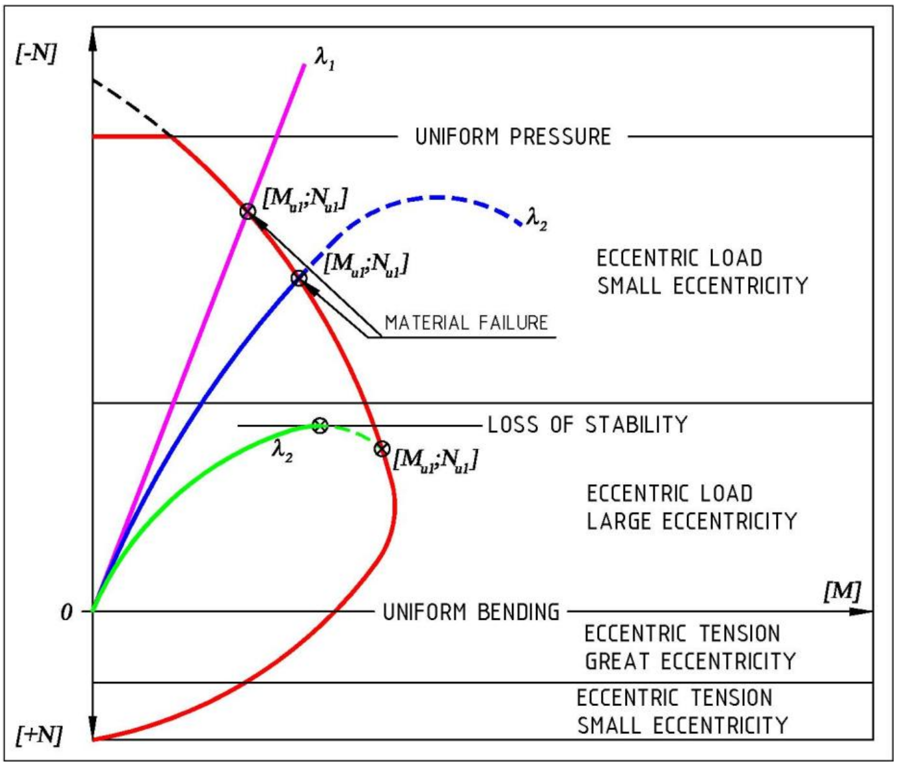

Figure 3. Effect of bar slenderness on load-bearing capacity

For thick and moderately slender rods, it is a strength problem of load-bearing capacity, while for very slender rods it is a stability problem of load-bearing capacity.

4. DESCRIPTION OF COMPUTATIONAL SYSTEMS

4.1 ANALYTICAL COMPUTATIONAL SYSTEM

The mutual interaction between the lining of circular profile and rock mass is modelled as full contact of the lining at radius R_1 with the surrounding elastic medium (Bulyčev 1982). The calculation of normal and shear stress (p,q) at the contact of the lining with the soil is determined from the boundary conditions of displacement of points written into the equations

$$u_k = u(\gamma H) - u_0 = \alpha \times u(\gamma H) = u(\alpha^* \times \gamma H);$$

$$v_k = v(\gamma H) - v_0 = \alpha \times v(\gamma H) = v(\alpha^* \times \gamma H); \qquad [9]$$

where u_0; v_0 are radial and tangential initial displacements of the lining perimeter points before lining installation, $u(\gamma H)$; $v(\gamma H)$ are displacements of rock mass points at the contact with the lining and u_k; v_k are displacements of the outer perimeter points of the lining at the contact with the rock. The coefficient expresses the increase in deformations of the rock mass between excavation and insertion of the fully activated lining (includes rheological properties of the lining and rock, effect of anchoring, etc.) The depth of tunnel installation is denoted H, and the unit weight of soil is denoted.

As a result of the interaction between the lining and the rock, normal forces p and tangential forces q arise at the contact (lining loading Fig. 4) and additional stresses on the excavation perimeter

$$\sigma_r^{(1)} = \sigma_r^{(0)} - p; \ \tau_{r\theta}^{(1)} = \tau_{r\theta}^{(0)} - q\,. \qquad [10]$$

Under the influence of these stresses, deformations arise as described in equation {10}. The solution of these equations assumes the placement of a circular tunnel at a sufficiently great depth, and the stresses in the vicinity of the excavation are calculated using a plane problem of elasticity theory in a half-space, considering the weight of the medium surrounding the excavation. The boundary conditions on the perimeter of the lining are written in the form

$$u_{\Pi}+iv_{\Pi} = u_k+iv_k$$

$$i \int (\sigma_r + i\tau_{r\theta})_{\Pi}\, ds = i \int (p + iq)_k\, ds \quad on\ L \qquad [11]$$

$$i \int (\sigma_r + i\tau_{r\theta})_k\, ds = 0 \qquad on\ L_1$$

Using the Kolosov-Muschelišvili method, we express the boundary conditions on the perimeter of the lining L and L_1 using complex potentials $\varphi(z)$ and $\psi(z)$ characterizing the stresses in the rock and complex potentials $\varphi_1(z)$ and $\psi_1(z)$ characterizing the stresses in the lining (Muskhelišvili 1966).

Let t be a point on the circle L and t_1 be a point on the circle L_1, then

$$t=R_1\, e^{i\theta}\ t_1=R_o\, e^{i\theta} \qquad [12]$$

and it is possible to prove that for the circle L holds

$$i \int \left(\sigma_r^{(0)} + i\tau_{r\theta}^{(0)}\right) ds = -\alpha^* \gamma\, H \left(\frac{1+\lambda}{2}\, t - \frac{1-\lambda}{2}\, \bar{t}\right) \quad [13]$$

Using substitution and solving by integration, we obtain equations for

$$p = p_o + p_2 \cos 2\theta;\ \ q = q_2 \sin 2\theta \qquad [14]$$

where the expressions for p_0, p_2 and for q, q_2 are expressed using special coefficients (Bulyčev 1982). In the next step we determine $\sigma_{\theta in}$ and $\sigma_{\theta ex}$ from which we can then simply determine the internal forces in the lining according to the equations

$$M = \frac{bt^2(\sigma_{\theta in} - \sigma_{\theta ex})}{12}; \qquad N = \frac{bt(\sigma_{\theta in} + \sigma_{\theta ex})}{2}; \ [15]$$

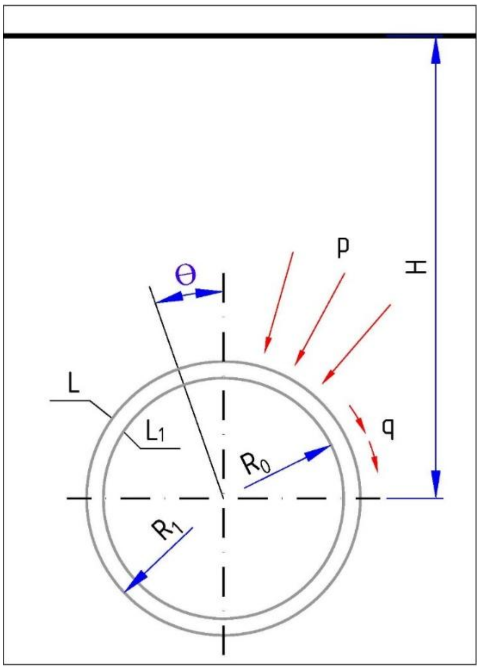

Figure 4. Computational scheme

5. POLYGONAL METHOD

This computational method is universal, capable of capturing any shape of structure and allows easy modification of input calculation parameters (loads, geotechnical parameters of the rock environment). However, the calculation only informs us about deformations and internal forces in the lining and tells us nothing about the stress state of the rock mass.

Computational programs designed for calculating tunnel lining using the polygon method use either the force method or the deformation method (Aldorf 1992). The exact shape of the tunnel lining is replaced by a polygon in the calculation. The rock mass is modelled in the calculation as a system of hinged bars connected to the lining at the vertices of the polygon. The action of the rock mass on the lining is modelled by external active loads acting at the polygon vertex and the passive resistance of the rock is modelled by Winkler springs. The calculation proceeds in iterative cycles and in the first iterative cycle all hinged bars modelling the rock are in operation. After the first iterative cycle, tensile bars are excluded and the calculation continues until all tensile bars are excluded and at the same time all compressed hinged bars are in operation. Hinged bars simulating the rock are introduced into the calculation with unit area and length and their stiffness is specified by different values of the elastic modulus E. The characteristic of elastic supports can be written in the form

$$\overline{K_i} = K^{(\sigma)} \frac{l_i + l_{i+1}}{2} b \qquad [16]$$

where $K^{(\sigma)}$ coefficient of elastic resistance of rock depends not only on the geotechnical parameters of the rock, but also on the shape of the structure and we usually determine it according to B. G. Galerkin

$$K_{(\sigma)} = \frac{E}{R(1+\nu)} \quad [17]$$

where l_i is the length of the polygon side and b is the ring width, which is usually 1m. Let us denote the deformations and rotation of the i-th node as U_i, V_i, θ_i and the internal forces of the i-th node as X_i, Y_i, M_i and external load as $\underline{X_i}$, $\underline{Y_i}$, then we introduce a column matrix $\{P_i\}$ for internal computational parameters and a column matrix $\{P_i\}$ for external load, where

$$\{P_i\} = \begin{Bmatrix} U_i \\ V_i \\ \theta_i \\ X_i \\ Y_i \\ M_i \end{Bmatrix}; \qquad \{\underline{P_i}\} = \begin{Bmatrix} 0 \\ 0 \\ 0 \\ \underline{X_i} \\ \underline{Y_i} \\ 0 \end{Bmatrix}; \quad [18]$$

We introduce support reactions as internal forces for the i-th node, and the inclusion of elastic support deformation leads to a modified coefficient $[K_i]$, which affects the computational parameters. In the general case, loading can be applied to any node (nodes have special numbering, which we denote as j). Then the matrix notation of the equation for computational parameters for node n with known boundary conditions has the following form

$$\{P_n\} = \prod_{i=n+1}^{1} [K_i] \ \{P_o\} + \sum_{j=1}^{n} \prod_{i=n+1}^{j+1} [K_i] \ \{\underline{P_i}\}; \quad [19]$$

Where $\{P_o\}$ is a column matrix of initial computational parameters (Muschelišvili 1966).

6. FINITE ELEMENT METHOD

The finite element method is an approximate numerical method which, unlike exact methods based on solving differential equations of equilibrium and compatibility of deformations, where fulfilment of equilibrium conditions and material continuity at every point of the deformed body is required, replaces the investigated infinite domain with a finite one, divided into a finite number of elements, where contact between elements is realized only at nodal points (Bulyčev 1982). The conditions of equilibrium and compatibility of deformations in the FEM are satisfied only at the common nodes of the elements. The relationship between the vectors of deformations and forces at the element nodes is determined by the element stiffness matrix. For a specific example, the element stiffness matrix is first created, then the stiffness matrix of the entire finite element system is created, and a system of linear equations is assembled (RIBTEC 1996) in the form

$$\{X\} = [k] \{u\} \qquad [20]$$

where $\{X\}$ and $\{u\}$ are vectors of forces and deformations acting at the nodes of the finite element mesh

$[k]$ is the stiffness matrix of elements of order 2M, which is assembled from terms

$$K_{ij} = \sum_{r=1}^{N} k_{ij}^{(r)} \qquad [21]$$

where $k_{ij}^{(r)}$ is an element of the stiffness matrix of the r-th element, characterizing the influence of the j-th unit deformation on the i-th term of nodal forces. M is the number of nodes and N is the number of elements in the system. By FEM calculation, we obtain the stress and strain of the rock mass and tunnel lining as well as the load on the tunnel lining p_{Li} at the contact between the rock and the tunnel lining.

6.1 CALCULATION RESULTS AND EVALUATION OF LINING LOAD-BEARING CAPACITY

For comparison of calculations, we will use a circular tunnel lining placed in sandy-clayey soil of natural moisture with unit weight γ = 18.5kN/m^3, Poisson's ratio ν = 0.32, coefficient of elastic resistance of soil k_o = 70MN/m^3, coefficient of lateral earth pressure λ = 0.7 and tunnel depth H = 10m. The tunnel lining is made of monolithic concrete C25/30 and is symmetrically reinforced on both surfaces with 48mm/m' of steel 10425. The inner diameter of the lining is d = 5.2m, the outer diameter of the lining is D = 5.92m, the deformation modulus of concrete is E_b = 2.9 × 10^{4}MPa. For calculation using the polygonal method, a vertical uniform external active load q_x = 143kPa and horizontal uniform load q_y = 90kPa are considered.

7. ANALYTICAL COMPUTING SYSTEM (ACS)

The computational scheme of the tunnel lining is modelled as a two-layer ring, the value of the coefficient α^* is considered maximum (on the safety side) regarding the shallow foundation of the tunnel α^* = 1. The modulus of elasticity is determined from equation [16] and for the analytical computational system, E_o= 273.5MPa was established.

$$K_{(o)} = \frac{E_o}{r_1(1+\nu)} \qquad [22]$$

After the corresponding calculations, we obtain these values of internal forces

Angle θ	*Internal Forces ACS*	
	Moment (kNm)	*Normal Force (kN)*
0	*-17,4*	*358*
π/4	*1,6*	*445*
π /2	*20,6*	*531*
π 3/4	*1,6*	*445*
π	*-17,4*	*358*

The course of moments, normal forces, loads, and bearing capacity coefficient is shown in the figure (Zlámal 2010)

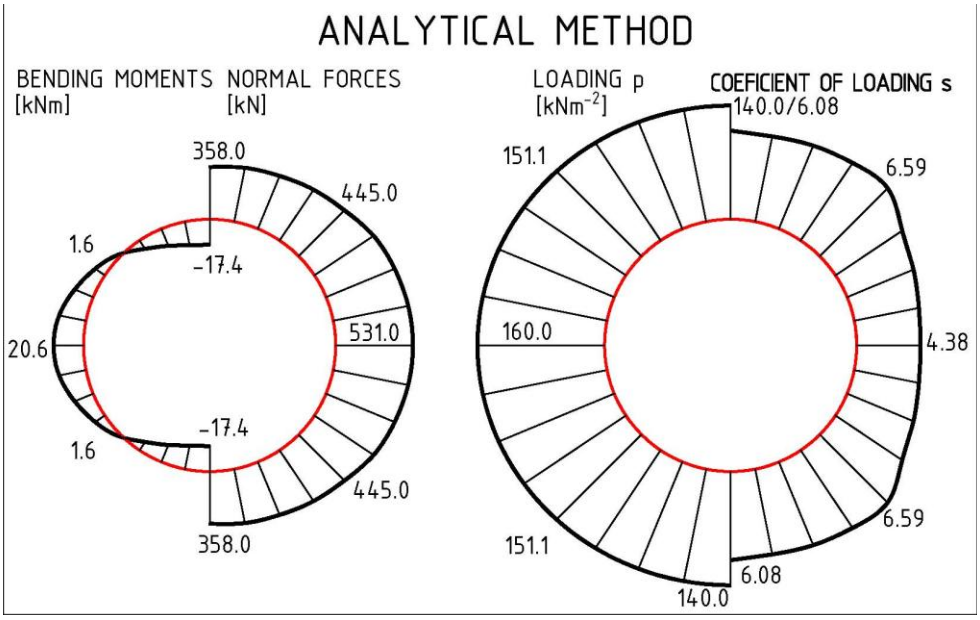

Figure 5. Figure 5. Calculated static quantities in the model for the Analytical method

8. POLYGONAL METHOD

The circle’s centerline was replaced by a 32-sided polygon. Passive resistance arises in the compression springs modeling the rock mass, which are attached to the vertices of the polygon; tension springs are not included in the calculation. After several iteration cycles, the tension springs in the upper part of the lining were eliminated, and the following values of internal forces were obtained (Zlámal 2010). After the relevant calculations, we obtain the following values of internal forces

Angle θ	*Internal Forces PM*	
	Moment (kNm)	*Normal Force (kN)*
0	-51,3	294,7
π/4	14,7	364,9
π /2	30,5	396,2
π 3/4	-17,5	456,7
π	-5,1	509,9

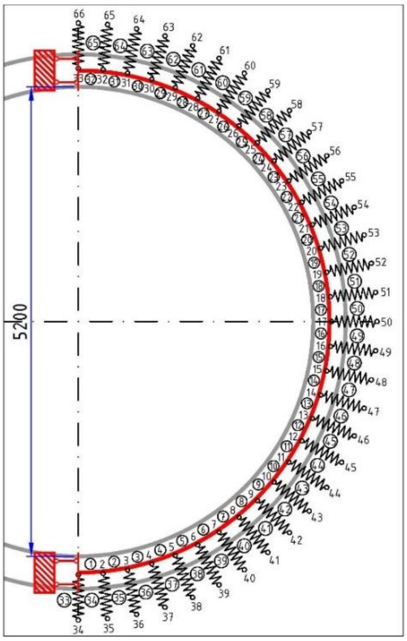

Figure 6. Static diagram

In the general deformation method (Aldorf 1992), the continuous centreline is replaced by a polygon, and the interaction between the lining and the rock is modelled using a system of oscillating members. The distribution of moments, normal forces, loads, and bearing capacity coefficients is shown in the figure.

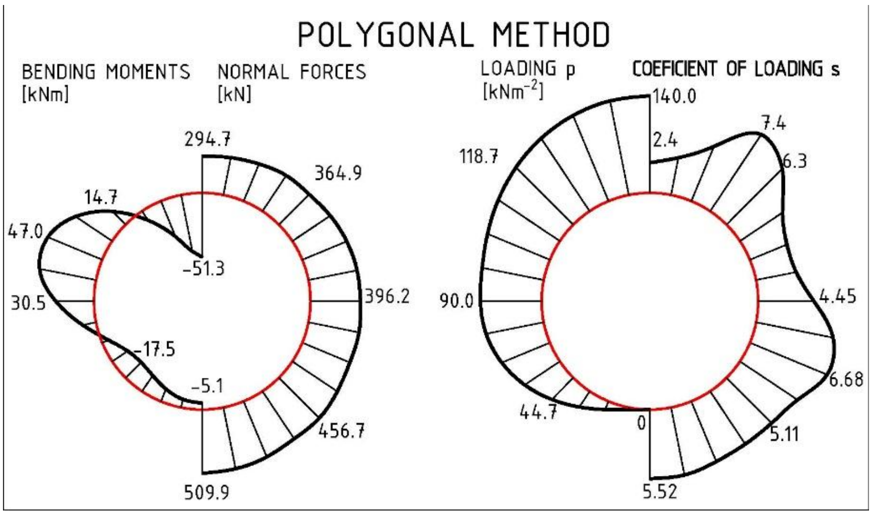

Figure 7. Calculated static parameters in the model for the Polygonal Method

9. FINITE ELEMENT METHOD

The FEM model was created with the aim of replicating the boundary conditions of the compared computational systems as closely as possible. The calculation was performed assuming linear rock deformations. The finite element mesh consists of 400 elements and 1,262 nodes and has dimensions of 60 x 49m. The element mesh was refined at the interface between the lining and the rock. After performing the relevant calculations, we obtain the following values for internal forces (RIBTEC 1996).

Angle θ	*Internal Forces FEM*	
	Moment (kNm)	*Normal Force (kN)*
0	-26,9	203,1
$\pi/4$	-2,2	330,4
$\pi/2$	33,4	494,1
$\pi\ 3/4$	5,1	427,0
π	-34,1	300,2

The distribution of moments, normal forces, loads, and bearing capacity coefficients is shown in the figure.

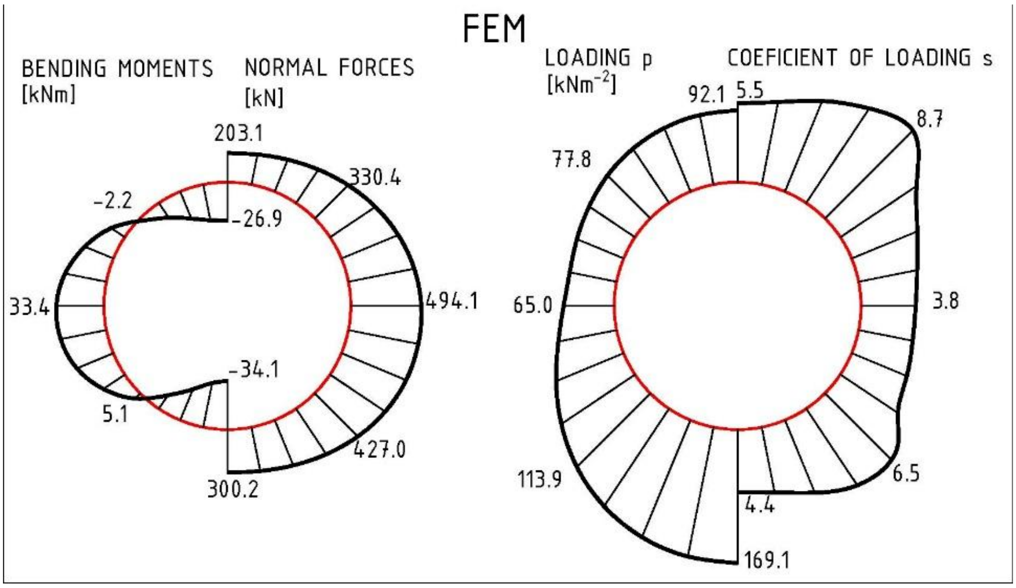

Figure 8. Calculated static parameters in the Finite Element Method

10. COMPARISON OF THE LOAD-BEARING CAPACITY OF THE LINING

The lining strength coefficient ***s*** describes the safety margin in the lining's load-bearing capacity and indicates the location where the load-bearing limit will be reached first. The location of the critical point in the lining varies: while in the polygonal method the critical point is at the apex of the arch, in the analytical method and the finite element method the critical point is at the horizontal centreline of the tunnel. According to Equation {18}, we conclude that the critical point of the lining is where the value of the coefficient s is smallest. In the analytical method, the minimum bearing capacity coefficient is s_A = 4.38; in the polygonal method, the minimum bearing capacity coefficient is s_P = 2.4; and in the finite element method, the minimum bearing capacity coefficient is s_K = 3.8. The values of the critical bearing capacity coefficient differ for the individual calculation methods; the relatively large difference in bearing capacity coefficients is most likely caused primarily by the different methodologies used to calculate the lining/rock mass system. Although the calculation using the analytical method was performed with the maximum possible safety margin, the deformation increase coefficient α^* (Aldorf 1992) between the excavation and the installation of the lining was set to α^* = 1, it appears that the rock mass and the lining interact most strongly in the analytical calculation method and least strongly in the polygonal calculation method.

11. STRENGTH AND MODULUS OF ELASTICITY OF CONCRETE

Determining the ultimate load-bearing capacity of the lining and, subsequently, the ultimate deformation during the first 28 days is only possible with knowledge of the increase in concrete strength and modulus of elasticity during this period. Data collected over a very long period are shown in Figs. 9 and 10. Two regression curves are plotted for the strength of sprayed concrete; the increase from 6 to 360 minutes (0.0041–0.25 days) is significantly different from that in the period from 360 to 40,320 minutes (0.25–28 days). For the period from 6 to 360 minutes, a regression curve is plotted with the equation

$$f_{ck} = 0{,}0051x + 0{,}2389$$

$$R^2 = 0{,}5416$$

For the range of 360 to 40,320 minutes, a regression curve with the equation

$$f_{ck} = 4{,}3793\, ln(x) - 20{,}935$$

$$R^2 = 0{,}4979$$

For the deformation module, a regression curve with the equation

$$E_{cd} = 3{,}2824\, ln(x) - 7{,}9132$$

$$R^2 = 0{,}5545$$

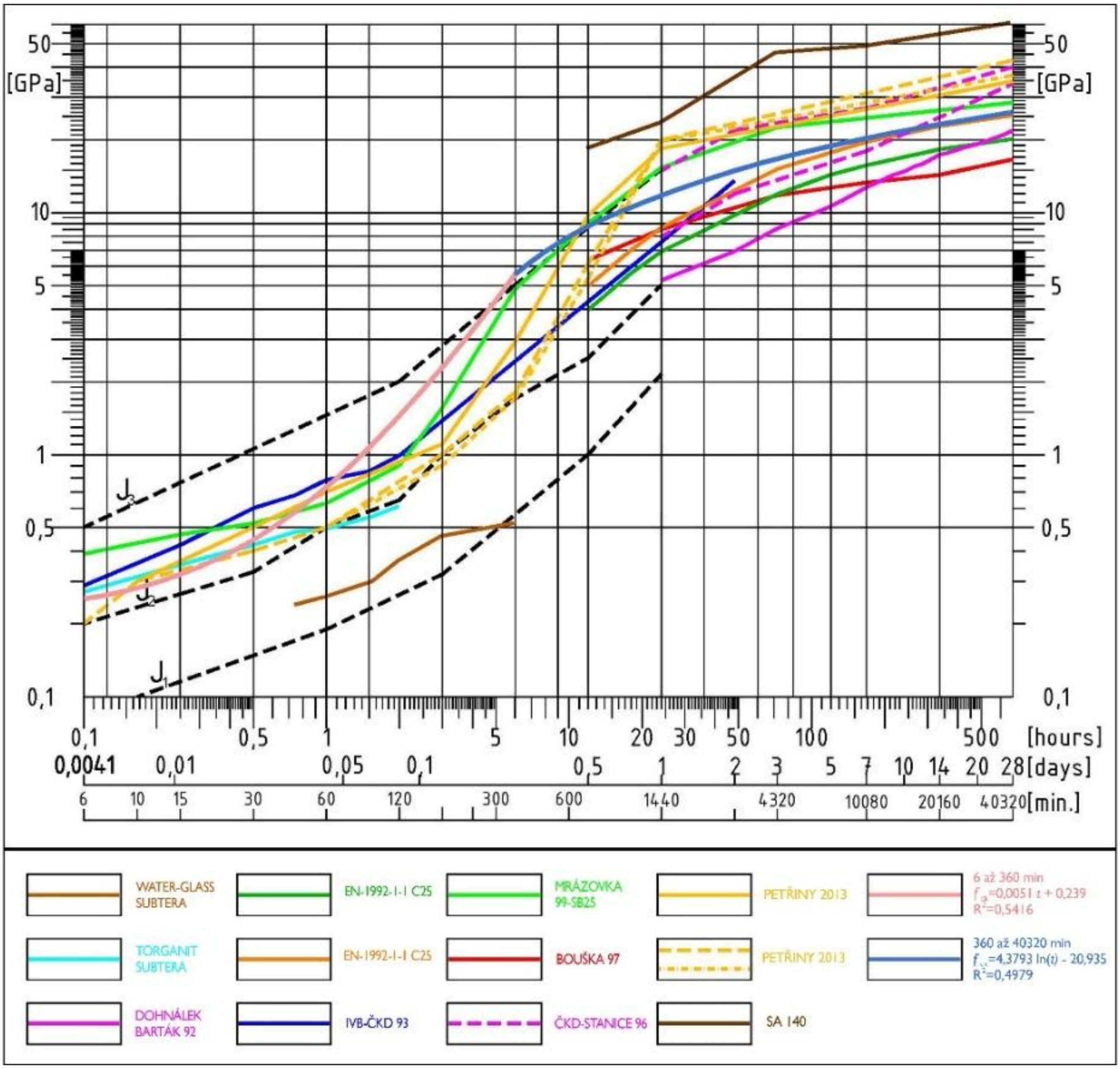

Figure 9. Measured strengths of sprayed concrete and regression curve (ZLÁMA 2010, ZLÁMAL 2007)

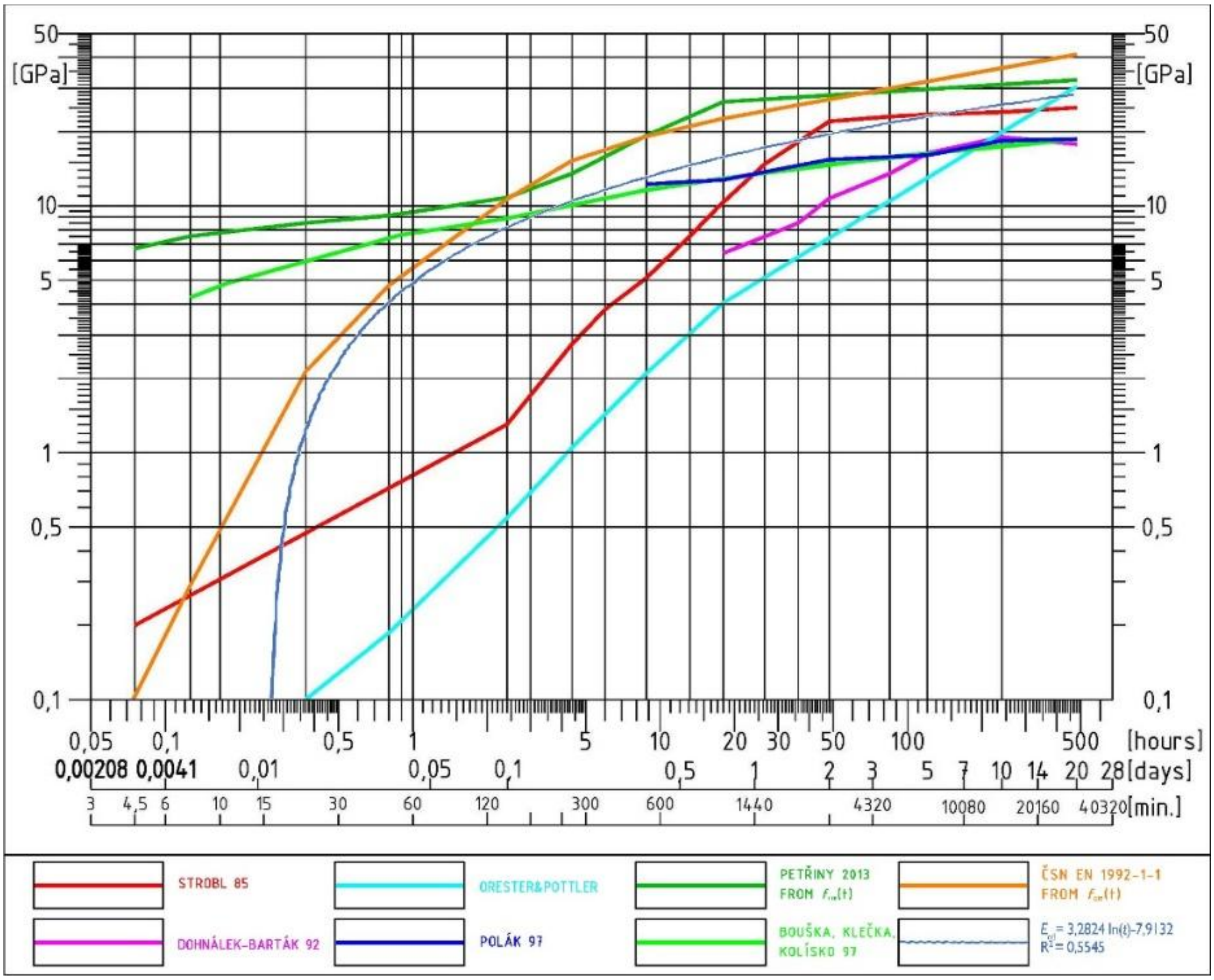

Figure 10: Measured strain modules and regression curve

12. CONCLUSION

When designing tunnel linings, we always use a calculation method that is both safe and cost-effective, taking into account the scope and importance of the task. The aim of comparing different calculation methods was to identify differences in structural parameters and the calculated load-bearing capacity of circular tunnel linings, and to determine the safety margins resulting from the use of different calculation systems. For the example given, the universal and simple polygonal calculation method is 1.82 times safer than the analytical calculation method and 1.58 times safer than the finite element calculation method. After evaluating the results of the calculation methods, we can conclude that for simple static calculations, the results from the polygonal method are reliable provided that this calculation method is supplemented with certain calculations, such as the assessment of a rock mass, a rock pillar, or a subsidence basin. In cases where it is necessary to understand the behaviour of the rock mass, we use the finite element method, which is 1.15 times more reliable than the analytical method. For simple calculations of tunnel linings, the use of the polygonal method is fully adequate.

REFERENCES

- ALDORF, J. The influence of time-dependant deformation properties of concrete on load and loading capacity of support of underground working. Rotterdam: A. A. Balkema, 1992. ISBN 90 5410 039 7.
- ALDORF, J. Mechanika podzemních konstrukcí. Ostrava: VŠB – Technická universita Ostrava, 1999. ISBN 80-7078-695-7.
- BARTÁK, J. a PRUŠKA, J. Podzemní stavby. Skriptum. Praha: ČVUT, 2011. ISBN 978-80-01-04789-7
- BULYČEV, N. S. Mechanika podzemnych sooruženij. Moskva: Nedra, 1982.
- DROZD, K. O přetvárných charakteristikách zemin a skalních hornin. Geotechnika. 2001, roč. 2001, č. 2, s. 3-6. ISSN 1211913X.
- MUSCHELISVILI, N. I. Nekotorye osnovnye zadaci matematiceskoj teorii uprugosti. Moscow: Nauka, 1966.
- RIBTEC. Tunnel. CD, Program. Stuttgart: RIB Bausoftware GmbH, 1996.

- ZAPLETAL, A.; BUCEK, M. a BARTÁK, J. Komentář ke směrnici pro navrhování ostění budovaných NATM. Směrnice. Praha: Metrostav & ČVUT, 1992.
- ZLÁMAL, J. Vyhodnocení únosnosti tunelového ostění. Praha: ITA/AITES cz, 2010. ISBN 978-80-254-7054-1.
- ZLÁMAL, J. Analysis of the ultimate states of the underground tunnel lining. Leiden: Taylor & Francis/Balkema, 2007. ISBN 978-0-415-40807-3.

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