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Design of the primary lining of a double-track metro tunnel

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List of contents

Abstract

The main objective of this work is the design of the primary lining of double-track metro tunnel of the currently constructed extension of the Prague metro line A with the application of numerical modelling of the construction using the finite element method software. The work also describes the New Austrian Tunnelling Method principles and technology as it is the method used for the construction of this section.

Abstrakt

Hlavním cílem této práce je návrh primárního ostění dvoukolejného tunelu metra právě prodlužované linky A pražského metra s pomocí numerického modelování konstrukčních fází v softwaru užívajícím metodu konečných prvků. Práce také stručně popisuje zásady Nové rakouské tunelovací metody jako metody použité pro výstavbu dané části.

1 Introduction

The Prague metro system has been developing over 40 years. It is the most important part of the city public transport; it connects the outer parts of the city with the city centre and presents the fastest and most comfortable mean of transport accessible in the city. As the city grows away from the centre, highly populated areas develop outside the reach of the existing metro lines and their extension is needed.

In 2010 the works commenced on the extension of metro line A from Dejvická station to the Prague international airport. The extension is divided into three sections. The operating section V.A from the Dejvická station to the Motol station is currently under construction (Figure 1). The section will interconnect highly populated areas west and south-west from the present end station and improve their accessibility by public transport. The 4,54 km long route segment between Dejvická station and Na Vypichu construction site is designed as two single-track rail tunnels and it will be bored by full-face tunnelling machines; representing the first use of such machines in the Czech Republic. The segment between the Na Vypichu construction site and the Motol station is made of one double-track rail tunnel and it will be bored by the predominant tunnelling method used in the country, the New Austrian Tunnelling Method. Also a 400m long double-track tunnel for turning tracks will be constructed using the same method behind the Motol station to allow the trains to turn around until the following sections are constructed. The typical profiles are presented in Figure 2.

The aim of this work is the design of the primary lining of double-track metro tunnel. The main task will be the development of the mathematical model of the construction process. Finite element method modelling in the GEO 5 – FEM-Tunnel program will be used to obtain the internal forces in the tunnel lining for the particular construction stages. The proposed design of the reinforced sprayed concrete primary lining will be then verified using the FIN EC – Beton 2D program.

Apart from the modelling and design the thesis gives an insight into the fundamentals of the New Austrian Tunnelling Method as the technology largely used in the Czech Republic and the technology used for the route segment in question.

2 Geological and Hydrogeological Conditions

The construction site of the Motol station and its close vicinity experience very complicated engineering geological conditions and quite variable lithological composition. This is caused mainly by the existence of major rupture zone of the Prague fault which runs diagonally through the construction site and also by the presence of ancient cretaceous block

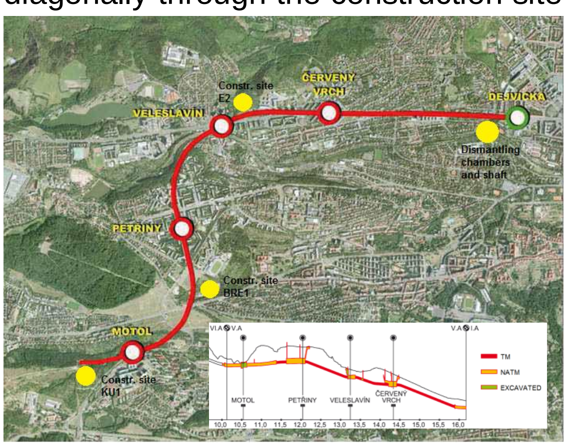

Figure 1 – OS V.A – layout and idealised longitudinal profile (Zakládání 3/2010)

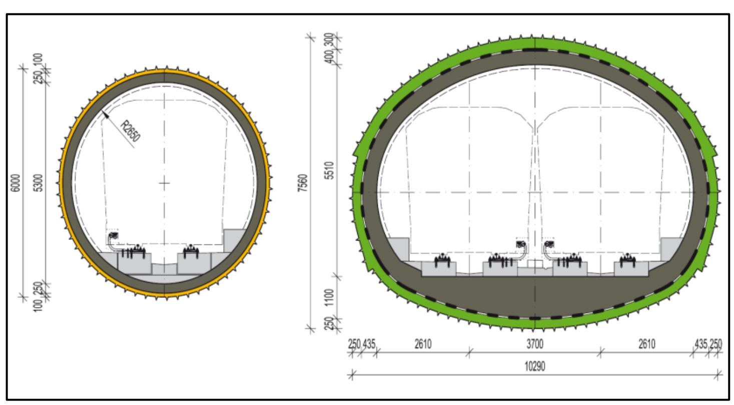

Figure 2 – Single-track tunnel bored by TM Double-track tunnel bored by NATM (RAD 3,7m) (Zakládání 3/2010)

slide on the south slope of Bílá Hora.

There have been 13 geotechnical types of rock identified in the respected area. In the following text only those relevant to the particular tunnel profile are described in detail.

The touched geological formations are the following:

- Slope sediments – Quaternary period
- Clayey shales – Ordovician period (Bohdalec formation)
- Marlites – Cretaceous period (Bílá Hora formation)
- Sandstones – Cretaceous period (Korycany formation)
- Lightly cemented rocks of Peruc formation – Cretaceous period

The main geotechnical types were derived from the geological formations with the exception of the area affected by the ancient block slide.

The body of the ancient block slide, sometimes abbreviated as a single geotechnical type R, is composed of three different geotechnical types.

The engineering geological survey produced fairly acceptable illustration of how the sliding blocks may be distributed, but since the respective profile is situated at the very bottom end of the slide, the blocks are most likely reduced to boulders or even gravel and the individual geotechnical types are mixed together, therefore simple stratification cannot be used.

The **ground water** reached in exploring boreholes was mostly of joint character. The permeability of Bohdalec formation is expected to be very low, except for the places where the rock is severely weathered or highly fissured. The ground water flow corresponds to the slope inclination.

The characteristics of individual geotechnical types are presented in Table 1.

Table 1 – Double track rail tunnel – Geotechnical characteristics (after Metroprojekt)

Geotechnical type		γ (kN/m^3)	E_{def} (MPa)	ν (-)	c (kPa)	φ (°)
GT1-D	Deluvium	19,0	15	0,30	0	30
GT2-Kb	Marlites	22,0	600	0,20	250	50
GT3-Kk	Sandstones	19,5	400	0,20	100	40
GT4-Kp	Lightly cemented rocks	20,0	60	0,25	50	30
GT10-Bd	Clayey shales – completely weathered	22,0	45	0,30	35	25
GT11-Bz	Clayey shales – weathered	24,0	200	0,25	75	35
GT12-B	Clayey shales – partly weathered/sound	23,0	600	0,25	70	50

3 Technological Solution of the Respective Profile

The solved profile is located in the close vicinity of the Motol station. The profile denoted as *Profile DTA 80,0m*, is a part of the double-track rail tunnel from Motol station towards the Petřiny station.

As mentioned above, the tunnels constructed from the BRE1 construction site towards the Motol station and the tunnels for turning tracks behind the Motol station will be bored in compliance with the NATM technology. Therefore the following section is devoted to the New Austrian Tunnelling Method itself and the description of the particular profile is in section 2 of this chapter.

3.1 The New Austrian Tunnelling Method

The New Austrian Tunnelling Method is the most widely used tunnelling method in the Czech Republic and under local modifications and names probably around the world. The most known variations are the Sequential Excavation Method - SEM used in the United States and the Sprayed Concrete Lining – SCL used in the United Kingdom.

3.1.1 Principles of NATM

The original description by Prof. L.v. Rabcewicz presented in November 1964 was that the NATM is "… *a new method consisting of a thin sprayed concrete lining, closed at the earliest possible moment by an invert to a complete ring – called an "auxiliary arch" – the deformation of which is measured as a function of time until equilibrium is obtained."*(Rabcewicz 1964)

The definition was later, in 1980, redefined by the Austrian National Committee on Underground Construction of the International Tunnelling Association (ITA) to resolve the conflicts in literature. The new definition reads: "*The New Austrian Tunnelling Method (NATM) is based on a concept whereby the ground (rock or soil) surrounding an underground opening becomes a load bearing structural component through activation of a ring like body of supporting ground*" (quoted by Karakus & Fowell 2004)

Based on the above quoted and numerous other statements, it is widely accepted that the NATM is more of a philosophy of tunnelling with certain principles than actual construction method even though the name would imply so.

The whole concept of New Austrian Tunnelling Method comes from Rabcewicz's theory of failure around a cavity by stress rearrangement pressure.

At first, wedge-shaped bodies on either side of the tunnel are sheared off along the Mohr surfaces and move towards the cavity (I). In stage two, the increase in the span leads to convergence of the roof and floor. The deformation at the crown and the floor of the cavity increases more and the rock buckles into the cavity under the constant lateral pressure (II). The pressures that arise in stage (III) are termed "squeezing pressures" and rarely occur in civil engineering activities due to shallow depth of excavations.

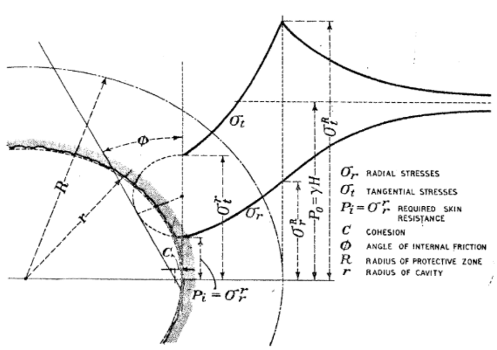

Figure 4 – Stress distribution around a cavity under hydrostatic pressure (after Kastner, quoted by Rabcewicz 1964)

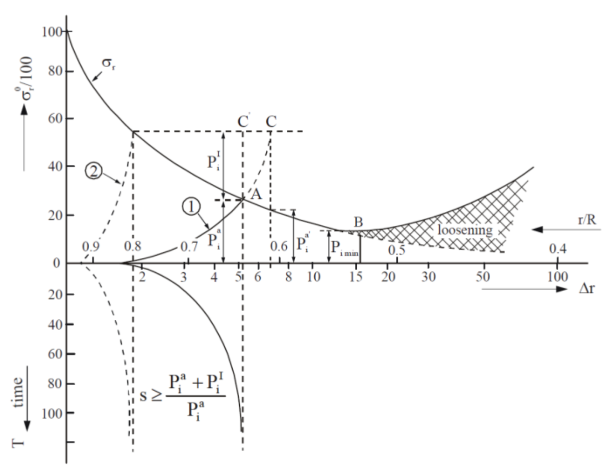

Figure 5 – Ground support interaction curve (after Fenner & Pacher, quoted by Rabcewicz 1973)

The stress rearrangement pressure must be counteracted by the so-called skin resistance. The skin resistance is in other words the load bearing capacity of the primary lining. The required skin resistance decreases as the peak of tangential stresses moves away from the cavity which radius is simultaneously decreasing. The mathematical relationship was described by Kastner (after Karakus & Fowell) as:

$$p_i = -c\cot\varphi + p_0[c\cot\varphi + (1-\sin\varphi)]\frac{r^{\frac{2\sin\varphi}{1-\sin\varphi}}}{R}$$

Omitting the cohesion, the equation yields to

$$p_i = p_0(1-\sin\varphi)\frac{r^{\frac{2\sin\varphi}{1-\sin\varphi}}}{R} = np_0$$

The values of n are given as a function of p_0 and φ (see Rabcewicz 1964). The formulae are derived from stress distribution in rock after cavity has been open, as illustrated in Figure 4.

The relationship between the decreasing skin resistance required for the support and the increasing deformation is presented in so called Fenner-Pacher curve as shown in Figure 5. The curve shows the increase in radial stress as a function of increasing deformation and the deformation-time relationship.

It is shown that a stiff support installed in short time after cavity opening (line 2) will limit the deformation development (lower part of Figure 5) and will result in great load to the support (point C). If the support is installed after a certain displacement is allowed (line 1) and the support is of the right stiffness, it will interact with the rock in means of gradually taking up loading and decreasing the rate of deformation, eventually reaching equilibrium of its bearing capacity and the loading (point A).

The ideal case would be to apply such support at such time that the equilibrium would take place in point B, thus carrying the minimal possible loading. After point B, the rock around the cavity starts to loosen applying additional load to the support.

In 1978 Prof. Müller, one of big supporters of the method, published 22 principles of the New Austrian Tunnelling Method. In general, these can be summarised as the following seven major principles (Müller 1990 quoted by Karakus & Fowell):

i. The inherent strength of the soil or rock around the tunnel domain should be preserved and deliberately mobilised to the maximum extent possible
ii. The mobilisation can be achieved by controlled deformation of the ground. Excessive deformation which will result in loss of strength or high surface settlements must be avoided.
iii. Initial and primary support systems consisting of systematic rock bolting or anchoring and thin semi-flexible sprayed concrete lining are used to achieve the particular purposes given in (ii). Permanent support works are usually carried out at a later stage.
iv. The closure of the ring should be adjusted with an appropriate timing that can vary dependent on the soil or rock conditions.
v. Laboratory tests and monitoring of the deformation of supports and ground should be carried out.
vi. Those who are involved in the execution design and supervising of NATM construction must understand and accept the NATM approach and react co-operatively on resolving any problems.
vii. The length of the unsupported span should be left as short as possible.

3.1.2 Technology of NATM

The boring operations according to the New Austrian Tunnelling Method are done in cyclic fashion; it means that a given sequence of operations is repeated in the same manner along the whole tunnel in sections of so called advance length.

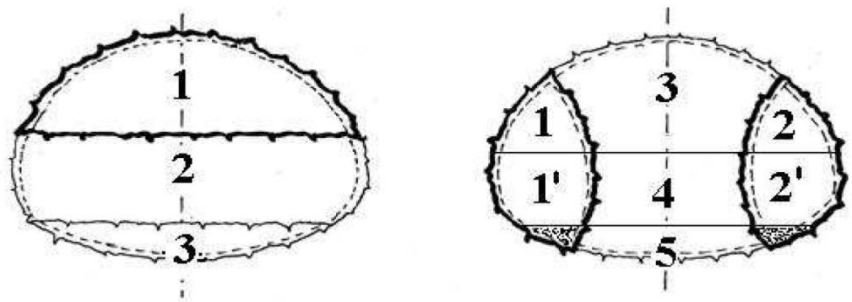

Figure 6 – Tunnel face subdivision (after Barták 2009/10)

A) Horizontal face subdivision
1 – Top heading, 2 – Bench,
3 – Invert

B) Vertical face subdivision
1,1',2,2' – Side galleries
, 3 – Top Heading (core),
4 – Bench, 5 – Invert

Based on geological conditions blasting, roadheaders, impact hammers or tunnel excavators are used for rock disintegration. Each increment of face advance is after excavation secured by successively built support.

Firstly, primary lining made of sprayed concrete reinforced by steel mesh and when needed supplemented by steel arches and rock bolting is constructed. The primary lining is usually quite thin, depending on the tunnel dimensions and geotechnical conditions it varies from 100 to 400mm.

Not until the deformations cease and the rock-primary lining system reaches equilibrium the secondary lining is installed. This is controlled by interval measurements of convergences of the primary lining.

In case the deformations would not cease and would approach critical values given by the design, additional stabilization measures must be used to prevent overloading of the primary lining and collapse of the tunnel. Stabilization measures usually used are strengthening of the primary lining, intensification of rock bolting, shortening of advance length, face support by rock pillar or its anchoring, change in face sub-division, spilling, forepoling, umbrellas of micro piles or sub horizontal jet grouting columns, rock improvement by pressure grouting, improvement of overlaying strata by jet grouting and others.

The secondary lining is usually made of reinforced concrete casted in-situ into movable formwork. It usually ensures the load bearing function for the entire lifetime of the structure (the primary lining is likely to deteriorate in time due to direct contact with the rock/soil environment). Intermediate foil (alternatively sprayed) insulation is usually placed between primary and secondary lining to protect the latter from the aggressive outer environment and to seal the tunnel interior. In some cases the insulation may be replaced by watertight concrete with appropriate exposure class.

The underground structure is preferred to be bored full-face, that means the whole profile is excavated for individual face advance. However, face subdivision may be required for geological and geotechnical reasons (increase of stability, decrease of face area, limitation of deformations and subsidence) and also for technological reasons (machinery used, manipulation space). Two basic schemes of face subdivision are horizontal face subdivision and vertical face subdivision (Figure 6)

When applying either one of

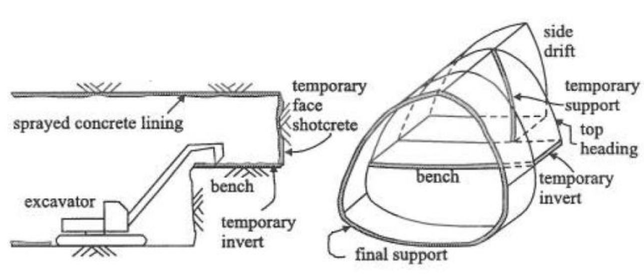

Figure 7 – Excavation sequence of the NATM (after Thomas)

the subdivision types, each excavation stage follows the previous in certain interval. This is to allow the stress redistribution in the surrounding rock and also from technological reasons (manipulation space, access to advanced excavation). An example of face subdivision and advancing excavation of individual parts is shown in Figure 7.

3.1.3 NATM technological classes

For the purposes of realisation, design, preparation and budgeting of the construction of underground spaces built using the New Austrian Tunnelling Method it uses classification into 6 technological classes. The classes represent the rock quality, influence of ground water, likely overbreaks and predetermine probable face subdivision and accessory needed. Generally the lowest class represents the best conditions for boring operations; with increasing class the geological and geotechnical conditions become more complicated.

It is necessary to say that the class is usually estimated from engineering geological survey in the design stage, but is finally determined during the construction based on the actual conditions.

3.1.4 Geotechnical monitoring for NATM

As mentioned in previous subsections, NATM is based on continuous observation of the rock and support behaviour to verify the design and the ability of lining to support the rock. The monitoring is done by many different instruments, but the main aim is to provide systematic and regular data for evaluation.

The main measurement methods are:

- Measurement of convergences
- Measurement of the load to the support through measurement of contact stresses between the support and the rock
- Measurement of stresses in the concrete
- Measurement of deformations in the rock body
- Measurement of surface deformations (shift and subsidence)
- Measurement of forces in bolts and anchors
- Measurement of inclination changes in the rock body, in close underground spaces, on surface buildings

If possible, the measurement should start before the construction starts to have a “zero reading” not influenced by previous stages and should last long enough to cover even the late manifestations of the construction.

3.2 Double-track Rail Tunnel – Profile DTA 80,0m

Profile DTA 80,0m is one of the characteristic cross sections chosen for the design of the double-track rail tunnel from Motol station towards BRE1 construction site and further to Petřiny station. The identification data, geological and geotechnical conditions and technical and technological solution of the profile are described in the following subsections.

3.2.1 Identification data

Construction purpose:	Extension of metro line A from Dejvická station
Construction name:	Operating section V.A – Dejvická (excl.) – Motol
Building complex:	SOD 08 Route segment Petřiny - Motol
Building section:	SO 08-23/01 Bored double-track rail tunnel – Boring and primary lining – part 2

Profile designation: DTA 80,0m
Distance along tunnel axis: 80,0m
Stationing (Left track) 10,709.581 km
Rail axis distance: 4,5m
NATM technological class: 5b

3.2.2 Geological and geotechnical data

The geological conditions with the relevance to the tunnel structure are shown in Figure 8. The geotechnical type classification corresponds to the one presented in chapter 3 and the individual type characteristics are presented in the same section in Table 1.

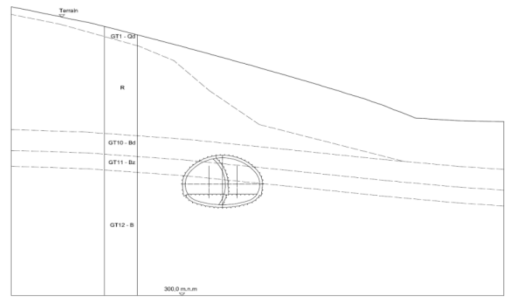

Figure 8 – DTA 80,0m – Geological conditions (after Metroprojekt, modified)

3.2.3 Technical and technological solution

(Figure 9)

Vertical face subdivision in designed for this profile. First partial stope is designed to be 6,505m wide and 8,15m high. The second partial stope will widen the space to its final dimensions, 11,395m wide and 8,22m high. The primary lining is designed 400mm thick made of concrete grade SB25 (C20/25) and reinforced on the inner and outer perimeter by steel mesh of the wire dimension 9mm and spacing 100mm in both directions. Also steel reinforcing arches will be added to the reinforcement every advance length.

The geometry of the lining is based on the rail axis distance and the train passing clearance. The shape is made of arch segments of various radii. The top arch has inner diameter of 5,9m, it passes into the upper side arch segment with the diameter of 3,20m and then into lower side segment with the diameter of 2,22m. The invert is closed by an arch segment with the inner radius of 10,70m. The transition from one radius into another is always smooth ensuring continuous shape of the stope to eliminate any stress concentration.

Horizontal face subdivision is expected for both partial stopes. Top heading and the bench will be bored together and the invert will be excavated in later time. The maximum distance where the invert is not yet closed is 6m from the tunnel face. The distance may be changed according to the monitoring data.

Elephant’s foot, 0,25m wide, is designed at the bottom of the first stage of primary lining at the border of the initial excavation and the invert for both temporary and permanent supports. It is supposed to increase the stability of the primary lining once the invert has been excavated but the bottom has not been yet sprayed with concrete or the concrete doesn’t have sufficient strength.

Based on the geological conditions and the geotechnical data, the expected technological class of this profile is 5b. The accessory proposed is rock bolting of the top and sides and

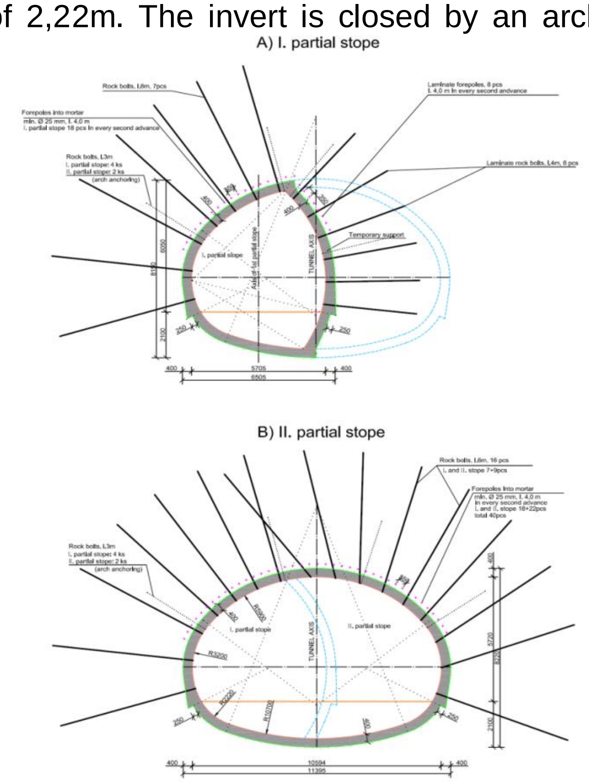

Figure 9 – DTA 80,0m – Technical and technological solution (after Metroprojekt, modified)

forepoling.

4 Numerical Modelling

One of the main features of today's engineering practice is the need and effort to produce the optimal and most efficient design. This effort leads to the creation of still more complex and detailed numerical models of engineering structures. The method used for the solution of such models is of primary importance regarding the expected output quality and accuracy. The development of computers in the second half of 20th century allowed the application of classical numerical methods in wide range and encouraged the evolution of new methods which would not be possible to use without computers. The finite element method (FEM), the finite difference method (FDM) and the boundary element method (BEM) are just some of those methods. The finite element method gained over time the leading role among the others. Its universality and flexibility predetermine it for wide range of use; therefore it became the most widespread method for solution of complex engineering problems.

4.1 Finite Element Method (FEM)

The basic idea of finite element method is the *element discretization* (Figure 10). This is the process of breaking up the geometry of the problem into small regions, *finite elements*. These elements are of simple geometric shape, finite area and defined deformational characteristics. The elements are connected by *nodes* on the element boundary. Nodes may be also placed within the element. Different shapes of elements with different number of nodes (Figure 11) are used for specific problems, however, for planar problems triangular or rectangular elements are most common.

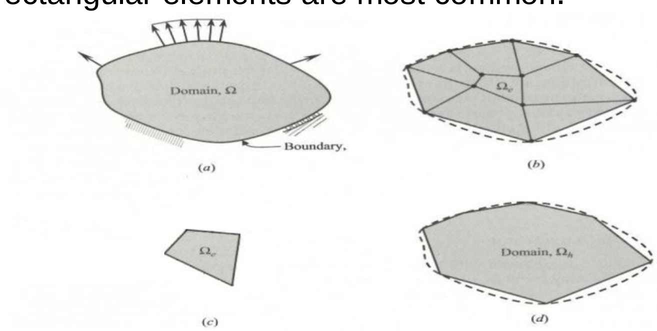

Figure 10 – FEM domain discretization (Reedy)

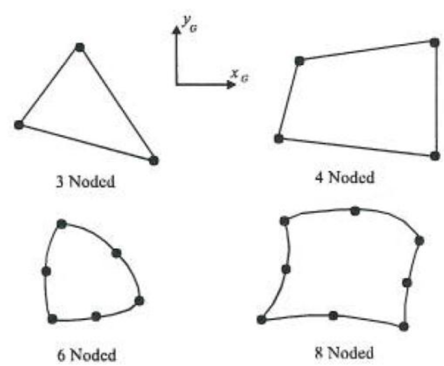

Figure 11 – Examples of finite elements (Potts & Zdravkovič)

The collection of finite elements, *the finite element mesh*, roughly approximates the geometry of the continuum, e.g. a circle is modelled as an n-sided polygon, the number of sides and elements depending on the choice of the resolver. The loading of the system is not transferred directly to the elements; it is done through forces acting in the nodes; the forces being statically equivalent to the original load.

After *primary variable* has been established and its variation defined, *element equations* are derived for each element separately. Each element represents an independent domain. The element equations are then combined into *global equations*. The formulation of *boundary conditions* then modifies the global equations to its final version. The global equations are in the form of a large number of simultaneous equations. These are solved to obtain the primary variable values at all the nodes. From these values, all secondary quantities are evaluated. Using interpolation, values of any quantity at any place may be calculated from the obtained nodal values.

4.2 Finite Element Method in Geotechnics

The peculiarity of modelling geotechnical problems is the nature of the material (rock or soil) which creates the continuum; therefore different constitutive material models are very essential to geotechnical software.

There is a large number of computer programs denoted to the numerical modelling of geotechnical problems, they may vary in the used method, but the finite element method is the most popular by far. Another distinguishing feature of such software is the ability to work in three-dimensional space (3D). However, most of the software is limited to solving two-dimensional problems (2D).

The 3D modelling considers a three-dimensional space and is therefore more suitable for tunnel construction modelling. Nevertheless, its enormous time demand for setting up the model and higher price of the software makes it less attractive. It is usually used only for modelling of very complicated problems (such as intersecting tunnel tubes, large underground spaces, irregular shapes, etc.) which cannot be easily and reliably modelled in using two-dimensional tools.

Commonly used programs for finite element modelling in geotechnics used in the Czech Republic are PLAXIS (for both 2D and 3D modelling) and GEO (for 2D modelling).

4.3 GEO 5 – FEM-Tunnel Geotechnical Program

The numerical analysis of the profiles described in sections 5.2 and 5.3 was done using the GEO 5 – FEM-Tunnel geotechnical program. The software uses the finite element method to solve two-dimensional problems and is an upgraded version of the standard GEO 5 – FEM program.

The standard program can model a wide range of geotechnical problems; the Tunnel extension contains two options for modelling linings, use of a relaxation factor to model the 3D behaviour of the body during excavation and others.

Generally, the use of the program is divided into three stages Firstly; in the *topology input stage* we define the geological conditions by drawing the layer interfaces, defining soil/rock types and assigning them to individual profile layers. The geometry of all parts of the structure used through the whole calculation is described by free points which are then connected by free lines. Also other setup options and values are defined in this section, such as analysis method and contact types. The boundary conditions are set automatically, but can be redefined by the user.

The second step is the finite element mesh generation. The topology correction and mesh generation are fully automatic. However, user may define the finite element edge length and also refine the mesh around a single points or lines. The default setting of the program generates a mesh made of 6-node triangular elements. The option to switch to 3-node triangular elements is provided but discouraged.

After successful mesh generation, user is allowed to set the construction stages. This is done by activation or deactivation of individual regions, construction of beams along free lines, changing the properties of previously defined elements, etc.

Results of the calculation are available after each construction stage and can be shown as total (for all previous construction stages) or as incremental (the change with respect to the previous construction stage). For presentation of the results isolines or isosurfaces may be used, also internal forces may be drawn along the beams. Additionally, contact stresses, deformed structure, deformed mesh and translation vectors for each node may be shown.

4.4 Material model

From the number of material models available, the Mohr-Coulomb model was chosen. The Mohr-Coulomb model assumes that the material fractures at a point where the plastic equilibrium is overcame and the shear strength exhausted. The shear strength (τ) is defined as the resistance against shearing on a slip surface. The value of shear strength is not constant but varies with the value of normal stress (σ) and with the material properties; angle of internal friction (φ) and cohesion (c), by the following relationship described by Coulomb.

$$\tau = \sigma . \tan \varphi + c$$

Graphically the equation is represented by set of Mohr circles representing individual failure stress states and the envelope representing the failure criterion (Figure 12). For soils, it has generally linear shape and the tension strength is not assumed. For rocks, the line becomes of higher order and certain tension strength is expected.

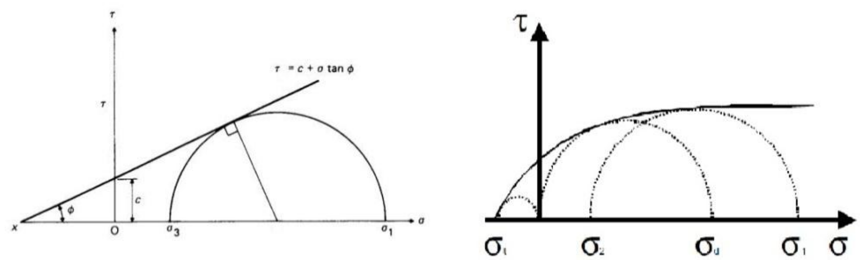

Figure 12 – Mohr-Coulomb failure criteria for soils (left) and for rocks (right) (Geever, Pruška)

5 Model development

The development of the model is essential to the output correctness and that is why maximum attention should be given to every part of the input procedure. Majority of the input data are obtained from the literature or construction documents, but there are some details which have to be solved analytically or by experience. The following sections deal with the individual aspects of the model development.

5.1 Margins and Interfaces

The whole model space is defined by its margins. The margins create so-called world of the model. The world should be big enough to cover the soil deformations in full extent. On the other hand, too big world would unnecessarily increase the number of elements and the time of calculation.

The margins for the model were applied at the same extent as the original profile drawings. That is 30m to the left and 40m to the right of the tunnel axis and approximately 15,9m below the tunnel tube for Profile DTA 80,0m

5.2 Soils

The geotechnical characteristics of the soil layers were adopted from the engineering geological survey of the site (see chapter 2). The geotechnical types and soil abbreviation correspond to the chapter 2 (Table 1) and Figure 8 with one exception.

The layer marked as R in the Figure 8 is the area affected by the ancient block slide. The slide is made of three geotechnical types (GT2-Kb, GT3-Kz and GT4-Kp) and because the geotechnical conditions are not exactly known and these layers are expected to be mixed together, the characteristic values of the weakest soil type (GT4-Kp) were used and assigned to the whole region, this being on the safe side.

5.3 Geometry

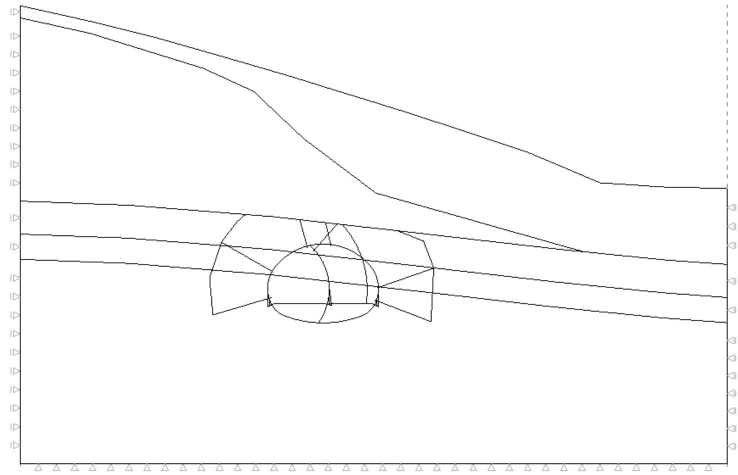

Figure 13 – DTA 80,0m - Geometry

The geometry of the tunnel lining, face subdivision, areas affected by rock bolting, temporary support and the shape of the elephant's foot were first prepared in means of points and lines in AutoCAD from the drawings of the tunnel design. The primary lining is represented by a line placed into the lining axis. The file was then transferred into DXF format and imported to the GEO 5 – FEM-Tunnel program. Using free points and free lines the geometry was defined. The final geometry of the model is presented in Figure 13.

5.4 Sprayed Concrete Primary Lining

The modelling of the primary lining presents one of the obstacles of the tunnel modelling. The sprayed concrete itself exhibits changing properties in time; for time independent modelling certain approximation must be used.

There are many empirical formulae for determination of sprayed concrete modulus of elasticity and compressive strength in time but values used in engineering practice were used instead.

The lining is modelled as a beam placed onto a free line in an appropriate construction stage. The cross-section was defined from a roll-out menu as rectangular concrete wall with the thickness corresponding to the respective profile (0,4m). The beam end-point connection is designed as rigid (there is one exception described in section 5.5).

The material characteristics were input manually to ensure the time-dependent behaviour. The program does not allow time variation of the compressive and tensile strength, which is why variation only in modulus of elasticity and shear modulus were used for the simulation. The values used are presented in Table 2.

Table 2 – Material properties of sprayed concrete

<table>
<tr><td></td><td>γ [kN.m^{-3}]</td><td>E [GPa]</td><td>G [GPa]</td><td colspan="2">f_{ck} [MPa]</td><td colspan="2">f_{ct} [MPa]</td></tr>
<tr><td>Young sprayed concrete</td><td>25,0</td><td>15,5</td><td>6,45</td><td colspan="2">20</td><td colspan="2">2,2</td></tr>
<tr><td colspan="3">Mature sprayed concrete</td><td>25,0</td><td>18</td><td>7,5</td><td>20</td><td>2,2</td></tr>
</table>

5.5 Elephant's Foot Modelling

As described in previous sections the primary lining contains so-called elephant's foot 0,25m wide. To model the influence of the foot two approaches were considered.

The foot model is made of two beam elements. First, sub horizontal, represents the footing side of the foot, it has rigid connections on both sides. Second, vertical, represents the side of the foot and has rigid connection to the footing element but pin connection to the lining, this is to transfer no bending moment through the top connection.

The modelling was done in two steps to model the real stress distribution in individual construction stages. After the construction of the primary lining in the top heading and bench, the supporting area is actually made of the foot together with the full lining width (Figure 14-A). In the model this is solved by maintaining the footing area while excavating only the real area of excavation (Figure 14-B). This is because the lining itself is modelled as a beam element in its axis; therefore the support reaches outside the excavated area.

Just before the start of the invert excavation, the model is modified to reduce the supporting area by the width of the primary lining leaving only the area corresponding to the real width of the foot to support the section. This matches the situation when the rock/soil below the original foot is excavated and then the primary lining of the invert is put into place. The final solution of the elephant's foot model is shown in Figure 14-C, D

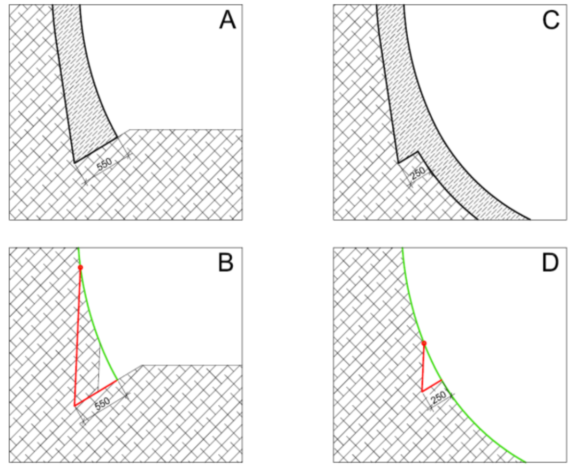

Figure 14 – Elephant's foot modelling

5.6 Contact Types

The contact elements are used in applications that require studying an interaction of a structure and a soil. They can be further used to model joints or interfaces of two distinct materials (soil – rock interface). A typical example of using contact elements is the modelling of sheeting structures, retaining walls or tunnel lining. In such applications the contact elements are used to model a relatively thin layer of a soil or rock loaded primarily in shear.

Because the estimation of the properties, especially when the contact runs through different soil layers, may be very complicated, trial-error method in the form of parametric study may be used to obtain realistic values. This is done through variation of one parameter through the recommended range while the other stays constant and vice-versa. Major result values as lining deformation and internal forces, surface subsidence, etc. are evaluated and then parameter values corresponding to realistic values are adopted.

The values of stiffness parameters resulting from parametrical studies are presented in Table 3. The values of the reduction coefficients were estimated.

Table 3 – Contact types and their parameters

	Contact type	K_n [kN/m^3]	K_s [kN/m^3]	δc [-]	δμ [-]	ψ [°]	R_t [kPa]
Profile DTA 80,0m	Plastic	60000	20000	0,80	0,80	0,00	0,00

5.7 Rock Bolts

The introduction of rock bolts into the numerical modelling was done by assuming the area affected by rock bolts to have improved properties. In particular, cohesion of the soil was increased according to the following formulas.

$$c_{s+b} = c_s + c_b$$

where c_{s+b} is the total cohesion of the soil improved by the rock bolting [kPa]
c_s is the original soil cohesion [kPa]
c_b is the cohesion increase due to rock bolting [kPa]

The increase cohesion of the soil is given by the formula:

$$c_b = \frac{N_b}{A_b} \cdot \frac{1 + \sin\varphi}{2 \cdot \cos\varphi} \cdot \frac{1}{\gamma_{bc}}$$

where N_b is the bearing capacity of the rock bolt [kN]
A_b is the area influenced by the rock bolt [m^2]
φ is the angle of internal friction of the soil [°]
γ_{bc} is the rock bolting safety coefficient [-]

The rock bolting in the respective profiles affects different soil layers and also the bolting pattern varies for each profile; therefore the entire calculation is not presented here, but just one layer is solved and the other results are presented in Table 4. The solved example is done for geotechnical type Bd:

$$c_b = \frac{N_b}{A_b} \cdot \frac{1+\sin\varphi}{2\cdot\cos\varphi} \cdot \frac{1}{\gamma_{bc}} = \frac{50}{5{,}6} \cdot \frac{1+\sin 25°}{2\cdot\cos 25°} \cdot \frac{1}{1{,}1} = 16\ kPa$$

$$c_{s+b} = c_s + c_b = 35 + 16 = 51\ kPa$$

The rock bolting pattern for the Profile DTA 80,0m is complicated and was therefore divided into five regions (Figure 15) and the layer R was not included. The regions 1-4 use the IBO type steel rock bolts of the length 6,0m, the region designated as LAM uses laminate rock bolts. Due to complicated geotechnical conditions, the bolt bearing capacity was reduced to 50kN.

Table 4 – Cohesion improved by rock bolting

Soil layer	Region	c_s [kPa]	c_b [kPa]	c_{s+b} [kPa]
GT10-Bd	1	35	16	51
	2	35	10	45
	3	35	15	50
	LAM	35	21	56
GT11-Bz	1	75	20	95
	2	75	12	87
	3	75	18	93
	4	75	8	83
	LAM	75	26	101
GT12-B	1	70	29	99
	4	70	12	82
	LAM	70	38	108

5.8 The Convergence-Confinement Method

For modelling of the three-dimensional behaviour of the soil/rock body in two-dimensional workspace the convergence-confinement method was used. The same method is widely known in the Czech Republic as the β-method. The convergence-confinement method is one the methods of tunnel excavation modelling developed for NATM tunnelling. The method assumes the excavation (removal of the finite element mesh) of a given region all at once but the load is applied gradually. An internal force vector β $\{F_0\}$ is applied to the nodes on the excavation boundary ($\{F_0\}$ being equivalent to the initial soil stress $\{\sigma_0\}$ and β being the convergence-confinement parameter). The initial value of β is 0 and is progressively increased to 1. At a prescribed value β_d the lining is installed, the remaining stress is applied to create the stresses in the lining in one or more steps. The stress reduction with the lining in the place is then (β_d) $\{\sigma_0\}$. If the tunnel is not excavated full-face, the method can be used for individual parts sequentially.

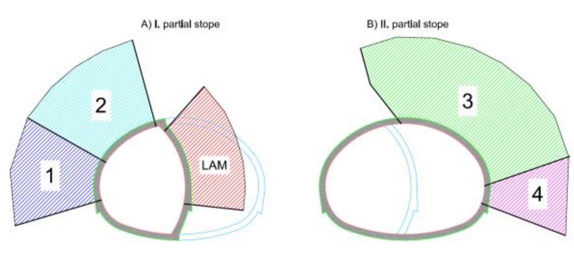

Figure 15 – Bolting regions

For shallow tunnels, the values of β can range between 0,2-0,5. Generally used values are 0,35-0,65. The determination of the β value highly depends on the particular tunnel geological conditions and also technological specifics (especially unsupported length of the tunnel). Due to rather complicated geological and geotechnical conditions factor β_d = 0,3 was estimated for both profiles. The further progression was done in two steps. Increase to β = 0,6 was modelled for the young sprayed concrete and then increase to β = 1 was assigned to the mature sprayed concrete. Basically, the unsupported stope was loaded by 30% of the total loading and the remaining 70% was applied in two steps (30% and 40%) after the support was constructed.

5.9 Finite Element Mesh Generation

The finite element mesh generation is one of the most important steps of the model setup. It directly affects the result quality and accuracy and also the computing time and the computer resources needed for the calculation. Ideally, the finer the mesh, the more accurate the results, but the longer the time of calculation. Generally, the modeller must reach the equilibrium between the two. Experience is essential.

In the GEO 5 – FEM-Tunnel program the mesh generation is governed by three main features; the element side length, the point refinements and the line refinements.

The general element side length was established to 2,0m. And number of refinements was done around points of interest and along the lines of tunnel outline.

5.10 Construction Stages

The modelling of construction stages corresponds to the construction process and also inhabits the time-dependent behaviour of the sprayed concrete and the stress redistribution in the soil due to face advancement away from the modelled profile. The construction stages are described below.

Stage 1 - Primary stress state calculation

Stage 2 - I. partial stope – top heading and bench excavation,
Convergence-confinement factor β=0,3

Stage 3 - I. partial stope – application of primary lining to the top heading and bench
(young sprayed concrete – E=15,5GPa, G=6,45GPa),
Rock bolting (region 1,2 and LAM)
Convergence-confinement factor β=0,6

Stage 4 - I. partial stope – maturing of sprayed concrete (E=18,0 GPa, G=7,5 GPa),
Convergence-confinement factor β=1,0

Stage 5 - Change of elephant's feet geometry (see section 7.5),
Redistribution of internal forces

Stage 6 - I. partial stope – invert excavation,
Convergence-confinement factor β=0,3

Stage 7 - I. partial stope – application of primary lining to the invert
(young sprayed concrete – E=15,5GPa, G=6,45GPa),
Convergence-confinement factor β=0,6

Stage 8 - I. partial stope – maturing of sprayed concrete (E=18,0 GPa, G=7,5 GPa),
Convergence-confinement factor β=1,0

Stage 9 - II. partial stope – top heading and bench excavation,
Convergence-confinement factor β=0,3

Stage 10 - II. partial stope – application of primary lining to the top heading and bench
(young sprayed concrete – E=15,5GPa, G=6,45GPa),
Rock bolting (regions 3 and 4)
Convergence-confinement factor β=0,6

Stage 11 - II. partial stope – maturing of sprayed concrete (E=18,0 GPa, G=7,5 GPa),
Convergence-confinement factor β=1,0

Stage 12 - Demolition of temporary support,
Change of elephant's feet geometry (see section 5.5),
Redistribution of internal forces

Stage 13 - II. partial stope – invert excavation,
Convergence-confinement factor β=0,3

Stage 14 - II. partial stope – application of primary lining to the invert
(young sprayed concrete – E=15,5GPa, G=6,45GPa),
Convergence-confinement factor β=0,6

Stage 15 - II. partial stope – maturing of sprayed concrete (E=18,0 GPa, G=7,5 GPa),
Convergence-confinement factor β=1,0

6 Calculation Results

The following sections contain the results obtained from the numerical models developed in the GEO 5 – FEM-Tunnel program according to the principles described in the previous chapters. The results are presented as graphical output of the program.

The results are considered after I. partial stope is finished (after construction stage 8) and after the whole construction is finished (after construction stage 15

The program allows variation of result presentation tools, for this work, the distribution of internal forces along the designed beams is of the most importance. An example of moment distribution after construction stage 8 is presented in Figure 16.

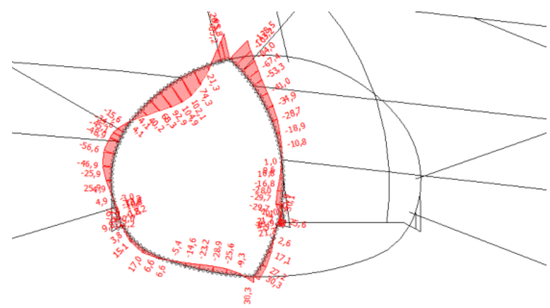

Figure 16 – Output example

7 Primary Lining Design and Verification

The design of the primary lining arises from the lining thickness design, the data obtained from the numerical modelling in the form of the cross-section loading and from the characteristics of the building materials used for its construction.

The design and verification were done in the FIN EC – Beton 2D program according to the ČSN EN 1992-1-1 *Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings*. A cross section of the same height as the primary lining thickness and the width 1,0m and made of the same material was designed for the combination of bending moment and normal force obtained from the numerical modelling.

7.1 Material characteristics

The materials used for the primary lining are sprayed concrete SB 25 (C20/25) and reinforcing steel B500. The detailed material characteristics used in the design are presented in Table 5.

Table 5 – Building materials design characteristics

Material characteristic			SB 25 (C20/25)	B500
Modulus of elasticity		E [GPa]	18	200
Shear modulus		G [GPa]	7,5	81
Compressive strength [MPa]	Characteristic value	f_{ck} [MPa]	20	-
	Design value	f_{cd} [MPa]	**13,33**	-
Tensile strength [MPa]	Characteristic value	f_{ctk} [MPa]	2,2	-

	Design value	f_{ctd} [MPa]	**1,47**	-
Yield strength [MPa]	Characteristic value	f_{yk} [MPa]	-	500
	Design value	f_{yd} [MPa]	-	**434,7**
Safety coefficient [-]		γ_c / γ_s	1,5	1,15

7.2 Primary lining loading

The primary lining is designed for the combination of bending moment and normal force. The extreme values of both have to be therefore identified and the corresponding values of the other described. That means to find the extreme bending moment (both negative and positive) and the normal force corresponding to this bending moment. The same is done for the extreme values of normal force. The normal force is mostly negative along the beam and the positive values are very small, however, these are also taken into account.

The design values are listed in Table 6.

Table 6 – Design load combinations

	Design combination	N [kN/m]	M [kNm/m]
I. partial stope	maximum positive bending moment	**-243,7**	104,9
	maximum negative bending moment	**-261,0**	-128,5
	maximum negative normal force	-379,0	**-34,9**
	maximum positive normal force	6,1	**-28,9**
Profile DTA 80,0m II. partial stope	maximum positive bending moment	**-493,2**	126,8
	maximum negative bending moment	**-710,4**	-156,0
	maximum negative normal force	-728,2	**-126,7**

7.3 Primary lining design

The designed primary lining is made of sprayed concrete, steel meshes along both surfaces and reinforcing steel arches. Also the rock bolts are considered part of the primary lining but their effect was introduced into the modelling as soil body improvement.

The primary lining design for this profile is:

- Sprayed concrete SB 25 (C20/25) of the thickness 400 mm,
- Reinforcing steel mesh 9/100 x 9/100 (Figure 17a) along both inside and outside surface, type 60 9003 E (AQ 90) produced by Železárny Annahütte spol. s r.o.
- Reinforcing steel arch RT5 318 mm high (Figure 17b) at the axial distance 1,0m

The designed reinforcement cover is 22mm at the outside and 24mm at the inside (the distance from rebar axis to the concrete surface being 26,5mm) to allow for assembly tolerances.

7.4 Primary lining design verification

The primary lining design verification was done in the form of interaction diagram where the design loading combinations were plotted and verified. The interaction diagram was developed in the FIN EC – Beton 2D program according to ČSN EN 1992-1-1.

The cross-section used in the verification consists only of the sprayed concrete and the steel reinforcing mesh. The reinforcing steel arches are not considered in the calculation; it is one of possible design approaches and it is on the safe side. The diagram is presented in Figure 18.

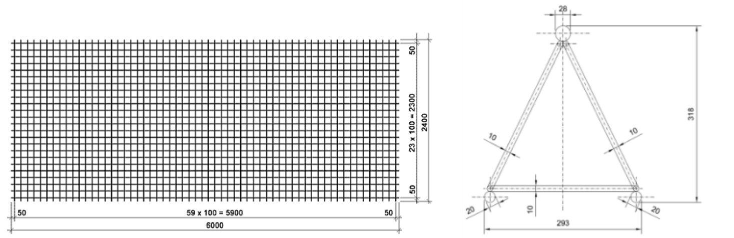

Figure 17 – Profile DTA 80,0m – Reinforcing elements (Annahütte, Metroprojekt)
a)steel mesh 9/100 x 9/100 b)steel arch RT5 318 mm high

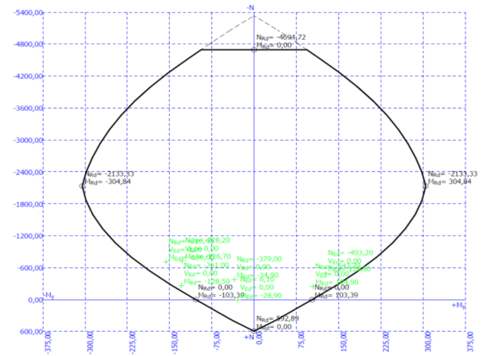

Figure 18 – Interaction diagram

The primary lining design is satisfactory; the maximal cross-section utilization is 88,6% for the combination of maximum negative bending moment and corresponding normal force in the I. partial stope. The same combination in the II.partial stope results in the utilization of 75,2%.

8 Conclusion

The main aim was to design the primary lining of double track metro tunnel which is to be built as a part of the extension of the Prague metro line A.

The designed profile is in the Petřiny – Motol route segment. The geological and geotechnical conditions were determined from the engineering geological survey. Then two-dimensional numerical model was developed to simulate rock-primary lining interaction during the entire construction process. The numerical analysis was carried out in the GEO FEM-Tunnel program which uses the finite element method for the calculation and is disposed for solving underground construction.

The stresses and deformations of the soil body, the internal forces in the primary lining and the surface subsidence above the constructed tunnel were obtained from the numerical analysis and the primary lining design was carried out based on them. The primary lining thickness and reinforcement were designed for the extreme combinations of bending moment and normal force. The cross-section corresponding to 1m wide section of the primary lining was then verified in the form of interaction diagram in the FIN EC – Beton 2D program according to the Eurocode 2.

The designed primary lining is 400mm thick section of sprayed concrete SB 25 (C20/25) with the reinforcement of 9/100 x 9/100 steel mesh along both surfaces and steel reinforcing arch RT 5 318mm high. The cross-section utilization is 89%. The verification was done without accounting for the steel reinforcing arches; this being on the save side. The rock bolting used in the respective profiles was not considered during the verification, because the influence of the rock bolts was entered into the models.

In conclusion, the author has reached the main objective – to design the primary lining of the double-track metro tunnel successfully. The contribution to the engineering practice is mainly in the prediction of the soil body and the soil – primary lining interaction during the boring operations in the tunnel. However, the modelling usually arises from imperfect and sometimes incomplete data and includes a number of simplifications which has to be taken into account and the modelled behaviour has to be confronted with the geotechnical monitoring. The data observed on the real structure may then be used for the calibration of the model for further understanding of the environment and possible more efficient design in similar conditions.

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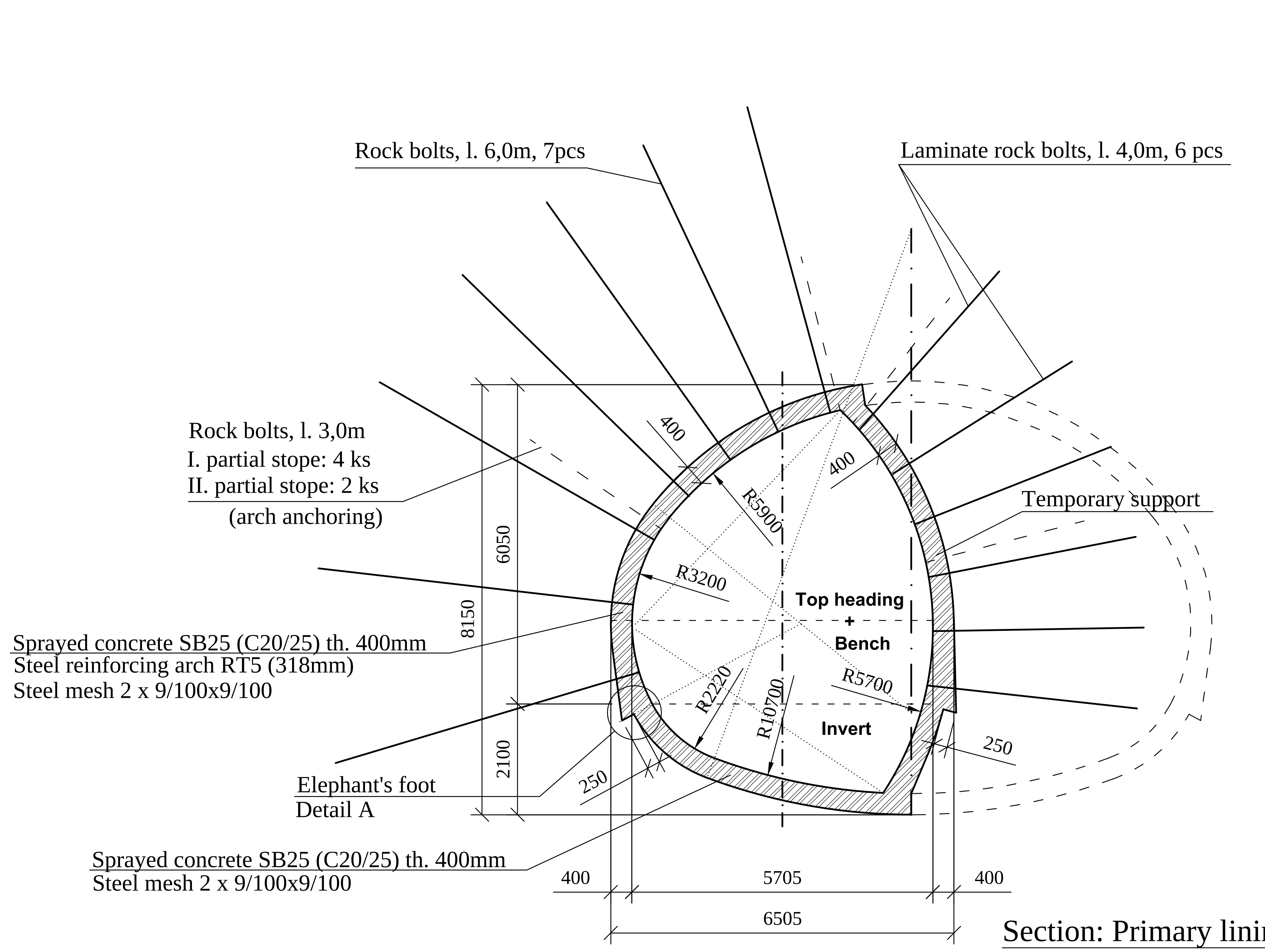
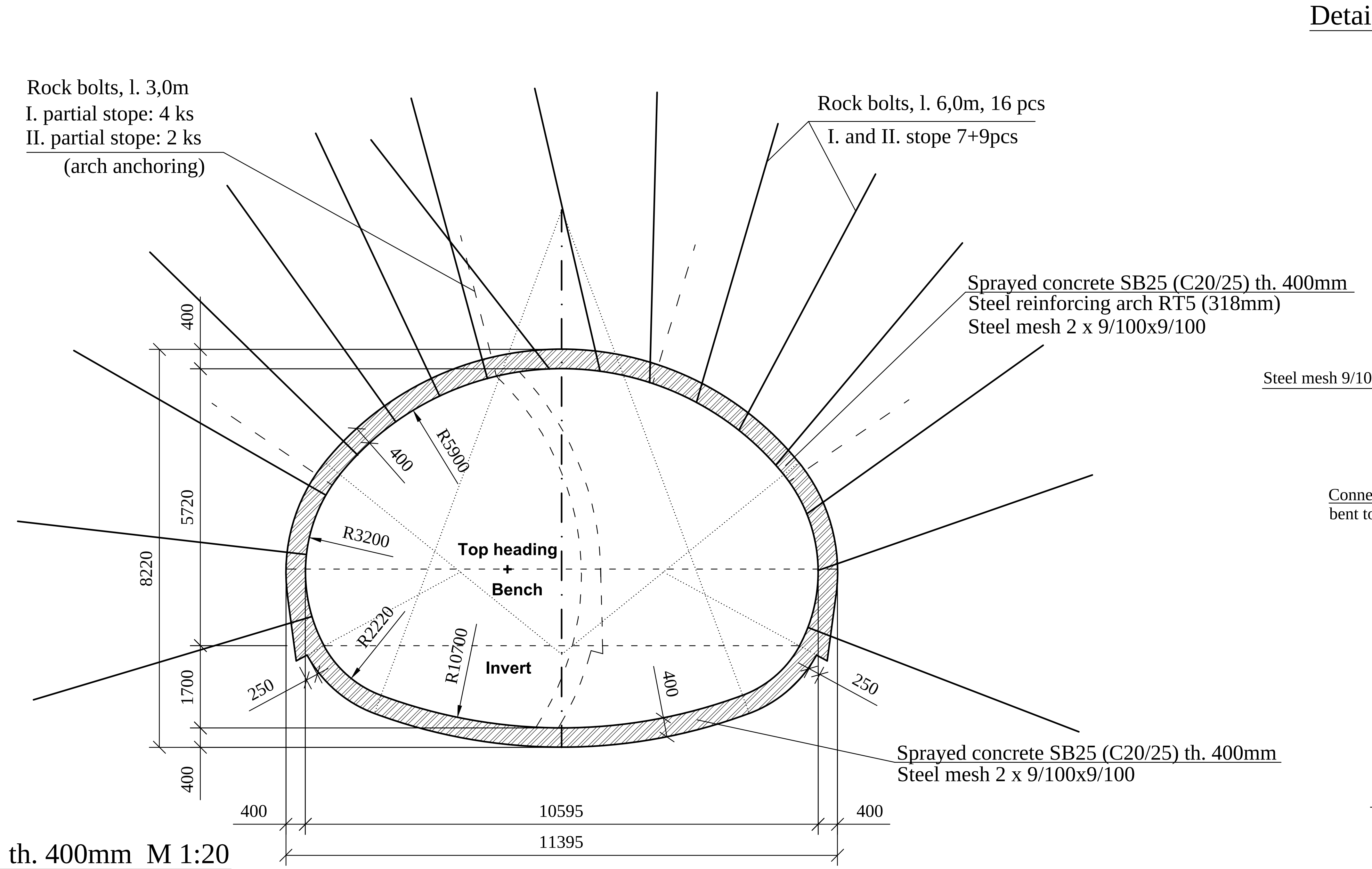
Appendix A

Profile DTA 80,0m – Primary lining design

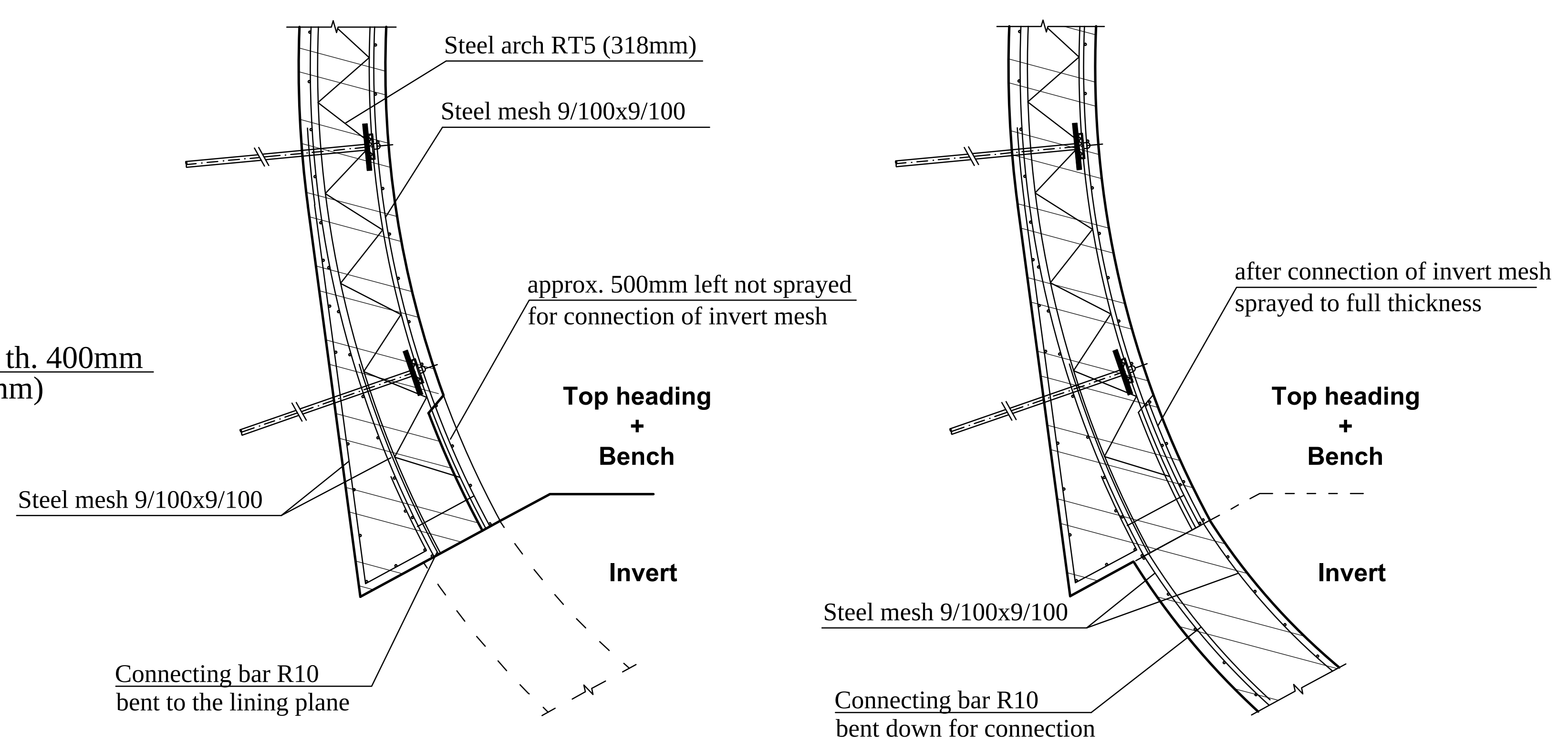
Primary lining section - I.partial stope M 1:100



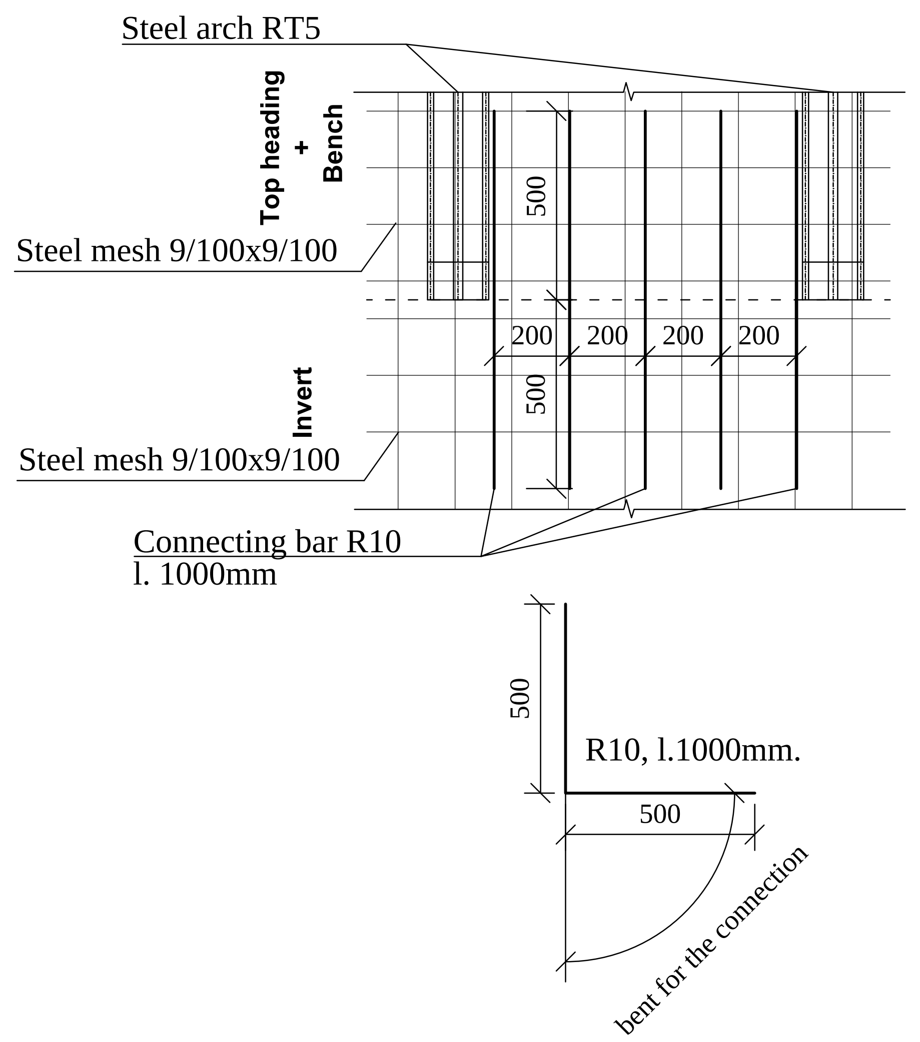
Primary lining section - II.partial stope M 1:100



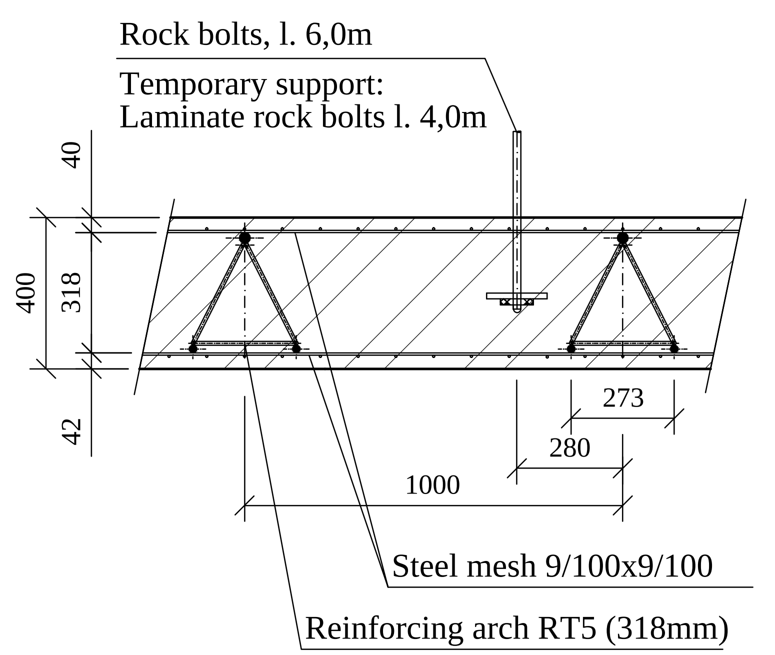
Detail A: Mesh connection between the bench and invert M 1:25



Detail A: View M 1:20



Section: Primary lining, th. 400mm M 1:20



Sprayed concrete SB 25 (C20/25)
Reinforcing steel S500, cover min. 20 mm

	Czech Technical University in Prague Faculty of Civil Engineering Department of Geotechnics		
Designed by: Jiří Šach			Format: 7 x A4
Work advisor: Prof. Ing. Jiří Barták, DrSc.		Academic year: 2011/12	Date: IV/2012
Work title: Design of the primary lining of double-track metro tunnel			Scale: 1:100, 1:20
Drawing title: Profile DTA 80,0m - Primary lining design			Drawing number: appx. A