KORISNI KONCEPTI U PRIMENI NOVE AUSTRIJSKE METODE ZA GRADNJU TUNELA (NATM)

USEFUL CONCEPTS FOR APPLICATION OF NEW AUSTRIAN TUNNELLING METHOD IN TUNNEL CONSTRUCTION (NATM)

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PREGLEDNI RAD
REVIEW PAPER
UDK: 624.191.1(436)
doi: 10.5937/grmk1504021J

UVOD

Tunele poznajemo kao podzemne prostore koji služe za transport ljudi i/ili materijala. Tunelske konstrukcije su rezultat specifične podzemne građevinske aktivnosti; izvode se iskopom stenske mase ili tla, pri čemu se statička ravnoteža obezbeđuje pomoću interakcije tunelske obloge i stenske mase odnosno tla oko tunela. Potporne mere za postizanje ravnoteže pri iskopu (primarna podgrada) obuhvataju tunelsku oblogu i različite oblike ojačanja stenske mase ili tla. Tuneli po pravilu imaju male dimenzije poprečnog preseka u odnosu na svoju dužinu, pri čemu niveleta tunela, osim u primeru hidrotehničkih tunela, ne odstupa bitnije od horizontalne.

Upotreba NATM metode predstavljena je za tunele na autoputevima, koji imaju specifične dimenzije i zahteve u pogledu oblika. Na slici 1 prikazan je tipičan karakteristični poprečni presek autoputnog tunela bez podnožnog svoda. Dimenzije autoputnog tunela prilagođene su svetlom profilu približnih dimenzija 9,0 m x 5,5 m, pri čemu je visina tunela nad kotom nivelete približno 7 m. Ukupna površina iskopa ovakvog tunela kreće se od 85 m² za tunel bez podnožnog svoda do 100 m² za tunel s podnožnim svodom. Zaključci o primeni NATM metode u ovom članku primenjivi su na sve tunele pribli-

INTRODUCTION

Tunnels are universally known as underground structures used for the transport of people and/or materials. Tunnel structures are the result of the specific underground engineering activity; they are excavated in the rock or soil mass and equilibrium is achieved within the interaction of tunnel lining and the rock mass or soil around the tunnel cavity. The support measures needed to reach the equilibrium during the excavation (primary support) encompass tunnel lining and the different support elements of rock mass or soil reinforcement. Tunnels are by rule elongated structures in which the vertical alignment, except in the case of hydro-technical tunnels, does not significantly deviate from horizontality.

The use of NATM is presented in the paper for the motorway tunnels, which have specific dimensions and the conditions with regard to their shape. The typical characteristic cross section of a two-lane motorway tunnel without invert is shown in Figure 1. The dimensions for the motorway tunnel are adapted to the required "profile of light" of approximate dimensions of 9,0m x 5,5m, wherein the free height of the tunnel is some 7m above the roadway. The total excavation surface of this tunnel is some 85m² if without the invert and some 100 m² for a tunnel with the invert. The con-

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žnih dimenzija kao što su dimenzije tunela prikazanog na slici 1.

clusions about the application of the NATM method given in this paper are applicable to all tunnels of the approximate dimensions as given in Figure 1.

Slika 1. Karakteristični poprečni presek autoputnog tunela
Figure 1. Characteristic cross section of a highway tunnel

Tuneli na auto-putevima obično su kraći od dužine koja je rentabilna za gradnju s TBM strojem. Stoga, skoro u svim slučajevima gradnje tunela u regionu u posljednje dve decenije bila je izabrana NATM metoda odnosno SCL metoda (Sprayed Concrete Lining). Metoda SCL, to jest metoda upotrebe mlaznog betona za primarnu oblogu tunela, svakako je tehničko primerenije ime nego NATM. U literaturi se često nailazi na kritični odnos do uopštene upotrebe imena NATM, jer je mlazni beton najverovatnije bio prvi put primenjen za oblogu tunela u Australiji u tridesetim godinama prošlog veka. U našem regionu je izraz NATM odomaćen jer je shvaćen u širem smislu i zbog toga će biti korišćen u nastavku.

Osnovne koncepte NATM metode posle Drugog svetskog rata postavili su austrijski inženjeri Rabcewicz, Müller i Pacher (Golser, 1976), pri čemu je zanimljivo to što je metoda dobila svoje prve aplikacije tek u šezdesetim godinama prošlog veka u Venecueli. Metoda se pokazala kao ekonomski veoma efikasna, jer je osnovna ideja iskorišćavanje samonosivosti stenske mase. To uzrokuje manju potrebu za ugrađivanjem elemenata primarne podgrade i samim tim - ekonomičniju gradnju.

Very often, the motorway tunnels are shorter than what would be the economical tunnel length for the use of Tunnel Boring Machine (TBM). This was the reason that all the tunnels that were built in the region in the last two decades were constructed using the NATM method. This method is also known as the SCL (Sprayed Concrete Lining) method, which is technically more accurate use of terminology implying the utilisation of the shotcrete as the material for the tunnel primary lining. There is a lot of criticism for the overall use of the term NATM found in the literature as the first use of the shotcrete for the tunnel lining probably happened in the nineteen thirties in Australia. However, the term NATM is commonly used in our region in a wider sense and so it is preferred term used also in the continuation of this paper.

The basic concepts of the NATM method were laid out by Austrian engineers Rabcewicz, Müller and Pacher (Golser, 1976). It is interesting to note that the first application of this method was carried out in Venezuela in the nineteen sixties. The method was proven to be economically very efficient as the basic philosophy is based on the use the self-capacity of the rock mass, which reduces the need for the embedment of the man-

The typical elements of the primary lining in NATM are: a) the shotcrete lining and b) the rock bolts, which are installed radially. In the poorer geological conditions the shotcrete lining is reinforced with steel meshes and steel sets are used for the temporary support. Alternatively, in certain geological conditions a microfiber shotcrete can be used for the primary lining (Jovićić et al., 2009). The supporting elements in longitudinal direction typically used for stabilisation of the head of excavation comprise fibre-glass anchors and/or pipe roof, as appropriate. The typical elements of primary support in NATM are shown in Figure 2 including their layout in the plane of the tunnel.

NATM možemo posmatrati kao filozofiju projektovanja tunela, zasnovanu na sledećim konceptima: a) nosivost stenske mase oko otvora tunela mobilise se do maksimalno mogućeg stepena; b) to se postiže kontrolisanim praćenjem radijalnih deformacija u tunelu i drugih pokazatelja stabilnosti iskopa; c) elementi primarne podgrade ugrađuju se u skladu s geološkim uslovima gradnje; d) napredovanje iskopa tunela sinhronizira se s vremenskom komponentom razvoja radijalnih deformacija.

U smislu metode izgradnje, za NATM je karakteristično sledeće: a) otvor tunela kopa se u sekvencama i obično je u poprečnom smeru podeljen na kalotu, stepenicu i podnožni svod (vidi sliku 2); b) sekvenca iskopa u uzdužnom smeru prilagođena je geološkim uslovima gradnje, pri čemu je ključni faktor prelaz iz trodimenzionalnih graničnih uslova - 3D (blizina čela iskopa) na dvodimenzionalne - 2D (udaljeno čelo iskopa); c) izbor i frekvencija ugradnje potpornih elemenata prilagođeni su geološkim uslovima gradnje. U nastavku su prikazane teorijske osnove metode NATM, koje čine osnovu za njenu adekvatnu primenu u praksi.

TEORIJSKE OSNOVE ZA NATM

Metoda Konvergencije-Relaksacije

Ciljevi rešavanja naponsko-deformacijskog problema tunnelske konstrukcije jesu dokaz njegove stabilnosti i određivanje naponsko-deformacijskog odziva stenske mase. To se dobija na osnovu izračunavanja stanja napona, deformacija i pomeranja u stenskoj masi i primarnoj podgradi tunela koji odgovara konačnom ravnotežnom stanju koje zadovoljava uslove trajnosti.

Načelno, iskop tunela prolazi različite uslove gradnje, tako da sistem primarne podgrade mora tome biti prilagođen. Svrha statičkih proračuna jeste da dokažu stabilnost iskopa tunela u različitim geološkim uslovima i s različitim visinama nadsloja. Pri proračunu, treba imati u vidu to da iskop tunela izaziva relaksaciju početnih napona u stenskoj masi koja se zatim preraspodeli u interakciji između same stenske mase i podgrade. Ključni elementi podgrade tunela jesu pasivna stenska sidra (rock-bolts), koja se ugrađuju radijalno i koja obavljaju funkciju prenosa uticaja rasterećenja i omogućavaju mobilizaciju stenske mase u blizini otvora i po dubini.

U periodu građenja, naponsko-deformacijska analiza iskopa tunela trodimenzionalni (3D) je granični problem. Tada se javlja uticaj čela iskopa, na osnovu koga stenska masa u uzdužnom smeru ispred tunela preuzima i deo radijalnih opterećenja. Kada se čelo iskopa udalji dovoljno daleko od posmatranog poprečnog preseka, granični problem prelazi iz trodimenzionalnih (3D) uslova u dvodimenzionalne (2D) uslove ravnog stanja deformacije. Zbog toga se proračuni stabilnosti tunela u rutinskom projektovanju načelno rade primenom 2D modela za konačno stanje, pri čemu se posredno uvode 3D efekti - kao simulacija

The NATM can be understood as a design philosophy, which is based on the following concepts: a) the capacity of the rock-mass is mobilised to the highest degree, b) this is achieved by controlled monitoring of the convergence (radial) deformations and the other indicators during the excavation of the tunnel, c) the elements of primary support are installed with respect to the current geological conditions, d) the progress of the excavation and the primary support of the tunnel is synchronised with the time component of the development of the radial deformations.

From the point of view of the construction method, the NATM concepts are as follows: a) the tunnel cavity is excavated in sentences so that the cross section is usually divided into the top, the bench and the invert (see Figure 2), b) the sequence of excavation in longitudinal section is adapted to the geological conditions, wherein the key factor becomes the transition from 3D boundary conditions (near to the head of excavation) to the 2D boundary conditions (far from the head of excavation) and c) the selection and the frequency of the installation of the elements of the primary support are chosen according to the current geological conditions. The theoretical background of the NATM, which is the basis for its utilisation in practise, is shown in continuation.

THEORETICAL BACKGROUND OF NATM

Convergence – Confinement method

The aim of solving the stress-deformation problem of tunnel structure is the proof of stability and control of deformations. This is done by resolving the state of stresses, deformations and movements in the rock mass and the primary support of the tunnel for a particular cross section. The numerical analysis is carried out for the final equilibrium stage of the construction considering the long term performance and sustainability of tunnel structure.

In general, the excavation of the tunnel is traversing through different building conditions so that the primary support system must be fully adapted, as appropriate. The purpose of the static calculations is to prove the stability of tunnel excavation for different geological conditions and for different amounts of overburden. The excavation of the tunnel is causing the relaxation of the initial ground stresses, which is in turn redistributed within the interaction between the rock mass and the tunnel primary support. The key elements of the primary support are the rock bolts, which are installed radially, perpendicular to the direction of the excavation. The rock bolts are enabling the mobilisation of the rock mass in the vicinity of the cavity and enable the transfer of the relaxation load deeper into the rock mass of better capacity.

The stress-strain deformation analyses is a 3D (three dimensional) boundary value problem only in the time of tunnel excavation. During that time there is the presence of the head of excavation so that the load is taken by rock mass also in longitudinal direction ahead of the tunnel. When the head of excavation is moved further from the observed cross section, the boundary value

procesa gradnje.

Načelno, u okolini tunela razlikujemo tri naponska stanja odnosno tri zone (slika br. 3):

- Stanje pre iskopa

Zona 1 - Netaknuta stenska masa. Uticaj prirodno napregnute stenske mase u unutrašnjosti budućeg tunela statički je ekvivalentan srednjem početnom naponu ($p = p_0$)

- 3D stanje u kome se oseća uticaj blizine čela iskopa

Zona 2 - Čelo iskopa. Stanje posle iskopa i pre ugradnje podgrade, kontaktni napon na konturi jednak je nuli ($p = 0$), deo opterećenja preuzima stenska masa ispred čela iskopa, granični problem je trodimenzionalan.

- 2D stanje u kome se ne oseća uticaj blizine čela iskopa

Zona 3 - Stabilizovan iskop. Stanje posle ugradnje podgrade je takvo da je uticaj podgrade statički ekvivalentan kontaktnom naponu na konturi - p ; granični problem je dvodimenzionalan.

problem is transferred from 3D conditions to 2D (two-dimensional) conditions of plane strain. This is the reason that the static calculations of tunnel stability are routinely carried out in 2D conditions, and the 3D effects and boundary conditions are taken indirectly into the account.

In this context, we differentiate three different stress conditions around the tunnel, that is three different zones shown in Figure 3:

- State before the excavation

Zone 1 - Undisturbed Rock Mass. The influence of naturally stressed rock mass in the interior of future tunnel, which is equal to the in-situ mean effective stress, ($p = p_0$).

- 3D state in which the proximity of the head of the excavation is felt

Zone 2 - Head of excavation. For the stress state after the excavation and before the installation of primary support the contact stress on the contour is equal zero ($p = 0$), part of the loading is taken by the rock mass in front of the head of excavation, the boundary value problem is three-dimensional.

- 2D state in which the proximity of the head of excavation is not felt

Zone 3 - Stable excavation. State of excavation after installation of primary support is established so that influence of the support is statically equivalent to the contact stress on the contour, the boundary value problem is 2D.

Slika 3. Karakteristične zone stanja napona u toku etapnog iskopa tunela
Figure 3. Characteristic stress state zones during the tunnel excavation

U savremenoj inženjerskoj praksi, granični problem interakcije stenske mase i podgrade tunela rešava se numeričkim putem i to najčešće primenom metode konačnih elemenata. Za dobijanje opšte slike o stabilnosti tunela i određivanja primerenih ulaznih podataka za numeričke analize potrebno je prethodno proveriti jednostavnije analitičke postupke koji imaju potvrđene teorijske osnove. Jedna od najkorisnijih metoda za tu svrhu jeste takozvana metoda Konvergen-

In contemporary engineering practice, solving boundary value problem of interaction of the rock mass and the tunnel support is usually performed by numerical methods; in particular the Finite Element Method is used most frequently. For the general picture on tunnel stability and for selection of appropriate input parameters it is necessary to check more simple analytical procedures, which have confirmed theoretical background. One of the most useful methods for this purpose is the

cije-Relaksacije (Convergence-Confinement Method), koja će ovde biti objašnjena u kratkim crtama.

Metoda Konvergencije-Relaksacije ima dvojni svrhu kada se koristi u sprezi s numeričkim analizama i to: a) za ocenu globalne stabilnosti tunela na pojedinačnim odsecima i b) za određivanje ulaznih parametara za numeričke analize, odnosno za posredno uvođenje 3D uticaja u 2D analizu.

Za rešavanje problema stabilnosti po metodi Konvergencija-Relaksacija potrebno je istovremeno analizirati: a) naprezanje na konturi tunela koje je rezultat relaksacije napona izazvanog iskopom tunela (Relaksacija) i b) radijalne deformacije i pomeranja konture tunelskog iskopa (Konvergencija). Osnovne proračunske pretpostavke metode Konvergencija-Relaksacija jesu sledeće: a) stenska masa se tretira kao izotropni kontinuum; b) granični problem je 2D; c) nadsoj je visok, odnosno tunel je dubok ($H/D > 4$), gde je H visina nadsloja, a D je prečnik tunela; d) tunel je kružnog oblika (R je radijus tunela); e) važi ravno stanje deformacija nakon uspostavljanja 2D graničnih uslova; f) početno naponsko stanje je za $k_0 = \sigma_h / \sigma_v = 1$; (σ_h – horizontalni napon je jednak σ_v – vertikalnom naponu). Iako prikazane pretpostavke predstavljaju umnogome idealizovane uslove koji se ne mogu naći u realnim tunelima, smatra se da su za potrebe određivanja globalne stabilnosti pri projektovanju autoputnih tunela načelno prihvatljive (autoputni tuneli s podnožnim svodom su jajasto-kružnog oblika s približnim poluprečnikom $R=5,5\text{m}$; naponsko stanje u stenskoj masi, usled delovanja tektonskih sila, verovatno je blizu $k_0=1$; stenska masa se može posmatrati kao kontinuum ukoliko uticaj diskontinuiteta nije dominantan, i dr.).

U smislu razvoja deformacija, ponašanje stenske mase u blizini iskopa tunela, opisano karakterističnom krivom relaksacije, prikazano je na slici 4.a. Usled relaksacije, naponi u stenskoj masi opadaju i javljaju se pomeranja na konturi. Naponi opadaju od vrednosti početnih napona σ_0 do vrednosti 0, pri čemu se stenska masa odziva elastično do vrednosti $p_i \geq p_{cr}$ odnosno plastično u slučaju da je relaksacija veća i da je $p_i < p_{cr}$, pri čemu je p srednja vrednost glavnih napona u stenskoj masi, p_i – stepen relaksacije napona i p_{cr} – granična vrednost.

Ponašanje podgrade definisano je karakterističnom krivom otpora podgrade (slika 4.b), pri čemu je uvedena pretpostavka njenog elastično-idealno plastičnog ponašanja. To znači da je u interakciji sa stenskom masom odziv podgrade, koja je instalirana pri pomeranju u_{in} , na početku linearno elastičan (koeficijent podgrade k_s), a zatim pri dostizanju nosivosti odnosno maksimalne vrednosti p_{max} idealno plastičan.

Ravnoteža interakcije između primarne podgrade i stenske mase uspostavlja se kada karakteristična kriva podgrade preseče karakterističnu krivu stenske mase (slika 5) u tački s koordinatama p_{eq} , u_{eq} , pri čemu je p_{eq} ekvivalentni napon i u_{eq} – odgovarajuće ekvivalentno pomeranje pri kome je uspostavljena ravnoteža.

Convergence-Confinement Method, which will be explained here in short form.

Convergence-Confinement Method has a double purpose in connection with the more advanced numerical analyses. It is used for: a) evaluation of global stability of the tunnels at particular sections and b) determination of input parameters for numerical analyses, in particular for the gradual integration of 3D effects into 2D analyses.

To provide with the solution using the Convergence-Confinement Method it is necessary to analyse in parallel: a) stresses on the contour of the tunnel, which are the result of the stress relaxation caused by excavation of the tunnel (Relaxation) and b) movements of the contour of the tunnel excavation caused by relaxation (Convergence). The main calculation assumptions of the Convergence-Confinement Method are: a) the rock mass is an isotropic continuum, b) the boundary value problem is 2D, c) the tunnel is deep ($H/D > 4$), H is the height of overburden, D is the diameter of the tunnel d) the tunnel contour is of circular shape, R is tunnel radius, e) the plane strain deformation state is established for 2D boundary conditions and f) the in situ stress state is hydrostatic, that is $k_0 = \sigma_h / \sigma_v = 1$, where σ_h and σ_v are in situ horizontal and vertical stress, respectively. The given assumptions are idealisation of real circumstances. However, they are considered accurate enough for evaluation of global stability of a motorway tunnel (motorway tunnels with the invert are egg shaped but close to approximate circle with the radius of $R=5,5\text{m}$; the stress state in the rock mass is affected by the tectonic forces and is likely to be close to hydrostatic state with $k_0=1$, the rock mass can be regarded as continuum, if the discontinuities are relatively small to the dimensions of the tunnel, which is very often, etc)

In terms of the deformation pattern the behaviour of the rock mass in the vicinity of the tunnel is described by the characteristic relaxation curve of the rock mass, which is shown in Figure 4a. The movements on the tunnel contour u are caused by relaxation of the stresses due to the excavation of the tunnel cavity. The stresses in the rock mass are decreasing from the in-situ values σ_0 up to zero and the response of the rock mass is elastic within the region $p_i > p_{cr}$ and plastic if the relaxation becomes larger than $p_i \leq p_{cr}$, where p_i is the amount of relaxed stress and p_{cr} is the boundary value.

The behaviour of the lining is defined with the characteristic curve of the lining resistance (Figure 4b), considering the assumptions that the behaviour is elastic-plastic. This means that the interaction between the rock mass and the lining (which is installed at the convergence movement u_{in}) results in the firstly elastic response (the coefficient of lining stiffness response k_s) and secondly, after the capacity of the lining is reached at p_{max} , in the ideally plastic response.

The equilibrium of interaction between the primary lining and the rock mass is established when the characteristic curve of the lining intersects relaxation curve of the rock mass (Figure 5) in the point with the coordinates p_{eq} , u_{eq} , where p_{eq} is the equilibrium stress and the u_{eq} the equilibrium movement. The aim of the NATM method is to use most of the capacity of the rock mass and in the same time that there is enough reserve in the capacity of lining, as schematically presented in Figure 5.

Slika 4. Karakteristične krive odziva: a) stenske mase i b) podgrade na iskop tunela
Figure 4. Characteristic curves of: a) rock mass and b) tunnel support response to tunnel excavation

Slika 5. Tačka preseka krive stenske mase i krive otpora podgrade
Figure 5. Point of intersection for curves of rock mass and tunnel support

U smislu iskorišćenja nosivosti stenske mase, od ključnog značaja je dostizanje tačke ravnotežnog stanja, što je posledica interakcije stenske mase i primarne podgrade. U slučaju da se podgrada ugradi prekasno, može doći do tako velikih deformacija da se progresivni lom stenske mase ne može više sprečiti. Takođe, ako nosivost podgrade nije zadovoljavajuća, podgrada može dostići granicu izdržljivosti pre dostizanja ravnoteže. U oba slučaja, podgradna konstrukcija neće imati efekta, jer nije uspostavljeno ravnotežno stanje, te dolazi do rušenja u tunelu.

Trenutak postavljanja podgrade određuje se na osnovu uslova stabilnosti nepodgrađene stenske mase i tehnološkog ograničenja deformacije konture, s ciljem da ne dođe do nastanka potprofila. Inženjerski gledano, stanje konačne ravnoteže mora imati određeni faktor sigurnosti - kako u odnosu na nosivost podgrade, tako i u odnosu na nosivost stenske mase.

In the terms of utilisation of the capacity of rock mass it is of key importance to achieve equilibrium on the optimal place, as a result of preferred interaction of the rock mass and the lining. If the lining is installed too late for a given schedule of excavation (i.e. too far from the head of excavation), there would be too much deformation of the rock mass and the progressive failure might occur. Also, if the lining is of insufficient capacity, it can collapse before the equilibrium point is ever reached. In both cases the supporting structure is without effect, as equilibrium will not be reached and the failure of the rock mass can occur.

The moment of the installation of the tunnel lining and the supporting elements is defined on the conditions of the stability of the unsupported tunnel and the technological limit of the amount of deformation for the tunnel contours so that the under-profile does not occur. From the engineering point of view, the state of the final equilibrium should have a certain factor of safety in terms of both the lining and the rock mass capacity.

Dinamika gradnje: prelazak iz 3D graničnih uslova na 2D granične uslove

Cilj postupka metode Konvergencija-Relaksacija jeste određivanje uslova za pravovremenu ugradnju primarne podgrade koja ima zahtevanu nosivost, tako da se dostigne ravnoteža i s time zahtevana stabilnost tunela. Pritom, treba uzeti u obzir da inicijalni pomak stenske mase (u_{in}), u trenutku postavljanja podgrade, zavisi od stepena relaksacije stene (λ), koji je u funkciji transfera s 3D graničnih uslova na 2D granične uslove ravne deformacije. To je direktno povezano sa udaljenošću nepodgrađenog dela tunela od čela iskopa tunela. Pri gradnji tunela taj razmak manifestuje se kao korak u tehnološkoj sekvenci iskopa i primarnog podupiranja tunela.

Panet i Guénot (1982) modelirali su transfer s 3D graničnih uslova na 2D granične uslove ravne deformacije uvodeći fiktivni pritisak σ_r^f koji deluje unutar tunela. Vrednost fiktivnog pritiska određena je iz inicijalnog napona σ_r^o koristeći izraz:

$$\sigma_r^f = (1 - \lambda) \cdot \sigma_r^o \quad (1)$$

$$\lambda = 1 - \frac{p}{p_o} \Rightarrow p = p_o (1 - \lambda) \quad (2)$$

U gornjem izrazu, λ predstavlja koeficijent redukcije napona kojim uvodimo 3D efekte čela iskopa u 2D model proračuna tunela. Na određenoj udaljenosti ispred čela iskopa je $\lambda=0$ (uslovi su potpuno 3D, $\sigma_r^f = \sigma_r^o$, odnosno $p=p_o$), dok se ta vrednost povećava sve do $\lambda=1$ (uslovi su potpuno 2D, $\sigma_r^f = 0$ odnosno $p=0$) na rastojanju od približno dva do tri prečnika tunela iza čela iskopa (slika 6).

Schedule of construction: the transition from 3D to 2D boundary conditions

The aim of the application of Convergence-Confinement Method is the determination of the conditions for timely installation of the primary lining of required capacity, so that the equilibrium can be achieved and with this the required stability of tunnel excavation. In this sense it should be taken into account that the initial movement (u_{in}) depends on the degree of the relaxation of the rock mass (λ). The degree of the relaxation of the rock mass (λ) is the function of transfer from the 3D boundary conditions to the 2D boundary conditions of plane strain, that is, the function of distance from supported part of the tunnel to the advancing head of excavation. This is seen in the technological sequence of tunnel construction as the one step of excavation and the primary support of the tunnel.

Panet and Guénot (1982) modelled the transfer from 3D to 2D boundary conditions by introducing the fictive supporting stress σ_r^f (Figure 6) acting within the tunnel cavity. The value of the fictive supporting stress is derived from the initial stress σ_r^o using the relationships:

in which λ is the coefficient of stress reduction. At a certain distance in front of the head of excavation $\lambda=0$ (the conditions are fully 3D, $\sigma_r^f = \sigma_r^o$, that is $p=p_o$), and this value increases up to $\lambda=1$ (the conditions are fully 2D, $\sigma_r^f = 0$, that is $p=0$). The 2D conditions are realised at the distance that is approximately 2 to 3 times the diameter D of the tunnel.

Slika 6. Model prelaza sa 3D graničnih uslova na 2D granične uslove ravne deformacije (Kavvasas 2004)
Figure 6. Model of transition from 3D boundary conditions to 2D plain strain boundary conditions (Kavvasas, 2004)

Uvođenje prelaza s 3D graničnih uslova na 2D granične uslove stanja ravne deformacije u samom

The introduction of the transfer from 3D to 2D boundary conditions is done in three steps: (1)

postupku numeričke analize sprovodi se u tri faze, tako da se vrši: (1) evaluacija početnog stanja napona; (2) delimično uvođenje opterećenja izazvanog iskopom proporcionalno koeficijentu λ , da bi simulirali deformacije nastale pre podgrađivanja - (modeliranje 3D uslova); (3) aktivacija podgrade i uvođenje preostalog opterećenja proporcionalno vrednosti $(1-\lambda)$ - (prelazak na 2D uslove). Ključnu ulogu pri modeliranju 3D efekta ima koeficijent λ . U nastavku će biti prikazane promjenljive od kojih koeficijent λ zavisi, kao načini i postupci za ocenu njegove vrednosti.

Panet je uzdužni raspored deformacija nepodgrađenog tunela opisao empirijskom kvadratnom funkcijom, koristeći se rezultatima analitičkog rešenja za problem osno-simetričnog modela tunela u elastičnoj steni (slika 7), što je analitički prikazano jednačinom:

$$\frac{u_r}{u_r^{\max}} = 0,25 + 0,75 \cdot \left[1 - \left(\frac{0,75}{0,75 + x/R} \right)^2 \right] \quad (3)$$

pri čemu je u_r konvergentno pomeranje na rastojanju x od čela iskopa, $u_r^{\max}$ maksimalno konvergentno pomeranje za 2D granične uslove ravne deformacije; R je radijus tunela.

evaluation of the in-situ stresses, (2) partial introduction of the load caused by the relaxation of the in-situ stresses in proportion to coefficient λ , so that we simulate deformation occurred before the installation of the support system - (modelling of 3D conditions) and (3) activation of support and introduction of the rest of the relaxation load up to the full relaxation proportional to $(1-\lambda)$ - (transfer to 2D conditions). The key role in modelling 3D effect is proper choice of the coefficient λ . The main variables that influence the value of λ and procedures for their determination will be shown in continuation.

Panet has described longitudinal distribution of the unsupported tunnel with an empirical quadratic function, using the closed form analytical solution for the problem of axi-symmetrical tunnel in elastic continuum (Figure 7) given in the following equation:

$$\frac{u_r}{u_r^{\max}} = \left[1 + \exp\left(\frac{-x/R}{1,10}\right) \right]^{-1,7} \quad (4)$$

In which u_r is the convergence movement at distance x from the head of excavation, $u_r^{\max}$ is the maximum convergence movement for 2D boundary value conditions of plane strain; R is the radius of the tunnel.

Slika 7. Uzdužni raspored radijalnog pomeranja nepodgrađenog tunela (prilagođeno za Hoek-om, 1999)
Figure 7. Longitudinal distribution of radial displacements in unsupported tunnel (adapted after Hoek, 1999)

Na slici 7 takođe je prikazana empirijska kriva kojom je Hoek (1999) opisao uzdužni raspored deformacija nepodgrađenog tunela, koristeći terenske podatke i rezultate svojih numeričkih analiza. Hoek-ova empirijska kriva prikazana u jednačini 4 razlikuje se od Panet-ove, jer je predviđena za upotrebu pri manjoj nosivosti

The empirical curve derived by Hoek (1999), also presented in Figure 7, is given in equation 4. Hoek derived the curve using the in-situ measured data and the results of the back analyses. Hoek empirical curve is different from Panet's curve, and is more appropriate for the use for rock mass of low capacity, that is, in the case

stenske mase, odnosno u slučajevima kada je odziv stenske mase elasto-plastičan.

Iz gornje slike jasno je uočljivo da je odnos između u_r i $u_{r,max}$, odnosno dužina zone uticaja 3D efekta i brzina njegovog opadanja sa udaljavanjem od čela, zavisna od nosivosti stenske mase i nivoa opterećenja koje ona trpi zbog relaksacije napona izazvanog iskopom tunela. Nivo relaksacije napona direktno je proporcionalan visini nadsloja nad tunelom (relaksacija napona je veća za dublje i manja za plić tunele).

PRORAČUN KOEFICIJENTA STABILNOSTI N_s

Pri oceni uslova stabilnosti u tunelu, treba uzeti u obzir da su mogući različiti tipovi rušenja, u zavisnosti od dva vodeća kriterijuma: a) relativne veličine tunela u odnosu na veličinu blokova koji grade stensku masu; b) nosivosti stenske mase u odnosu na stepen rasterećenja, koji je izazvan iskopom tunela. Pri tome su moguća dva karakteristična mehanizma: a) diskretni mehanizam nestabilnosti (nestabilnost blokova); b) mehanizam nestabilnosti praćen razvojem deformacija koje su prethodno prikazane pomoću koncepta Konvergenije-Relaksacije.

Odnos između nosivosti stenske mase i stepena relaksacije jeste ključni faktor stabilnosti iskopa tunela za stenske mase umerene do male nosivosti. Taj odnos opisuje se pomoću parametra koji se imenuje kao faktor stabilnosti tunela N_s .

Faktor stabilnosti tunela N_s povezan je s dubinom na kojoj je pozicioniran tunel i na kvalitet odnosno nosivost stenske mase koja je obično prikazana pomoću njene jednoosne čvrstoće na pritisak. Odnos ove dve veličine - definisan vrednošću N_s - jeste pokazatelj stabilnosti naponsko-deformacijskog stanja u stenskoj masi u toku iskopa tunela. Faktor stabilnosti nepodgrađenog tunela definiše se relacijom:

$$N_s = \frac{2p_o}{\sigma_{cm}} \quad (5)$$

gde je,

$p_o = 1/3(\sigma_v + 2k_0\sigma_h)$ – srednja vrednost napona na dubini tunela;

σ_{cm} - jednoosna čvrstoća stenske mase na pritisak.

Za određivanje vrednosti jednoosne čvrstoće stenske mase na pritisak σ_{cm} koristi se više različitih metoda. Na slici 8 prikazan je postupak (Hoek I dr., 2002) određivanja σ_{cm} pomoću takozvane GSI klasifikacije (Marinos in Hoek, 2001; Marinos in dr, 2011), pri čemu je σ_{ci} jednoosna čvrstoća na pritisak intaktnog uzorka stenske mase, koji se obično dobija iz laboratorijskih rezultata. GSI klasifikacija ima u vidu sve bitne elemente stanja diskontinuiteta u stenskoj masi (npr. veličinu, raspored, uslojenost, frekvencu, orijentaciju, zapunu).

when interaction of the rock mass and the lining lead to the response that is elastic-plastic.

It can be seen in Figure 7 that the relationship between u_r and $u_{r,max}$ indirectly describes the length of the zone of influence of the 3D effect and also the gradient of its decrease with the advance of the head of excavation. This decrease is related to the capacity of the rock mass and the level of loading that is transferred to it due to the stress relaxation caused by the excavation of the tunnel. The amount of stress relaxation is directly proportional to the depth of the tunnel (i.e. the relaxation is smaller for a shallow tunnel).

CALCULATION OF THE STABILITY COEFFICIENT N_s

The conditions of tunnel stability involve possibilities for different types of failure considering two criteria: a) size of the tunnel relative to the size of the blocks that form the rock mass and b) capacity of rock mass relative to the degree of stress relaxation caused by the tunnel excavation. In this sense the two governing mechanisms are possible: a) discrete type of failure mechanism (wedge instability) and b) mechanism of stability that is followed by development of deformation as previously described using the Convergence-Confinement Method.

The relationship between capacity of the rock mass and the degree of stress relaxation is key factor of stability of tunnel excavation. This relationship is described by using the parameter Factor of Tunnel Stability annotated N_s .

N_s is directly related to the height of overburden (H) and the bearing capacity of the rock mass. The ratio between those two values, defined as N_s , is the index indicator of stress-strain state, which can be expected in the rock mass after the excavation of the un-supported tunnel. Factor of Tunnel Stability N_s is defined as:

where,

$p_o = 1/3(\sigma_v + 2k_0\sigma_h)$ - mean effective stress at tunnel depth

σ_{cm} - uniaxial strength of the rock mass domain.

Uniaxial strength of the rock mass domain σ_{cm} can be determined using several different methods. The procedure (Hoek et al., 2002), that is based on the GSI classification (Marinos and Hoek, 2001; Marinos et al., 2011) is shown in Figure 8, in which σ_{ci} is the uniaxial strength of the intact rock mass usually obtained from the laboratory tests. The GSI classification takes into account all the important aspects of the rock mass discontinuities in the observed rock mass domain (size, distribution, layering, frequency, orientation, filling and etc.)

Slika 8. Postupak određivanja vrednosti jednoosne čvrstoće stenske mase na pritisak σ_{cm} na osnovu klasifikacijskog indeksa stenske mase GSI i vrednosti za jednoosnu čvrstoću intaktne stene σ_{ci} (po Hoek i dr., 2002).

Figure 8. The method of determination of uniaxial strength of rock mass σ_{cm} based on the classification index of rock mass GSI and the value of uniaxial strength of intact rock mass σ_{ci} (after Hoek et.al, 2002)

Ukoliko je odziv stenske mase elastičan, vrednost faktora stabilnosti jeste $N_s \leq 1$. Za vrednosti faktora $N_s > 1,0$ odziv stenske mase je elastoplastičan, odnosno deo stenske mase u najbližoj okolini nepodgrađenog tunela jeste plastificiran. Na slici 9 šematski je prikazana promena veličine i oblika plastificirane zone stene oko tunela (Kavvadas, 2004), zavisno od veličine N_s .

U primeru podgrađenog tunela, jedan deo opterećenja preuzima podgrada tunela, tako da dubina plastifikacije stenske mase zavisi i od nosivosti podgrade. U određenom smislu, 3D efekat stabilnosti čela u funkciji je smanjivanja dubine plastifikacije, odnosno ona se ne može u potpunosti realizovati dok se iskop čela ne udalji dovoljno od posmatranog poprečnog preseka. Na taj način, stepen relaksacije, brzina napredovanja i deo opterećenja koji preuzima podgrada tunela direktno su povezani.

Uticao podgrade tunela možemo prikazati kao interni pritisak p koji djeluje u otvoru tunela, pri čemu je p_0 početni pritisak. Za $k_0 = \sigma_h / \sigma_v = 1$ stanje je stepen relaksacije:

In the case that the rock mass response is elastic, the value of N_s is less or equal one, $N_s \leq 1$. For the values of $N_s > 1,0$ the rock mass responses is elastic-plastic, which means that certain amount of rock mass around the unsupported tunnel remains in plastic state. The change of the shape and the size of the plastic region around the tunnel with the change of N_s (Kavvadas, 2004) are schematically shown in Figure 9.

In the case of supported tunnel, one part of the relaxation load is taken by the supporting elements, so that the depth of the plastic zone around the tunnel is related to the capacity of the lining and the installed rock bolts. In a certain sense, the 3D effect of the stable head of excavation is in the function of the decrease of the depth of plasticization, which means that it cannot be fully materialised until head of excavation is not advanced enough from the observed cross section. By these means, the development of relaxation, the timing of the advancement and the part of the loading taken by the tunnel supporting system are directly correlated.

The influence of the tunnel lining can be presented as internal pressure p , which acts within the interior of the tunnel, where p_0 represents the initial hydrostatic pressure. For the hydrostatic state $k_0 = \sigma_h / \sigma_v = 1$, the degree of relaxation is expressed as:

$$p = (1 - \lambda) p_0 \quad (6)$$

Može se pokazati da za linearno-elastične uslove iskopa kružnog tunela u homogenoj i izotropnoj stenskoj masi važi:

It can be shown that for linearly-elastic conditions the conditions of the tunnel excavation of the circular tunnel in homogeneous and isotropic rock mass is:

$$\sigma_\theta - \sigma_r = 2p_0 \lambda \quad (7)$$

$$\frac{r_p}{R} = \left[\left(\frac{2}{k+1} \right) \frac{N_s + \frac{2}{k-1}}{(1-\lambda)N_s + \frac{2}{k-1}} \right]^{\frac{1}{k-1}}$$

Slika 9. Veličina i oblik plastificirane zone oko nepodgrađenog tunela u zavisnosti od vrednosti N_s (Kavvadas, 2004), R - radijus tunela, r_p - radijus plastične zone oko tunela, k - koeficijent $k = \tan^2(45 + \varphi/2)$, φ - ugao smičuće otpornosti stenske mase.

Figure 9. The size and shape of plasticised zone around unsupported tunnel depending on value N_s (Kavvadas, 2004), R - tunnel radius, r_p - radius of plasticised zone around the tunnel, k - coefficient $k = \tan^2(45 + \varphi/2)$, φ - angle of shear resistance of the rock mass.

Pri čemu je σ_r radijalni i σ_θ tangencijalni napon. Ukoliko je njihova razlika (devijator napona) veća od jednoaksijalne čvrstoće σ_{cm} dolazi do plastifikacije stenske mase. Iz prethodne jednačine vidi se da je to direktno zavisno od faktora relaksacije λ . To znači, postoji granična vrednost koeficijenta relaksacije $\lambda = \lambda_{cr}$, (Kavvadas, 2004), pri kojoj se stenska masa plastificira:

where is σ_r radial and σ_θ tangential stress that acts in the rock mass around the cavity. The difference between σ_r radial and σ_θ tangential stress is the deviator stress. If deviator stress is greater than uniaxial rock mass strength σ_{cm} plasticization around the tunnel cavity occurs. It can be seen that this is directly related to the degree of relaxation λ . This means that there is a certain value of the degree of relaxation $\lambda = \lambda_{cr}$, (Kavvadas, 2004) which will lead to the plasticization of the rock mass. This λ_{cr} can be calculated as:

$$\lambda_{cr} = 1 - \left(\frac{2}{1+k} \right) \left(\frac{N_s - 1}{N_s} \right) \quad (8)$$

gde je $k = \tan^2(45 + \varphi/2)$, φ je ugao smičuće otpornosti stenske mase.

where $k = \tan^2(45 + \varphi/2)$ and φ is the angle of shear resistance of the rock mass.

Iz toga sledi da je stenska masa oko tunela u elastičnom stanju ako je:

This means that the rock mass around the tunnel cavity is inelastic state:

$$N_s \leq 1 \quad \text{or} \quad (N_s > 1 \text{ i } \lambda \leq \lambda_{cr}) \quad (9)$$

ili u plastičnom stanju ako je

or in the plastic state if:

$$N_s > 1 \text{ and } \lambda > \lambda_{cr} \quad (10)$$

Slika 10 Raspodela faktora N_s i λ_{cr} po sektorima duž ose tunela GAJ
Figure 10 The distributon of factors N_s and λ_{cr} along the sections of tunnel Gaj

Ukoliko želimo da pri $N_s > 1$ stanje u okolini tunela ostane elastično, moramo postići to da relaksacija napona usled iskopa tunela ne prelazi vrijednost λ_{cr} . To znači da moramo ugraditi oblogu dovoljne nosivosti na određenoj distanci od čela iskopa, koja obezbeđuje da 3D efekat još uvek zadržava potrebni nivo relaksacije napona.

Prilikom tumačenja vrednosti koeficijenta N_s , potrebno je imati u vidu to da je on izveden na osnovu teorije kontinuuma i da njegova vrednost od $N_s \leq 1$ ne garantuje stabilnost tunela u slučaju nepovoljno orijentisanih pukotina u stenskoj masi. U tom slučaju, potrebno je izvesti proračune stabilnosti protiv ispadanja blokova, koji mogu da prouzrokuju rušenja u tunelu.

Za vrednosti $N_s \geq 1,0$ glavni mehanizam odziva stenske mase zasnovan je na njenoj smičućoj otpornosti i elastoplastičnom ponašanju. U tim slučajevima, redovno se proverava stabilnost upotrebom numeričkih analiza i to najčešće metodom konačnih elementa. Za te analize potrebno je korektno proceniti raspodelu koeficijenta relaksacije λ_{cr} uzduž tunela. To se potom koristi da se opredeli skup numeričkih analiza sa upotrebom metode konačnih elemenata, da bi se izvršilo dimenzionisanje potpornih mera na različitim odsecima u tunelu.

Na slici 10 prikazana je raspodela faktora N_s i λ_{cr} po sektorima različitih kombinacija nadsloja i geoloških uslova duž ose tunela Gaj, koji je izveden na AC Sarajevo – Tarčin v BiH. Na osnovu slike možemo uočiti da su na određenim deonicama vrednosti N_s manje od 1 (to je bio deo tunela s niskim nadslojem u krečnjacima), dok na određenim deonicama prelaze vrednosti i od 6,0 (to je bio deo tunela u slaboj stenskoj masi s višim nadslojem). U praktičnom smislu, to je značilo da su za vrednosti od $N_s \geq 6,0$ numeričke analize ukazale na potrebu opsežnijih potpornih mera tj. jače primarne obloge i dužih stenskih sidara, kao i na manje predviđeno rastojanje od čela iskopa do zatvaranja

In the other words, if we want to achieve that for $N_s > 1$ state remains elastic around the tunnel, we must achieve that relaxation of stresses caused by excavation of the tunnel does not exceed values that correspond to λ_{cr} . This means that we have to install the supporting system (lining and the rock bolts) of given capacity at a certain distance from the head of excavation enabling conditions in which the 3D effect actually contains the amount of relaxation at the required level.

In the interpretation of the Factor of Tunnel Stability N_s it should be considered that this concept was derived on the theory of continuum mechanics and that the value $N_s \leq 1$ does not necessarily guaranty stable conditions for unfavourable layering of discontinuities in the rock mass. In this case a wedge analyses should be carried out to prove or mitigate possible instabilities in the tunnel.

For the values of $N_s \geq 1,0$ the main mechanism of the rock mass response is based on its shearing resistance and elastic plastic behaviour. In these cases the numerical analyses for the proof of tunnel stability are routinely carried out using Finite Element Method. The appropriate approach for these analyses is to first determine the distribution of the coefficient of relaxation λ_{cr} along the tunnel. This was then used to devise the set of Finite Element analyses, which will cover all significant variations in the N_s values and define the appropriate primary support systems for different tunnel sections.

The distribution of N_s and λ_{cr} along the tunnel Gaj, which was constructed on Motorway Sarajevo-Tarčin in Bosnia and Herzegovina, is shown in Figure 10. The distribution is shown for different combinations of the amount of overburden and appropriate geological conditions that varied along the tunnel. It can be seen in the figure that at the certain tunnel sections the values of N_s are less than 1 (e.g. shallow part of the tunnel in hard limestone), wherein in some sections the values of N_s are exceeding 6,0 (e.g. deeper part of the tunnel in poor

podnožnog svoda. Na slici su prikazani proračuni N_s i λ_{cr} za različite vrednosti koeficijenta K_0 i to za $K_0=1-\sin\varphi$ i $K_0=1,0$, što se smatralo logičnim graničnim vrednostima za koeficijent pritiska tla u stanju mirovanja.

ZAVRŠNE NAPOMENE I ZAKLJUČAK

U skladu s principima NATM, dozvoljen je - čak i poželjan - određeni nivo plastifikacije oko otvora tunela, ali on ne sme prekoračiti dužinu sidara. U idealnim okolnostima, plastifikacija ne prelazi dubinu od 3 m do maksimalno 6 m, što odgovara približnoj vrednosti $N_s=5-6$. U slučajevima kada N_s prekorači vrednost 7, možemo očekivati teškoće pri gradnji tunela, jer je to približno i granica upotrebljivosti koncepta NATM.

Ako to dozvoljava nosivost stenske mase, pravilo je da treba težiti da njen odziv ostane pretežno elastičan. To garantuje dugoročnu trajnost tunela, odnosno manju zavisnost njegove stabilnosti od materijala ugrađenih u njegovu podgradu. Ključno pitanje na koje projektant mora odgovoriti jeste na kojem rastojanju od čela iskopa se javlja vrednost λ_{cr} i kakvu oblogu treba ugraditi na tom mestu da bi odziv stenske mase bio pretežno elastičan odnosno plastičan do željenog nivoa.

U numeričkim analizama potrebno je težiti tome da se usvoji vrijednost λ koja je bliska vrednosti za λ_{cr} , jer to omogućava dimenzionisanje podgrade koja će zadovoljiti potrebu dominantnog elastičnog odziva stenske mase. Ukoliko je vrednost faktora stabilnosti tunela $N_s < 1,0$, to implicira višu vrednost za λ jer stenska masa ima zahtevanu nosivost, tako da samo mali deo opterećenja preuzima primarna podgrada. U skladu s tim, unutrašnje sile u oblozi biće relativno male i obratno, manji λ -faktor znači veće unutrašnje sile i manje deformacije stenske mase.

Vrednost λ -faktora - koja se koristi kao ulazni podatak za numeričke analize - utvrđuje se na bazi teorijskih pretpostavki i ocene nosivosti odnosno kvaliteta stenske mase. Svakako, najrealnija ocena vrednosti λ -faktora i interakcije potpornog sistema i stenske mase postiže se povratnim statičkim analizama. Za efikasnu primenu povratnih analiza potrebno je obezbediti podatke na osnovu tekućeg praćenja svih važnih pokazatelja interakcije stenske mase i potpornih elementa. U tu svrhu, u toku građenja tunela, potrebno je kontinuirano: a) vršiti geološko kartiranje čela iskopa i valorizaciju parametra čvrstoće stenske mase; b) dnevno meriti radijalne deformacije u tunelu; c) po potrebi, ugrađivati i interpretirati merne profile za određivanje pomeranja po dubini stenske mase i stepena mobilizacije stenskih sidara. Merni profili (tj. ekstenzometri i merna sidra) po pravilu se koriste u slučaju pojave slabijih geoloških uslova. Na osnovu dobijenih informacija pokazatelja stanja u tunelu i rezultata povratnih analiza, projektant dobija mogućnost da pravilno odluči o eventualnoj promeni podgradnog sklopa radi optimizacije gradnje tunela.

Prethodna objašnjenje osnovnih pojmova principa NATM jesu osnova za uspešno, sigurno i ekonomski opravdano građenje tunela. U praksi je to, nažalost,

rock). In the sections with the values of $N_s \geq 6,0$ the numerical analyses were used to dimension the elaborate primary support including stronger primary lining, longer rock bolts and the requirement for the short distance from the head of excavation to the closing of the tunnel invert. The calculations of N_s and λ_{cr} in the figure are given for the different values of the coefficient of earth pressure at rest K_0 for $K_0=1-\sin\varphi$ and $K_0=1,0$, which are considered to be appropriate boundary values.

THE FINAL REMARKS AND CONCLUSIONS

In accordance to the principles of NATM, certain level of plasticization around the cavity of the tunnel is desirable, but its size must not exceed the length of the rock bolts. In the ideal circumstances the depth of plasticization around the tunnel should be kept around 3m and should not exceed 6m, which corresponds to the approximate value of $N_s=5-6$. In cases when N_s exceeds the value of 7,0 one can expect difficulties during the tunnel construction, as this indicates practical limit for successful use of NATM.

As a rule (if the self-capacity of the rock mass allows for it) one should strive to keep the rock mass in predominantly elastic state as this complements to the durability of tunnel construction, that is, ensures less dependence on the materials installed in the tunnel (e.g. tunnel lining and rock bolts). The key question that designer should answer is at what distance from the head of excavation is the value of λ_{cr} and what type of primary support at that position should be installed so that the response of the rock mass remains elastic or becomes plastic up to the desired level.

If possible, during the execution of the finite element analyses, the value of λ should be chosen to be close to λ_{cr} , as this will result into the dimension of the primary lining within the required elastic response of the rock mass. If the value of Factor of Tunnel Stability is $N_s < 1,0$ this implies higher value for λ as the rock mass have the required capacity and only a small fraction of the relaxation is taken by the supporting system. The opposite is also true; the lower value of λ implies higher internal forces in the lining and less deformation of the rock mass.

The value of the coefficient λ , which is used as the input parameter for the numerical analyses, is determined from the described theoretical concepts and the prediction of the rock mass quality. However, the most realistic determination of the factor λ and interaction between the supporting system and the rock mass is evaluated using the back analyses. For the efficient use of back analyses it is necessary to obtain all the relevant information with regard to the interaction of the rock mass and the tunnel lining. For this purpose a permanent monitoring of the key indicators is carried out during the tunnel construction: a) geological mapping of the head of excavation and evaluation of the quality of the rock mass, b) daily assessment of the convergence movement in the tunnel, and c) if necessary the installation and interpretation of measuring profiles for determination of the movements of the rock mass and the degree of mobilisation of rock bolts. The measuring profiles (e.g. extensometers and measuring anchors) are typically installed within the poor geological conditions. On the basis of the interpretation of the indicators of the

veoma teško dostići, jer se kontinuirano praćenje gradnje tunela ne izvodi pravovremeno i kvalitetno. To posledično onemogućava pravovremeno donošenje pravilnih odluka u smislu prilagođavanja primarne podgrade u skladu s promenom uslova gradnje.

Sve pokazatelje uspešnosti gradnje u praksi kontinuirano prati projektant u okviru projektantskog nadzora. Projektantski nadzor obuhvata prikupljanje informacija o geološkim i hidrogeološkim uslovima gradnje, praćenje merenja radijalnih pomeranja na konturi iskopa, praćenje rezultata drugih mernih profila, izvršenje povratnih analiza stabilnosti iskopa tunela i pravovremeno predlaganje promene podgradnog sistema ako je to potrebno.

Za uspešno delovanje projektantskog nadzora moraju biti jasno određene obaveze i dužnosti svih aktera gradnje: izvođača, inženjera, investitora i projektanta. Osnovni odnosi moraju biti definisani fleksibilnim tipom ugovora koji omogućava optimizaciju gradnje i sprečava nepotrebno trošenje finansijskih sredstava, kao i materijalnih i ljudskih resursa. Iskustvo autora govori u prilog tome da se samo u tom slučaju dobijaju zadovoljavajući rezultati i iskorišćavaju potencijali koje u sebi nosi pravilna upotreba metode NATM. U suprotnom, gradnja tunela može biti nepotrebno skupa i neefikasna, s pogubnim finansijskim posledicama za investitora i izvođača. Postizanje fleksibilne ugovorne strukture koja je bazirana na crvenoj knjizi FIDIC-a (naplaćivanje prema izvedenim količinama) po iskustvu projektanta jeste najprimerenije rešenje za aplikaciju principa NATM u tunelogradnji.

state in the tunnel and the results of the back analyses, the designer becomes able to achieve the correct decisions with regard to eventual change of the supporting system and thus optimisation of tunnel construction.

Understanding of the basic NATM principles presented in the paper is the basis for successful, safe and economically viable tunnel construction. Unfortunately, in practise, this is usually difficult to achieve as the permanent monitoring is not carried out on time and with required quality. In consequence, the well-timed and correct decisions are not taken and there is no appropriate change of primary support to conform the change in boundary conditions.

The permanent evaluation of indicators of tunnel construction should be carried out by the designer within the process of designer supervision. The designer supervision comprises the acquisition of geological and hydrogeological data, monitoring of the convergence movements on the excavation contour, monitoring and interpretation of the results of the measuring profiles, execution of the back analyses for the evaluation of tunnel stability and propositions for the change of supporting system when appropriate.

For a successful execution of the designer supervision it is necessary that all the actors in construction: designer, contractor, engineer, and client have clearly defined roles. The relationships should be defined within the flexible contractual framework that enables the optimisation of construction and prevent unnecessary wasteful use of resources: human, material and financial. The experience of the authors is such that only under those circumstances there are satisfactory results and proper utilisation of the potential of NATM. Otherwise, the tunnel construction can be unnecessary expensive bringing up deadly financial consequences to the client and/or to the contractor. The establishment of flexible contractual framework within the red book of the FIDIC type of contract (admeasurement) is the most appropriate solution for application of NATM in practice by the experience of the authors.

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REZIME

KORISNI KONCEPTI U PRIMENI NOVE AUSTRIJSKE METODE ZA GRADNJU TUNELA

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U članku su predstavljeni koncepti razvijeni u dugogodišnjoj praksi pri projektovanju i gradnji tunela primenom Nove austrijske metode (New Austrian Tunnelling Method - NATM). Cilj članka jeste da predoči osnovne teorijske koncepte ove metode i da iznese iskustva kako pri projektovanju tunela, tako i pri aplikaciji tih principa na konkretnim primerima. Prikazana su uputstva za primenu korisnih koncepta prilikom dimenzionisanja primarne podgrade, kao i metodologija ocene stepena upotrebljivosti NATM metode za date granične uslove. U članku su takođe obrađeni postupci primene NATM metode u uslovima izvođenja radova, pri čemu je kao osnova za uspešan rad ispostavljena aktivna uloga projektanta i fleksibilnost ugovornih odnosa između investitora i izvođača.

Ključne reči: projektovanje, tuneli, NATM, SCL, stenska masa, nosivost

SUMMARY

USEFUL CONCEPTS FOR APPLICATION OF NEW AUSTRIAN TUNNELLING METHOD IN TUNNEL CONSTRUCTION

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The concepts that were developed in the long-standing practice in designing and constructing tunnels using New Austrian Tunnelling Method (NATM) are presented in the paper. The aim of the paper is to highlight the basic theoretical concepts and to expose the experience in the design and the application of NATM principles following the concrete examples. The procedures used in the application of these principles are also addressed in the paper, in which the active role of the designer and the contractual flexibility between the contractor and the client are seen as the key conditions for a successful work.

Key words: design, tunnels, NATM, SCL, rock mass, capacity