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Prorov za vozila u četvrti Tuen Mun u Hong Kongu

Mr. sc. Davorin Kolić,
Neuron Consult ZT, Pasching, Austrija
Mr. sc. Huaren Dou, Mr. sc. Leslie Swann,
Jacobs Babtie Ltd, Hong Kong, NR Kina

Sažetak

Ovaj članak ponajprije prikazuje geotehničke vidove prorova za vozila izgrađena ispod jedne od najprometnijih autocesta u Hong Kongu. Prorov presjeka što se sastoji od gornjega svoda i obrnute ploče približno je visok 6,2 m u kruni, a širok je 15 m u dnu, dok je dug 40 m. Izričito se zahtijevalo da se građenje odvija tunelograđevnim postupcima kako bi se gornji promet održao neprekinutim.
U članku se opisuje projektiranje privremenoga podgrađivanja za tunelske iskope, što je bilo izvedeno pomoću svoda od vodoravnih cijevi ugrađenih postupkom zabijanja cijevi. Svod od cijevi bio je zatim poduprt čeličnim rebrima temeljenima na stijeni ili na masivnom betonu. Dan je osvrt i na rješenje trajne obloge. Predočit će se i rezultati numeričkog modeliranja međudjelovanja tlo-građevina uz primjenu 2-D računalnoga programa FLAC. Ti su rezultati uspoređeni s onima što su dobiveni motrenjem za vrijeme građenja, a pokazalo se je da se oni dobro slažu s mjerenjima u svjetlu pomakā tla. U članku će biti govora i o izazovima pri građenju.

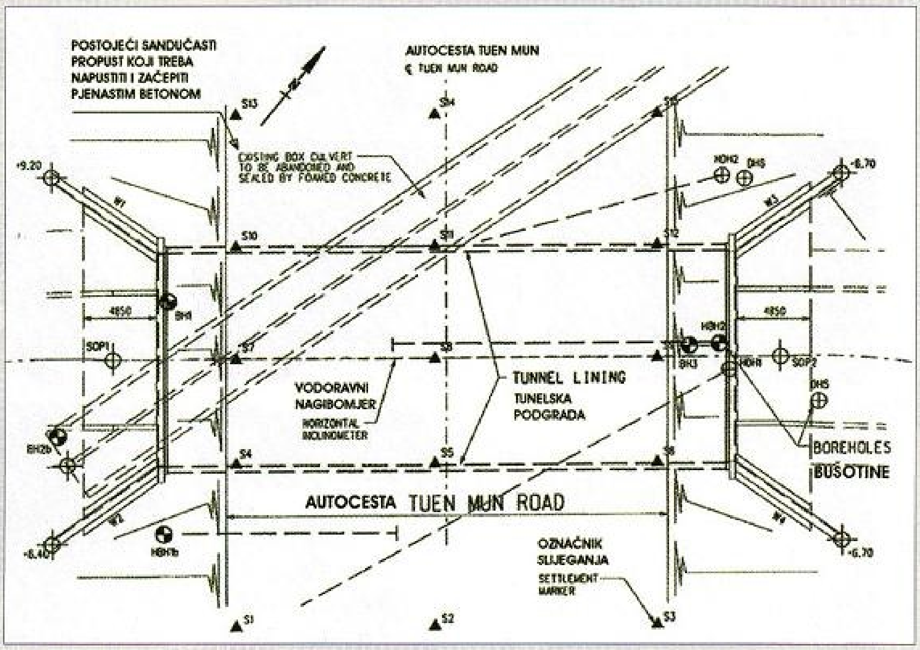
1 Uvod

Građenje prorova u sklopu je proširenja ceste So Kwun Wat u 56. okrugu četvrti Tuen Mun u Hong Kongu. Dvotračna cesta križa se s postojećom autocestom Tuen Mun, jednom od najprometnijih, prolazeći kroz dvotračni prorov, što ga treba proširiti na četverotračni.

Proširenje iziskuje da se novi dvotračni tunel izgradi samo 1,5 m ispod prometne autoceste Tuen Mun. Predviđeni prorov za vozila što prolazi ispod postojeće autoceste Tuen Mun nalazi se približno 25 m zapadno od postojećega prorova na ulici So Kwun Wat. Novi prorov (slika 1.) s ukupnim unutarnjim izmjerama 40 m (duljina), 15 m (širina) i 6,2 m (visina) treba imati dvotračni kolnik kako bi udovoljio zahtjevima proširenja. Gornja autocesta Tuen Mun, s dvama trotračnim kolnicima, širine oko 30 m, na ovom je potezu na razini između 12,6 i 13,5 m iznad razine mora u Hong Kongu.

Promet je na autocesti Tuen Mun oko 95.500 vozila dnevno, a po ugovoru promet na autocesti mora ostati neprekinut za sve vrijeme građenja prorova. Građenje mora biti takvo da najveći pomak tla na autocesti mora biti manji od 20 mm uz najveći nagib ograničen na 1 posto. Izričito se zahtijevalo da se građenje odvija tunelograđevnim postupcima.

Privremeno podgrađivanje iskopa tunela izvedeno je svodom sastav-



Slika 1. Položajni nacrt

Vehicular Underpass in the Tuen Mun Area 56, Hong Kong

Davorin Kolić, M.Sc.
CE, PE , Neuron Consult ZT, Pasching, Austria
Huaren Dou, M.Sc., Leslie Swann, M.Sc.
CE, PE, Jacobs Babtie Ltd, Hong Kong, PR China

Abstract

This paper mainly presents geotechnical aspects of a vehicular underpass that was built across under one of the busiest highway in Hong Kong. The underpass, in a section comprising an upper arch and invert slab, is approximately 6.2 m high at crown and 15 m wide at the base level with 40 m in length. The construction inevitably requires the underpass to be built using tunnelling techniques to keep the overhead road traffic uninterrupted.

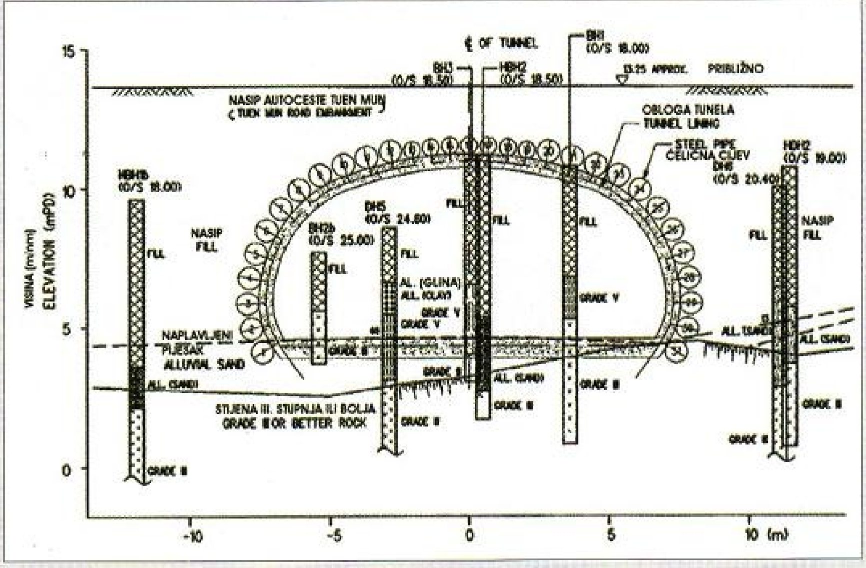
The paper will describe the design of temporary support for the tunnel excavation that was provided by an arch formed with horizontal pipes, which were installed using the pipe ramming method. The pipes arch was then supported by steel ribs founded on rock or mass concrete. Permanent lining design consideration was also discussed. Results of numerical modelling of the soil-structure interaction using the 2-D computer code FLAC will be presented.

The predictions have been compared to the monitored results during construction, which indicates that the prediction is in reasonable agreement with the measurement in the light of ground movement. The construction challenges will also be discussed in the paper.

1 Introduction

The construction of the underpass is a part of the road widening project for So Kwun Wat Road in Area 56 of Tuen Mun District in Hong Kong. The two-lane road passes an existing, one of the busiest highway – Tuen Mun Road via a present two-lane underpass, which is to be upgraded to four lanes. The upgrading requires a new two-lane tunnel to be constructed only 1.5 m below the live Tuen Mun Road. The proposed vehicular underpass crossing underneath the existing Tuen Mun Road is located approximately 25 m to the west of the existing underpass at So Kwun Wat Road. The new underpass (Figure 1), with an overall inner dimension being 40 m long, 15 m wide and 6.2 m high, is to provide a two-lane carriageway to meet the road widening requirements. The overhead Tuen Mun Road in this area, with a dual three-lane carriageway of approximately 30 m width, varies between 12.6 to 13.5 m level relative to Hong Kong Principal Datum.

The traffic on Tuen Mun Road is about 95,500 vehicles per day and the Contract stipulates that traffic on the Highway must be maintained uninterrupted at all times during the construction of the new underpass. The construction also requires that a maximum ground movement on the Highway must be less than 20 mm with a maximum gradient limited to 1%. The construction inevitably requires the underpass to be built using tunnelling techniques.



Slika 2. Tipičan geološki poprečni presjek

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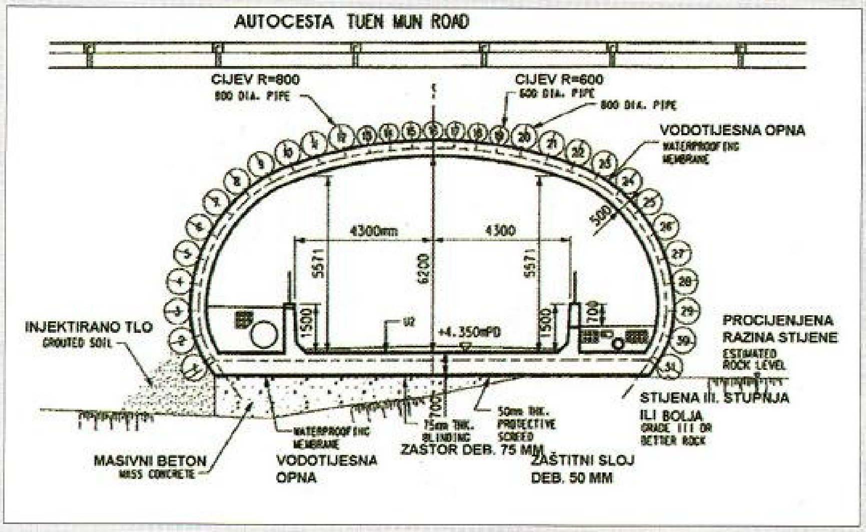


Slika 3. Razmještaj trajne obloge

ljenim od vodoravnih cijevi. Cijevi su ugrađene postupkom zabijanja cijevi. Glavni je izazov za izvoditelja i projektanta držanje pomaka tla pod nadzorom za vrijeme građenja unutar propisane granice od 20 mm. Ograničeni pokrov od 1,5 m ispod površine kolnika autoceste, iznad tunela, predstavlja praktičnu poteškoću u ispunjenju gornjih zahtjeva.

Može se pokazati potrebnom i obradba tla primjenom tlačnog injektiranja, kako bi se poboljšale vlastitosti tla izvan dosega tunela prije iskapanja tunela, ponajprije radi smanjenja slijeganja tla. Projektant mora biti sposoban izraditi projekt što će jamčiti izvedbu prorova kako je zahtijevano.

2 Uvjeti tla

Istražni radovi i laboratorijska ispitivanja tla obuhvatili su okomite i kose bušotine kako bi se utvrdili geološki uvjeti na mjestu. Ispod površine nasipa nalazi se tanak sloj aluvijalnog nanosa na trošnoj i zdravoj stijeni (granit). Vjerojatni geološki presjek duž obloge prorova prikazan je na slici 2. Tlo od kojeg je izgrađen nasip autoceste Tuen Mun sastoji se od finog do grubog a srednje gustog pijeska s muljem, šljunkom i oblutcima, pri čemu mu debljina seže od 7 do 9 m, na potezima ispod ceste. Aluvij, debljine 1,0 do 1,8 m, sastoji se od srednje gustog pijeska sa šljunkom i muljem.

Potpuno do jako rastrošeni granit (svrstan u stijenski razred V./IV.) opkoljen je glinovitim do muljevitim grubim pijeskom debljine do 2 m. Umjereno do slabo rastrošeni granit (nazvan stijenom III. razreda ili boljom stijenom) nađen je na razinama oko 2,7 do 3,0 m/nm. On je općenito srednje čvrst do čvrst (tlačna čvrstoća 90 do 190 N/mm²) s gustim do srednje gustim raspuklinama.(slika 2.)

Podzemna je voda nađena na razini oko 2,5 do 4,0 m/nm u pjezometrima ugrađenima na gradilištu.

3 Projektiranje

3.1 Trajna obloga

Po izvornom (sukladnom) projektu prorov je bila pravokutna sandučasta građevina. Nakon potanjega proučavanja mjesnih uvjeta i ograničenja, za prorov je predložena trajna armiranobetonska svođena građevina (slika 3.), jer bi ona bila sklopovno »kruća« od pravokutnog oblika uz pretpostavku jednake debljine. Ova svođena obloga ima po projektu debljinu 550 mm, na osnovi rezultata proračuna međudjelovanja tlo-građevina. Svod je omeđen približno na razini ceste, oko 4,2 m/nm, gdje je sklopovno povezan s temeljnom (cestovnom) pločom debljine 700 mm, što počiva na stijenskoj podlozi ili na masivnom betonu preko stijenske podloge. Obloga je tanja za 600 mm od krovne ploče izvornoga sklopa. (slika 3.)

3.2 Slijed građenja

Bila je predložena privremena podgrada kao svod od čeličnih cijevi za iskop tunela. Svod se sastoji od pripijenih cijevi ugrađenih vodoravno oko tunela postupkom zabijanja cijevi. Cijevi su bile projektirane tako da budu međusobno spojene sa susjednima. Ovo bi spriječilo prodor vode i gubitak gradiva između cijevi. Budući da je oblikovan svod od cijevi, mogli su se izvoditi radovi na zahtijevanoj obradbi tla kako bi se stabiliziralo čelno tlo unutar tunela uz portal i ugradila prva rebrasta podupora uz portal.

Uslijedio je iskop za tunel (slika 4.). Iskop se odvijao postupkom odozgo prema dolje. Glavna privremena čelična rebra trebalo je podupirati i okomitim potpornjem pri kruni svoda kako bi se smanjili pomaci tla i vodoravnim potpornjem pri petama čeličnih lučnih rebara, temeljenih na stijeni ili masivnom betonu, za vrijeme iskapanja ili betoniranja (slika 5.).

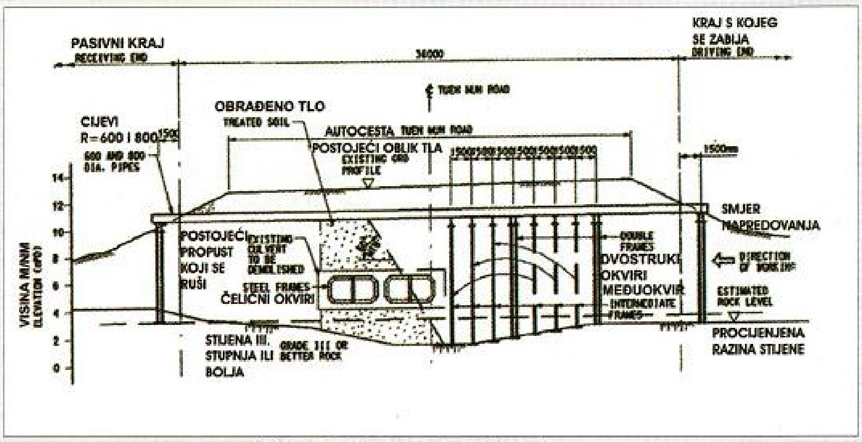
3.3 Projektni parametri

Geotehnički parametri rabljeni u proračunu temelje se na podatcima istraživanja na mjestu i rezultatima dobivenima iz odgovarajućih ispitivanja na mjestu i u laboratoriju. Projektni parametri što se tiču međudjelovanja tlo-građevina predočeni su u tablici 1.

Youngov modul (modul deformacije) tla, Es, izveden je iz empirijskog suodnosa vrijednosti N (broj udaraca/300 mm), izmjerenih pri standardnim penetracijskim pokusima na mjestu. Za gradiva tla, Es = c×n (MN/m²), gdje je c koeficijent suodnosa. Predočen je niz vrijed-

Tablica 1: Geotehnički parametri rabljeni u proračunu po FLAC-u

Material Gradivo	Unit weight Jedinična težina γ_t [kN/m³]	Young's modulus Youngov modul Es [MPa]	Cohesion Kohezija c' [kPa]	Angle of internal friction Kut unutarnjega trenja Φ' [°]
Embankment fill Nasip	20	35	0	35
Alluvial sand Naplavni pijesak	20	20	0	35
CDG	21	35	5	35
Bedrock Srasla stijena	25	5,000	100	45



Slika 4. Slijed radova i podupore

The temporary support for the tunnel excavation was provided by an arch formed with horizontal pipes. The pipes were installed using the pipe ramming method. The main challenge of the project to the Contractor and the Design Engineer is to control the ground movement within the specified limit of 20 mm during construction. The limited cover of the 1.5 m overburden to the Highway pavement surface, above the tunnel, presents practical difficulty in achieving the above requirements.

Ground treatment using pressure grouting may therefore be required to improve the soil properties outside the tunnel lining prior to the tunnel excavation, to mainly minimise ground settlement. The Design Engineer must be capable to produce a design that is able to demonstrate that the underpass can be constructed as required.

2 Ground Conditions

Ground investigation, along with laboratory tests, was carried out which consisted of vertical and inclined boreholes to define the geological conditions at the site. Subsurface conditions of embankment fill underlain by a thin layer of alluvial deposit overlying decomposed rock and bedrock (granite). As inferred geological section along the alignment of the underpass is shown in Figure 2.

The fill forming the Tuen Mun Road embankment comprises mainly fine to coarse grained medium dense sand with silts, gravel and cobbles, varying in thickness from 7 to 9 m in areas below the roadway. Alluvium, 1.0 to 1.8 m thick, consists of medium dense sand with gravel and silt. Completely to highly decomposed granite (classified as Grade V/IV rock) comprises clayey to silty coarse sand with a thickness up to 2 m. Moderately to slightly decomposed granite (termed Grade III or better rock) was encountered at levels ranging from about 2.7 to 3.0 mPD. It is generally moderately strong to strong (UCS=90 to 190 MPa) with closely to medium spaced joints.

Groundwater was measured approximately at a level of 2.5 to 4.0 mPD in the piezometers installed at the site.

3 Design Considerations

3.1 Permanent Lining

The original (conforming) design of the underpass was a rectangular box structure. After detailed study of the site conditions and constraints, a permanent reinforced concrete arch structure was proposed for the underpass (Figure 3), because this would be structurally more "rigid" than the original rectangular shape assuming the same thickness. This arch lining was designed to have a thickness of 500 mm, based on results of the soil-structural interaction analysis. The arch is approximately half way terminated at the road level of about 4.2 mPD which is structurally connected to the base (road) slab of 700 mm thickness that is supported an the bedrock or mass concrete over the bedrock. The lining is 600 mm thinner than the roof slab of the original box structure.

3.2 Construction Sequence

A temporary steel pipe arch support for the tunnel excavation was proposed. The arch is formed by contiguous steel pipes installed horizontally around of the tunnel using the pipe ramming method. The pipes were designed to have an interlocking with each adjacent pipe. This would prevent water ingress and loss of materials between the pipes. After the pipes arch is formed, the required treatment works to stabilize the face soil inside the tunnel at the portal can be performed and the first steel rib support installed at the portal.

The excavation for the tunnel is then followed (Figure 4). The excavation is carried out using a top-down method. The main temporary steel ribs also have to be supported by a vertical prop at the crown of the arch to reduce the ground movement and a horizontal prop against the toe of the steel arch ribs founded on rock or mass concrete during excavation and concreting (Figure 5).

3.3 Design Parameters

Geotechnical parameters used in the analysis are based on the site investigation data and results obtained from the corresponding in situ and laboratory tests. The design parameters related to the soil-structural interaction analysis are summarized in Table 1.

The Young's Modulus (deformation modulus) E_s of the soils is derived from the empirical correlation with the field measured penetration resistance N values (blows/300mm) from the Standard Penetration Test (SPT). For soil materials, $E_s = c\,N$ (MPa) where c is a correlation coefficient. A range of c values is reported. Plumbridge et al (2000) reports c = 3 to 6 based on back-analysis of full-scale pile load tests. Davies (1987) reports c = 1.4, which is used for completely decomposed rock in Hong Kong by some designers. Whiteside (1986) indicates c = 0.2 to 2 for granite saprolite. In general the value of the coefficient c ranges from 0.2 to 6.

The higher range corresponds to higher pre-loaded, compacted or dense materials. For the FLAC modelling presented herein, a c = 2.3, 2 and 1 is adopted for the embankment fill, alluvial sand and CDG material, respectively. The adopted design values of Young's moduli are within the typical range for soils in Hong Kong (GEO 1993).

A Young's modulus (E) of 200 GPa is used in the analysis for the steel or props. An elastic modulus of 26 GPa is utilised for the concrete used for the permanent lining in the modelling. For design pur-

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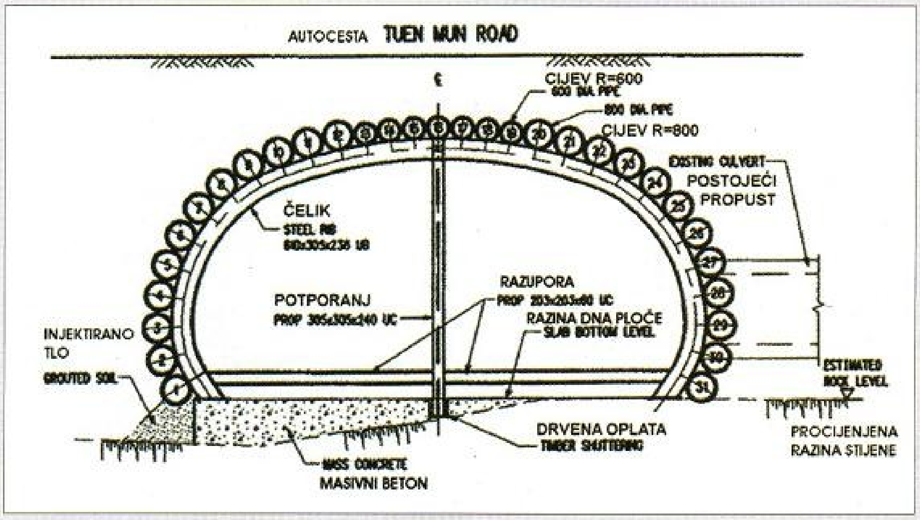


Slika 5. Privremena podgrada

nosti c. Plumbridge i dr. (2000.) navode da je c = 3 do 6 na osnovi naknadnoga proučavanja pokusnog opterećivanja pilota u naravnoj veličini. Po Daviesu (1987.) c = 1,4.

To neki inženjeri rabe za potpuno rastrošenu stijenu u Hong Kongu. Whiteside (1986.) uzima c = 0,2 do 2 za granitni saprolit. Općenito se vrijednost koeficijenta c kreće od 0,2 do 6. Više vrijednosti odgovaraju predopterećenim, zbijenim ili gustim gradivima. Za ovdje predočeno modeliranje FLAC, uzeto je, redom, c = 2,3; 2 i 1 za tlo u nasipu, aluvijalni pijesak i gradivo CDG. Odabrane projektne vrijednosti Youngovih modula u granicama su tipičnih vrijednosti za tla u Hong Kongu (GEO, 1993.).

Youngov modul (E) od 200 GN/m^2 rabljen je za proračun čelika ili potpornjeva. Modul elastičnosti od 26 GN/m^2 uziman je u modeliranju za beton rabljen za trajnu oblogu. Za proračunske svrhe uzeto je da je razina podzemne vode 8 m/nm, kako je zahtijevano u Posebnim uvjetima za projektiranje tunela u ugovoru.

3.4 Numerička predviđanja

Iskapanje i podgrađivanje za izvedbu predloženoga prorova modelirano je i proračunano uporabom 2-D računalnoga programa FLAC (Itasca Consulting Group, 1995.). Pripadni građevni radovi, uključujući podgrađivanje pri iskapanju i ugradbu trajne obloge, obuhvaćeni su proračunom.

3.5 Proračunski model

Kritični presjek blizu sredine trase tunela (slika 2.) odabran je za numerički proračun dvodimenzijskoga problema. Pri modeliranju je rabljen poopćeni geološki profil. Diskretizacija kontinuuma i rubni uvjeti pomaka modelirani su tako da obuhvate i geometriju tunela i slijed građenja. Mreža modela prikazana je na slici 6. Glavna privremena čelična rebra i potpornji, te trajna betonska obloga modelirani su kao niz elastičnih grednih elemenata.

U numeričkom su modelu zanemarivana privremena čelična međurebra, radi veće sigurnosti, jer su ona uklanjana nakon ugradbe glavnih rebara. Pojedinačna nosivost ili krutost vodoravnih cijevi nije razmatrana jer cijevi prenose opterećenje nadsloja na glavna dvostruka čelična rebra, a spojevi među cijevima predviđeni su kao dodatna zaštita, ponajprije radi sprječavanja gubitka tla i procurivanja vode.

Okomiti rubovi modela stavljeni su na dostatnoj udaljenosti od područja iskapanja (općenito za red veličine pet do šest puta dubina iskapanja) kako bi se učinkovito odstranili rubni utjecaji. Donji vodoravni rub postavljen je na – 8,5 m/nm, osjetno ispod površine čvrste stijene. Dva okomita i donji vodoravni rub pridržani su i u vodoravnom i u okomitom smjeru. U blizini područja iskapanja, gdje su očekivane promjene skraćenja i naprezanja znatnije i zanimljivije, odabirani su mali elementi.

Mohr-Coulombov konstitutivni model rabljen je u proračunu i za tlo i za stijenu. U modeliranju po FLAC-u nije razmatrano snižavanje vode izvan područja iskapanja jer su svod od pripijenih spojenih cijevi i trajna obloga s vodonepropusnom opnom ugrađeni duž tunela. Ovako se ostaje na strani sigurnosti jer omogućuje da puni tlak vode djeluje na građevinu.

3.6 Modeliranje slijeda građenja

Građenje tunela što se odvijalo postupkom odozgo prema dolje napredovalo je od sjevernoga kraja. Slijed građenja modeliran je što je bilo moguće vjernije u proračunima po FLAC-u za dvodimenzijske probleme. Pripadni građevni radovi svrstani su u stanja građenja, počev od utvrđivanja početnih uvjeta tla do konačne razine iskopa. Stanja građenja, nakon početnih uvjeta tla i ugradbe cijevnoga svoda, općenito su modelirana ovako:

1. stanje: Odvodnja i iskop kako bi se oblikovao projektirani tunelski otvor, te ugradba glavnih čeličnih rebara, kao i okomitih potpornjeva od krune svoda do stijenske podloge, te vodoravnih razupora što povezuju pete čeličnih lučnih rebara (stanje građenja).

Tablica 2: Najveće rezne sile i pomaci tla izračunani po FLAC-u

Model Stage Stanje	Description Opis	Structural force / Rezna sila (moment kNm/m; force/sila kN/m)			Accumulative surface settlement at crown Ukupno površinsko slijeganje pri kruni [mm]
		Bend. moment Moment savij.	Axial force Uzdužna sila	Shear Popr. s.	
1	Steel rib Čel. rebro	198	533	166	4.3
	Vertical prop Potporanj	N/A	404	N/A	
	Horiz. prop Razupora	N/A	33	N/A	
2	Perm. lining Trajna obloga	350	821	293	19.4
	Base road slab Donja koln. pl.	444	250	218	

poses, ground water level is taken at 8 mPD as required in the Particular Specifications for the Contract of the tunnel design.

3.4 Numerical predictions

The excavation and shoring support for the construction of the proposed underpass is modelled and analysed using the 2-D computer programme FLAC (Itasca Consulting Group, 1995). The appropriate construction activities, including excavation support, and installation of permanent lining, were considered in the analysis.

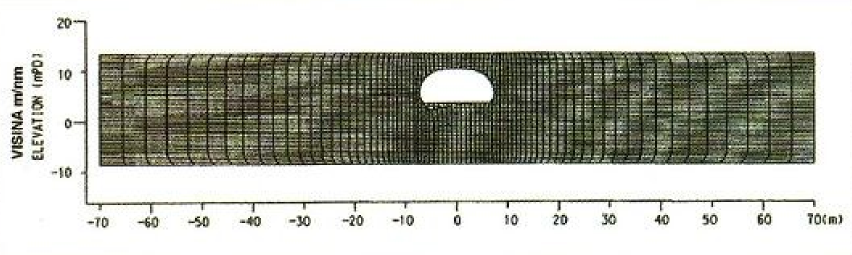
3.5 Analytical Model

A critical section near the middle of the tunnel alignment (Figure 2) was selected for the numerical analysis of a two-dimensional problem. A generalised geological profile was used for the modelling. The discretization of the continuum and the displacement boundary conditions were modelled to accommodate both the geometry of the tunnel and the construction sequence. The mesh of the model is shown in Figure 6.

The main temporary steel ribs and props, and the permanent concrete lining have been modelled as a series of elastic beam elements. In the numerical model, the temporary intermediate steel ribs is conservatively ignored since they will be removed after installation of the main ribs. The individual strength of stiffness of he horizontal pipes is not considered, since the pipes will transfer the overburden pressure to the main double steel ribs and the interlocking between pipes is provided as an extra protection mainly for the purpose of prevention of loss of ground and water leakage.

The vertical boundaries of the model were set a sufficient distance from the excavation region (generally on the order of 5 to 6 times of the excavation depth) to effectively eliminate boundary effects. The lower horizontal boundary was set at -8.5 mPD well below the bedrock surface. The two vertical boundaries and the lower horizontal boundary are restrained in both horizontal and vertical directions. Small model elements were used near the excavation region where change in strain and stress are expected to be more significant and of primary concern.

The Mohr-Coulomb constituent model was used for soil and rock in the analysis. In the FLAC modelling, no water drawdown was considered outside the excavation since a contiguous interlocked pipe arch and permanent lining with waterproof membrane were installed along the tunnel. This is conservative as it allows full water pressure acting on the structure.



Slika 6. Mreža modela FLAC s ugrađenim tunelom

3.6 Modelling of Construction Sequence

The construction of the tunnel carried out using a top-down construction technique advanced from the north driving end. The construction sequences were modelled as closely as possible in FLAC analyses for two dimensional problems. The appropriate construction activities were grouped into stages, starting from setting up the initial ground conditions to the final excavation level. The construction stages, after setting up initial ground conditions and installation of the pipe arch, are generally modelled as follows:

Stage 1: *Dewater and excavate to form the design tunnel void and install the main steel ribs, as well as vertical props from the crown of the arch to bedrock and horizontal prop connecting the toe of the steel arch ribs (construction stage).*

Stage 2: *Install permanent concrete lining including the tunnel base slab for the road, and remove all temporary steel ribs and prop supports (permanent conditions).*

3.7 Analytical Results

The predicted ground movements and structural forces are presented for both model stages. It is found that the most critical stage, in which maximum ground movement and structural forces were computed, occurs at completion of construction works (permanent conditions after installation of lining). A summary of the results computed from FLAC is given in Table 2 below. The table summarises the computed maximum structural forces and maximum accumulative ground movements at different stages. The structural forces computed from FLAC were used in the structural design of the tunnel.

It is seen (Figure 7) that during stage 1, the ground surface settlement at the crown location is predicted to be 4 mm, which essentially results from the deflection of the steel ribs. After removal of the temporary steel ribs, it is predicted that a further 15 mm ground settlement at the crown location will take place due to deflection of the permanent lining. The analysis shows that the ground settlement generally reduces with increasing distance from the crown of the arch (Figure 7 and 8).

Ground movements on both sides of the tunnel are approximately symmetrical around the centre line of the tunnel. The slight unsymmetry results from non-symmetrical geological profiles. Figure 8 shows the ground movement pattern after permanent lining is installed and temporary support removed. The plot indicates that the ground at the tunnel crown area would move downwards and the lower two-third portion of the tunnel on both sides displace outwards. The prediction suggests that the ground may be subject of 1 to 2 mm at about 10 to 15 mm distance from the centre line of the tunnel.

The tunnel will be essentially built within embankment fill material of Tuen Mun Road. Sensitivity studies with respect to the deformation modulus of the embankment fill have been conducted by reducing the modulus by 50 % (i.e. E_s = 18 MPa, c = 1.2) in the FLAC modelling. The results show that a reduction of the modulus by 50 % would lead to an increase in maximum ground settlement by 7 % at the end of the construction.

Analyses were also carried out to study the sensitivity to the structural stiffness and arrangement of temporary support in order to optimise the design. With consideration of the potential risk and practically,

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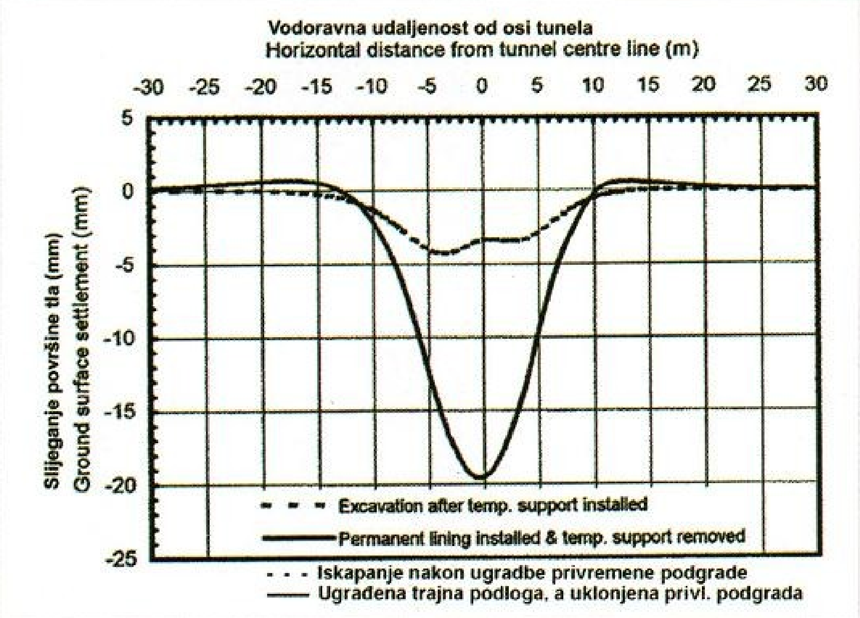


Slika 7.: Profil predviđenih slijeganja tla

2. stanje: Ugradba trajne betonske obloge, uključujući tunelsku temeljnu ploču za cestu, te uklanjanje svih privremenih čeličnih rebara i potpornjeva (trajno stanje).

3.7 Rezultati proračuna

Predviđeni pomaci tla i rezne sile predočeni su za oba modelska stanja. Nađeno je da je kritično stanje, u kojem su dobiveni najveći pomaci tla i rezne sile, ono kada se dovršavaju građevni radovi (trajno stanje nakon ugradbe obloge). Sažetak rezultata dobivenih po FLAC-u dan je u tablici 2. Tablica predočuje izračunane najveće rezne sile i najveće zbirne pomake tla u različitim stanjima. Rezne sile izračunane po FLAC-u rabljene su za dimenzioniranje tunela.

Vidi se (slika 7.) da je za vrijeme 1. stanja slijeganje površine tla pri kruni predviđeno da bude 4 mm, što zapravo potječe od progiba čeličnih rebara. Nakon uklanjanja privremenih čeličnih rebara predviđeno je da će se dogoditi daljih 15 mm slijeganja tla pri kruni zbog progibanja trajne obloge. Proračun pokazuje da se slijeganje tla općenito smanjuje s povećanjem udaljenosti od krune svoda (slike 7. i 8.). Pomaci tla na objema stranama tunela približno su simetrični s obzirom na središnju os tunela.

Blaga nesimetrija potječe od nesimetrije geoloških profila. Slika 8. pokazuje predložak pomakā tla nakon ugradbe trajne obloge. Crtež pokazuje da bi se tlo pri kruni tunela micalo prema dolje, a donje se dvije trećine tunela na objema stranama miču prema vani. Predviđanje daje naslutiti da bi tlo moglo biti niže za 1 do 2 mm na udaljenosti 10 do 15 mm od središnje osi tunela.

Tunel će se uglavnom graditi unutar gradiva nasipa autoceste Tuen Mun. Proučavanja osjetljivosti s obzirom na modul deformacije gradiva nasipa izvedena su uz smanjenje modula za 50% (tj. Es = 18 MN/m^2, c = 1,2) u modeliranju po FLAC-u. Rezultati pokazuju da smanjenje modula za 50 posto dovodi do povećanja najvećega slijeganja tla za 7 posto na kraju građenja. Proračuni su provedeni i radi proučavanja osjetljivosti sklopovne krutosti i razmještaja privremenih potpora kako bi se optimiziralo projektiranje. Uz razmatranje mogućeg rizika, a i iz praktičnih razloga, konačno je prihvaćeno rješenje po kojem je otpala potreba injektiranja izvan obloge zahvaljujući uvođenju krućih privremenih potpora.

Budući da uzdužne vodoravne cijevi nisu modelirane u 2-D proračunu po FLAC-u, proveden je poseban proračun po računalnomu programu SuperSTRESS (Integer 1995.) kako bi se procijenio progib cijevi pri kruni. Najveći progib vrha cijevi ispod kolnika poduprta glavnim čeličnim rebrima za vrijeme stanja građenja predviđen je da će biti 0,4 do 0,6 mm. Progib od 1,5 mm izračunan je za skrajnje poteze vršne cijevi pod pokosom nasipa, budući da ima veći raspon. Očekuje se da će utjecaj ovoga progiba cijevi na slijeganje površine tla biti zanemariv. Procjenjuje se da će ukupno zbirno slijeganje površine autoceste Tuen Mun biti reda veličine 19 mm, što je unutar projektnoga kriterija za najveći pomak od 20 mm.

3.8 Zabijanje cijevi i privremeno podgrađivanje

Temeljno privremeno podgrađivanje za iskapanje i građenje tunela sastoji se u primjeni i ugradbi vodoravnih cijevi što tvore privremeni potporni svod oko oboda tunela (slika 5.). Tehnika se očituje u primjeni stroja za zabijanje cijevi što može ugurati po pravcu cijev promjera 600 do 800 mm u tlo. Postrojenje za zabijanje prvenstveno uključuje opremu za zabijanje (zvanu GRANDORAM, proizvod njemačke tvrtke Grandoram Koloss), zračne kompresore i generatore.

Odabran je postupak ugradbe spojenih čeličnih cijevi oko oboda prorova, nezavisno od geometrije poprečnoga presjeka prorova. Utorasti uzdužni spojevi između susjednih cijevi djeluju kao vodilice za vrijeme ugradbe cijevi, a kasnije, za vrijeme građenja tunela, kao zaštita od prodora vode i tla. Utorasti spojevi nemaju sklopovne svrhe, iako se javljaju male poprečne sile u smjeru poprijeko na os cijevi.

Zabijanje cijevi općenito se izvodilo s jedne strane prorova (strana zabijanja), osim što su se, tamo gdje je bio postojeći propust, cijevi zabijale s obiju strana. Na strani zabijanja ugrađeno je čelično žmurje kako bi se učvrstilo lice nasipa gdje su čelične cijevi zabijane prema tijelu nasipa (slika 9.). Na suprotnoj strani pokos nasipa načelno nije zaštićivan dodatnim mjerama; umjesto toga pojačani su nadzor i opažanje kada bi cijev dosegnula izlaznu točku.

Budući da je poprečni presjek tunela iziskivao otvor visine do 7,5 m, izrađena je privremena podnica od čeličnih profila kako bi bilo dostupno mjesto gdje su zabijane gornje cijevi. Na podnici su pripravljene vodilice od tračnica za izvedbu zabijanja. Prvi odsječak čelične cijevi od 8 m imao je šiljak smješten na čelu čeličnog naglavka. Malj za zabijanje na stražnjem dijelu cijevi utjerivao je cijev kroz otvor u žmurju prema nasipu, a sljedeći odsječci cijevi od 8 m privarivani su na prethodne.

Vibriranje cijevi izvođeno je prvenstveno na stražnjoj strani čeličthe adopted final design is such that the grouting need outside the lining is eliminated with introduction of a stiffer temporary structural support.

Since the longitudinal horizontal pipes are not modelled in the 2-D FLAC analysis, a separate calculation using the computer programme SuoerSTRESS (Integer 1995) was performed to estimate the deflection of the pipe at the crown location. The maximum deflection of the top pipe under the carriageway supported by the main steel ribs during stage is predicted to be 0.4 to 0.6 mm.

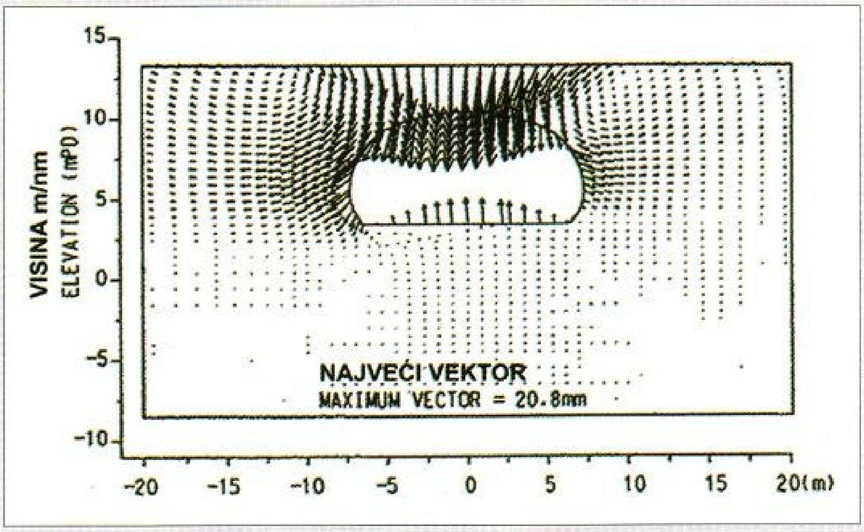
A 1.5 mm deflection is calculated for the end portion of the top pipe under the embankment slope as it has a longer span. The impact of the above pipe deflection on the ground surface settlement is expected to be negligible. The total accumulative maximum settlement on the Tuen Mun Road surface is estimated to be of the order of 19 mm, which is predicted to satisfy the design criterion of maximum 20 mm movement.

3.8 Pipe ramming and temporary support

The primary temporary support for the tunnel excavation and construction is the use of installation of the horizontal pipes to form a temporary arch support around the perimeter of the tunnel (Figure 5). The technique is to use a pipe ramming machine able to push 600 to 800 mm diameter steel pipes in a straight line into the ground. The ramming plant primarily includes the ramming equipment (called »GRANDORAM« made by Grundoram Koloss in Germany), air compressors and generators.

The method was adopted to install interlocked steel pipes around the perimeter of the underpass, independent of the geometry of the underpass cross section. Interlocked longitudinal joints between neighbouring pipes act as a guidance tool during pipe installation, and later on, during the tunnel construction, as a protection against water and soil inflows. The interlocked joints do not have a structural purpose, even though they have some shear in the pipe cross-section direction.

Pipe ramming was generally performed from one side (driving end) of he underpass with an exception that where the existing box culvert was present, the pipe ramming was carried out from both sides. At



Slika 8 Vektori pomakā tla

the driving end, a sheet pipe wall was installed to fix the face where steel pipes were rammed toward the body of the embankment (Figure 9). At the receiving end the embankment slope was basically not protected by additional measures, instead, a careful control and observation was enforced when the pipe reached the exit point.

As the tunnel cross section required an opening up to 7.5 m high, temporary platform made of steel members was formed to reach the location where upper steel pipes were to be rammed. On the platform, steel guide rails were prepared for the ramming operation. The first 8 m segment of the steel pipe had a "cutting nose" placed on the head of a steel sleeve. The ramming hammer on the rear part of the pipe drove the pipe through the hole cut in the sheet pile wall toward the embankment, and subsequent steel pipe segments in 8 m sections were welded on for the encountered and soil had to be removed.

Vibration of the steel pipe was primarily on the rear part of the steel pipe not on the front. The soil remained within the steel pipe installed, except that obstructions were removed. The method should not leave a gap between the exterior of the steel pipe and the surrounding soil; therefore, negligible settlement would be anticipated during ramming. Directional and positional control and guidance can be provided by a sensor-based system within the pipe. Correction is possible by moving the rear of the pipe. For actual installation of the pipes for this project, no sensor system was utilized.

Given the limited thickness of the overburden soil to the road pavement, seven 600 mm diameter pipes were used in the crown area of the tunnel. For the remaining portion, 800 mm diameter pipes were installed. After all pipes (total 31) around the design geometry of the tunnel were installed, the temporary support was essentially established. Excavation of the tunnel cross section on the face could then start after the steel ribs were installed at the portal first outside the sheet pile wall. Further excavation was carried out carefully in steps with installation of successive steel rib supports.

3.9 Face soil treatment and excavation

The soil inside the arch was treated prior to commencement of the excavation works for the tunnel to maintain the face stability. The treatment was required to ensure a steep temporary cut slope for the tunnel excavation advancing along the alignment. For a target factor of safety not less than 1.2 for the temporary cut slope of 60 degrees, the soil is required to have a strength of $c' = 17$ kPa and $\phi' = 35°$.

Cement grouting had been tried to stabilize the soil inside the tunnel. Trial excavation around the grouted location indicated that the influence zone of individual grouting was not satisfactory and the soil had not been properly cemented although a high grouting pressure was applied. Site inspection suggested that the soil was dense and

Slika 9. Zabijanje cijevi uz kraj za zabijanje

ne cijevi, a ne na prednjoj. Tlo je ostajalo unutar zabijene cijevi, osim kada bi se naišlo na kakvu zapreku, pa bi tlo trebalo odstraniti. Ovaj postupak ne bi smio ostavljati jaza između nutrine cijevi i okolnoga tla; zbog toga bi predvidivo slijeganje za vrijeme zabijanja bilo zanemarivo. Provjera smjera i položaja, te vodilica, izvođena je s pomoću sustava senzora unutar cijevi. Ispravak je moguć pomicanjem stražnjeg dijela cijevi. Pri ugradbi cijevi na ovoj građevini nije rabljen nikakav sustav senzora.

Zbog ograničene debljine nadsloja tla do kolnika ceste u području krune tunela uporabljeno je sedam cijevi promjera 600 mm. Za preostali dio ugrađene su cijevi promjera 800 mm. Pošto su ugrađene sve cijevi (ukupno 31) oko predviđenog obrisa tunela, privremeno podgrađivanje bilo je zapravo uspostavljeno. Iskapanje poprečnoga presjeka na licu tunela moglo je otpočeti nakon ugradbe čeličnih rebara uz portal prvo izvan žmurja. Dalje iskapanje izvođeno je oprezno, u koracima, uz ugradbu sljedećih podgrada od čeličnih rebara.

3.9 Obradba tla na licu nasipa i iskapanje

Tlo unutar svoda obrađivano je prije početka iskapanja za tunel kako bi se održala stabilnost lica nasipa. Obradba je bila nužna kako bi se osigurao strmi privremeni nagib pokosa usjeka za iskapanje tunela što je napredovalo duž obloge. Za ciljani faktor sigurnosti ne manji od 1,2 za privremeni nagib pokosa usjeka od 60^0, traži se da tlo ima koheziju $c' = 17$ kN/m^2 i $\phi' = 35^0$.

Iskušano je cementno injektiranje radi stabiliziranja tla unutar tunela. Pokusno iskapanje oko mjesta injektiranja pokazalo je da utjecajno područje pojedine injekcije nije bilo zadovoljavajuće i da tlo nije bilo valjano cementirano, iako je pri injektiranju bio primijenjen visok tlak. Pregled na mjestu pokazao je da je tlo bilo gusto, te da je sprječavalo širenje cementne injekcije. Kako bi se utvrdila čvrstoća tla, uzeti su uzorci iz predjela portala za laboratorijske troosne pokuse. Rezultati pokusa pokazali su da tlo bez dodatnog cementnog injektiranja ne bi dosegnulo dostatnu koheziju navedenu gore.

S obzirom na iskrsle poteškoće, predložen je drugačiji postupak obradbe, uporabom privremenih čavala u tlu, pri kojem bi čavli bili ugrađivani i injektirani s lica, prije svakoga kruga iskapanja. Oni bi morali biti najmanje dvostruke duljine jednoga kruga kako bi se zajamčio prijeklop. Oni bi se morali uklanjati pošto bi se lice postupno iskopalo. Ugrađeno je ukupno dvanaest čavala duljine po 12 m u lice tunela. Čavli imaju prijeklop od 3 m sa sljedećim ugrađenim čavlima.

Koraci iskapanja bili su ograničeni na 1,0 do 2,0 m duljine napredovanja (slika 4.). Nakon svakoga koraka iskapanja ugrađivana je privremena podgrada cijevnoga svoda u obliku čeličnih rebara sa spojnim limovima odmah uz cijevni svod. Mjere privremenog podgrađivanja omogućile su betoniranje konačne obloge uporabom oplate duljine oko 5,5 do 7,5 m.

Iskapanje je bilo projektirano tako da ima jedno lice napredovanja preko cijelog poprečnoga presjeka. Tipičan prizor postupka građenja prikazan je na slici 4. Nakon otvaranja poprečnoga presjeka kroz početno žmurje, iskapanje je počelo od vrha prema dnu ili do razine stijene ili do projektirane razine dna, kojegod bilo dublje. Između cijevi nanesen je sloj mlaznog betona kako bi se dobila »glatka« površina, a zatim je ugrađivano čelično rebro podgrade ili na stijensku podlogu ili na masivni beton preko nje.

Dvostruka rebra (okviri) ispod kolnika autoceste Tuen Mun bila su predviđena na razmacima od 6 m osim na krajevima uz portale gdje su ti razmaci bili 7,5 m i bila su poduprta okomitim potpornjem pri kruni, što je sezao do podložne stijene, kao i vodoravnim razuporama pri dnu čeličnih rebara. Slika 10. prikazuje dovršen privremeni cijevni svod pošto je tunel iskopan, čime je osigurana nužna podgrada za radove na trajnoj oblozi.

Razina podzemne vode unutar područja iskapanja bila je oko 2,5 do 3 m/nm, dobivena pjezometarskim mjerenjima na mjestu. Tako se iskapanje zapravo odvijalo u suhim uvjetima, jer je razina dna iskopa bila približno 3,5 m/nm.

Trajna obloga i ploča bile su betonirane u odsječcima pošto je iskapanje za tunel bilo potpuno dovršeno. Dovršena trajna tunelska građevina predočena je na slici 11.

3.10 Izmjereni pomaci tla i usporedba s predviđanjem

Kako bismo se uvjerili da su projektni kriteriji zadovoljeni, u tlo su ugrađeni instrumenti za motrenje građenja (slika 1.). To je obuhvaćalo označnike (»markere«) za površinske pomake na kolniku autoceste Tuen Mun, vodoravne nagibomjere (»klinometre«) ugrađene unutar vodoravne cijevi pri kruni, približno 1,8 m ispod površine tla, radi motrenja uzdužnog obrisa izobličenja duž obloge tunela, kao i stojeće cijevi za motrenje položaja podzemne vode. Instrumenti su praćeni redovito za sve vrijeme građenja.

Za vrijeme izvedbe zabijanja, a prije iskapanja tunela, pomaci su tla bili zapravo zanemarivi, ali je mjestimice uočavano uzdizanje do prohibited the dispersion of cement grout. To confirm the soil strength, samples were collected at the portal areas for laboratory triaxial tests. Test results indicated that the soil without the supplementary cement grouting would not achieve sufficient strength stated above.

Considering the difficulties encountered, an alternative treatment method of using temporary soil nails was proposed, in which the nails were installed and grouted into the face, prior to each round of excavation. These also had to be at least double the length of one round to ensure an overlap. They had to be removed as the face was progressively excavated. A total of 12 nails in 12 m length each was installed in the tunnel face. The nails have an overlap of 3 m with the following successive nails installed.

Excavation steps were limited to 1.0 to 2.0 m advance length (Figure 4). After each excavation step, the temporary support for the pipes arch was applied in the form of steel ribs with shims immediately against the pipes arch. Temporary support measures enabled concreting of the final lining using formwork in a length of about 5.5 to 7.5 m.

The excavation was designed to have one advancement face over the entire cross section. A typical detail of the construction procedures is shown on Figure 4. After opening the cross section through the starting sheet-pile wall, the excavation started form the top to the bottom down to either the rock level or the design formation level, whichever was deeper.

A shot- crete layer was placed between the pipes to form a "smooth" surface, and the supporting steel rib was then placed on either bedrock or mass concrete over bedrock. The double ribs (frames) below the Tuen Mun Road carriageway were designed to be placed at 6 m intervals except at the portal ends which were at 7.5 m, and be supported by a vertical prop at the crown extending to the bedrock, as well as a horizontal prop at the base of the steel ribs. Figure 10 shows the completed temporary pipes arch after the tunnel was excavated, which provided the required support for the permanent works.

The actual groundwater within the excavation area was around 2.5 to 3 mPD levels based on the piezometer measurement at the site. The excavation was thus performed essentially in dry conditions since the excavation level was approximately at 3.5 mPD.

The permanent lining and slab was cast in successive sections after the excavation for the tunnel was entirely completed. Completed permanent tunnel structure is shown in Figure 11.

3.10 Measured ground movement and comparison to predication

In order to ensure that the construction criteria are met, ground instrumentation (Figure 1) has been installed to monitor the construction. This included surface movement markers on the Tuen Mun Road pavement, a horizontal inclinometer installed inside the horizontal pipe at the crown location at about 1.8 m below the ground surface to monitor the longitudinal deformation profile along the tunnel alignment, as well as standpipes to monitoring groundwater conditions. The instrumentation has been monitored over the entire construction period on a regular basis.

During the pipe ramming process and prior to the tunnel excavation, ground movement was essentially negligible but heave at local area was observed up to 2 mm at the settlement markers directly above the crown area. This was caused primarily from the »push-up« of the observation encountered during the ramming due to the thin overburden cover to the ground surface.

A measured ground settlement profile along the centre lint of Tuen Mun Road over the tunnel, corresponding to the end of tunnel excavation (stage 1), is shown in Figure 12. It is seen that settlement up to 4 mm took place at locations to the east of the crown. This compares very well with the predicted value of 4.3 mm computed from FLAC (Figure 12), neglecting the effect of the horizontal pipe deflection of the Tuen Mun Road surface.

The deflection at the centre line location of the Tuen Mun Road is negligible because a main frame support for the pipes arch was provided at this location. Settlement markers installed at the edges of the Tuen Mun Road, particularly near the north portal of the tunnel, recorded a little larger settlement that was up to 6 mm. The additional amount of settlement is believed to have resulted from the horizontal stress relief due to vertical excavation at the portal slope of the embankment.

Ground settlement further occurred during the process of removing steel frames after casting the permanent lining with the installation of temporary central back-props against the crown. The settlement accumulated up to 8 mm over the tunnel crown area by the end of December 2002 after all temporary steel frames were removed. It is expected that at the end of the tunnel construction works, after removal of all the temporary back-props in February 2003, further ground settlement would occur but is likely to be less than the predicted magnitude of 19 mm at the crown location of the tunnel.

The measurement in the horizontal inclinometer installed along the alignment of the tunnel at the crown shows that the pipe (at the crown) downward deflection is up to a maximum 2 mm under the embankment slope at the receiving end. This measured value is approximately 30 % higher than the

Slika 10. Privremeni cijevni svod pri završetku iskapanja

 CIM god. 52. br. 1-6 (str. 88-101), Zagreb, siječanj/lipanj 2006.

Slika 11. Dovršena trajna građevina tunela uz portal

2 mm pri označnicima slijeganja izravno nad predjelom krune. Ovo je bilo uzrokovano prvenstveno »bujanjem tla« na što se nailazilo pri zabijanju, zbog tanka nadsloja do površine tla. Izmjereni obris slijeganja tla duž osi autoceste Tuen Mun iznad tunela što je odgovarao završetku iskapanja tunela (1. stanje) predočen je na slici 12. Vidi se da je dolazilo do slijeganja do 4 mm na mjestima istočno od krune.

To se vrlo dobro slaže s predviđenom vrijednošću od 4,3 mm izračunanom po FLAC-u (slika 12.), ako se zanemari učinak vodoravnoga progiba cijevi pri površini autoceste Tuen Mun. Progib pri osi autoceste Tuen Mun zanemariv je jer je glavna okvirna podgrada za cijevi upravo na tome mjestu. Označnici slijeganja umetnuti na rubovima autoceste Tuen Mun, osobito blizu sjevernoga portala tunela, zabilježili su nešto veće slijeganje, do 6 mm. Vjeruje se da dodatni iznos slijeganja potječe od opuštanja vodoravnog naprezanja zbog okomitog iskapanja pri portalnom pokosu nasipa.

Slijeganje tla događalo se nadalje za vrijeme uklanjanja čeličnih okvira nakon betoniranja trajne obloge s ugradbom privremenih središnjih potpornjeva što se oslanjaju o krunu. Slijeganje je naraslo na 8 mm iznad krune tunela do kraja prosinca 2002. pošto su uklonjeni svi privremeni čelični okviri. Očekivalo se je da će se pri kraju radova na građenju tunela, pošto se uklone privremeni potpornji u veljači 2003. događati i dalja slijeganja tla, ali je vjerojatno da će ona biti manja od predviđenog iznosa od 19 mm pri kruni tunela.

Mjerenje vodoravnim nagibomjerom ugrađenim duž obloge tunela pri kruni pokazalo je da progib cijevi prema dolje (pri kruni) doseže najviše 2 mm pod pokosom nasipa pri pasivnom kraju. Ova izmjerena vrijednost približno je veća za 30 posto od predviđenog 1,5 mm. Pod kolnikom autoceste Tuen Mun nagibomjer je zabilježio zanemarivo izdizanje. Ovaj se trend u biti nije promijenio do kraja prosinca 2002. nakon uklanjanja privremenih čeličnih rebara što podupiru cijevni svod, što je uslijedilo po betoniranju trajne obloge te uklanjanja središnjih potpornjeva.

3.11 Građenje i njegovi izazovi

Iako su provedena istraživanja tla, i iako su projekt prorova i postupak građenja bili dobro proučeni, bilo je nekoliko nepredvidivih poteškoća na koje se naišlo u tijeku građenja.

3.11.1 Zapreke pri zabijanju cijevi i poduzete mjere

Trideset i jedna čelična cijev ugrađena je po obodu tunela, od kojih je dvadeset i sedam bilo zabijeno s kraja za zabijanje kroz nasip autoceste do pasivnoga kraja, a preostale četiri cijevi uz postojeći okvirni propust ugrađene su s obaju krajeva i to do ruba postojećega propusta. Napredovanje nekolikih cijevi prestalo je na pola puta zbog zapreka, što je uglavnom bilo uzrokovano nazočnošću tvrdih, krupnih gradiva u nasipu.

Zapreke uz prednji kraj cijevi normalno su razbijane i uklanjane iz nutrine čelične cijevi s pomoću bušotina duž oboda tijela cijevi. Bušenje nije uvijek bilo učinkovito u uklanjanju zapreka. U nekim slučajevima moralo se posezati za ručnim iskopom, a to se radilo tako da su se radnici, opremljeni aparatima za disanje, uvlačili u cijevi (promjera 800 mm) i tamo kopali pneumatskim alatom.

Na osobito teške zapreke naišlo se u cijevima br. 3 i 6. Iako je mehaničko i ručno razbijanje i kopanje bilo dovršeno, nađeno je da se prvih nekoliko metara kružne cijevi bilo izobličilo, a kraj cijevi s kljunom za rezanje bio je izobličen i savijen u polukružan oblik.

Zabijanje cijevi br. 3 i 6 moralo se obustaviti na pola puta, a nadomjesne su cijevi zabijene sa suprotnoga kraja. Dodatni radovi na pojačavanju, kako bi se povezala dva komada cijevi, izvedeni su za vrijeme iskapanja tunela. Tvrdi materijal izvučen iz ovih cijevi bili su oblutci izvorne veličine veće od promjera cijevi. Šuplje su čelične cijevi tada zapunjene cementnom kašom po dovršetku zabijanja cijevi.

Za vrijeme zabijanja cijevi br. 21 i 22 uočeno je da se površina ceste blago izdiže u blizini krajeva cijevi kada su cijevi nailazile na zapreke. Nakon uklanjanja tla iz nutrine cijevi, nađeno je da su se cijevi nagnule prema gore za 200 mm u odnosu na projektiranu razinu. Da se zabijanje cijevi nastavilo nakon uklanjanja zapreka ispred cijevi naprijed navedenim mehaničkim postupkom, to bi vjerojatno ozbiljno pogodilo površinu ceste.

Osim toga, ove su dvije cijevi bile u neposrednoj blizini površine ceste, pa bi bilo vrlo rizično imati rupu ispred cijevi nakon uklanjanja zapreka. S druge strane, ove se dvije cijevi moralo dovršiti prije iskapanja tunela, jer su one činile dio temeljne podgrade krune tunela. Nije bilo drugog učinkovita načina osim otvorenog iskopa kako bi se završio preostali dio ovih dviju cijevi pod osobitim okolnostima, iako se time prekršilo temeljno rješenje s iskapanjem pod podgradom za građenje prorova.

Tri nezabijena dijela dviju cijevi nalazila su se ispod trotračnog zapadnoga kolnika. Budući da bi potpuno privremeno zatvaranje svih triju trakova u to vrijeme izazvalo neprihvatljiv prekid teškoga prometa, otkapanje ceste izvedeno je u dva navrata u ponoć. Privremeno zatvaranje bilo bi dopustivo samo od ponoći u subotu do 7.00 sljedećeg jutra. Zahvaćeni prometni trakopredicted 1.5 mm. Under the carriageway of Tuen Mun Road, the inclinometer recorded a negligible upward movement. This trend has essentially not changed by the end of December 2002 after the removal of all temporary steel ribs supporting the pipes arch, following the installation of the permanent concrete lining along with the central back-props are removed.

3.11 Construction and its chanlenges

Although ground investigation had been carried out and the design of the underpass and the method of construction were well integrated, there were some unpredictable problems encountered during the course of construction.

3.11.1 Obstructions to Pipe Ramming and Measures taken

Thirty-one (31) steel pipes were installed around the tunnel perimeter, of which 27 pipes were driven from the driving end through the road embankment to the receiving end, and the remaining four pipes at the position of the existing box culvert were installed from both sides and up to the edge of the existing culvert. The advancement of a few pipes was ceased midway due to obstructions, which was mainly caused by the presence of hard, large size materials in the embankment.

Obstructions at the pipe front end were normally broken up and removed from inside the steel pipe by drilling holes along the perimeter of the pipe shaft. Drilling was not always effective to remove the obstruction. In some cases, manual excavation had to be used, and this was done by workers, equipped with breathing apparatus, using pneumatic tools excavating inside the pipes (800 mm diameter).

Particularly difficult conditions were encountered in pipes no. 3 and 6. Although mechanical and manual breaking and excavation was carried out, it was found that the first few meters of the circular pipe were deformed and the pipe end with cutting nose was deformed and folded into a semi-circle shape.

Driving pipes no. 3 and 6 had to be stopped midway and compensatory pipes were driven from the receiving end. Additional strengthening works to connect the two pieces of pipes were carried out during the tunnel excavation. Hard material extracted from these pipes was boulders in original size greater than the pipe diameter. The hollow steel pipes were then filled with cement grout after completion of pipe ramming.

During the ramming of pipes no. 21 and 22, it was observed that the road surface near the pipe ends slightly heaved when the pipe encountered obstructions. After removing the soil inside the pipes, pipes were found to tilt upward for about 200 mm with respect to their designed level. If pipe ramming continued after removal of obstructions in front of the pipe by the aforesaid mechanical method, it was likely that the road surface would be seriously affected.

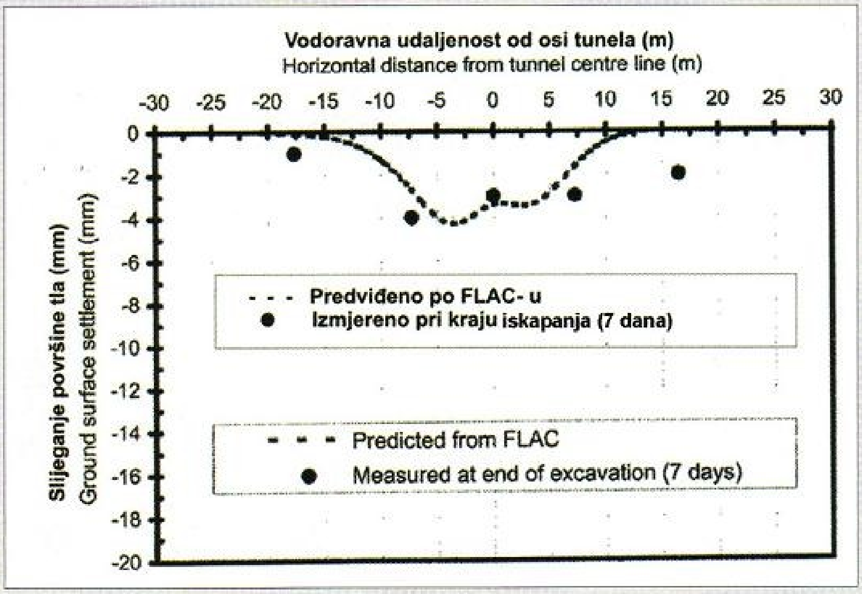
Besides, these two pipes were in close proximity to the road surface and it was too risky to have a void in front of the pipe upon removal of the obstructions. On the other hand, these two pipes must be completed prior to tunnel excavation as they were part of the primary supports of the tunnel crown. No other effective alternative but an open cut method was adopted to complete the remaining portion of these two pipes under the special arrangement although it violated the fundamental proposal of using trench less method to construct the underpass. The undriven portions of the two pipes were situated underneath the 3-lane westbound carriageway.

As complete temporary closure of all three traffic lanes at a time would cause unacceptable disruption to the heavy traffic, road opening was done in two stages at midnight. The temporary closure would only be allowed from the midnight of Saturday till 7:00 am next morning. The affected traffic lanes must resume normal afterwards. The work in each phase involved excavating a rescue shaft of 5 m long by 3 m deep, exposing and making good the rammed pipe ends, inserting two 6 m long steel pipes into the already driven or laid pipe with at least 500 mm overlapping, backfilling the trench and resurfacing the pavement.

In view of time constraint and limit working space, the remedial work to complete the steel pipes no. 21 and 22 were extremely difficult. The work required careful planning, consultation, formulation of contingency plans and advanced notices to road users. It had caused six weeks delay to the construction programme.

3.11.2 Deviation of Steel Pipes and Remedial Measure

The steel pipes had been installed with a deviation at the receiving and generally ranging up to 0.5 to 1% over the 40 m length. A maximum deviation of about 1.2 m occurred for pipe no. 16, first driven at the crown, which was "purposely" tilted up, by the tunnel subcontractor, by building the guide rail at a small upward gradient. The subsequent pipes were rammed without an effective interlock key installed. Experience from the project indicates the performance of the pipe



Slika 12. Izmjerena i predviđena slijeganja tla na kraju iskapanja

CIM god. 52. br. 1-6 (str. 88-101), Zagreb, siječanj/lipanj 2006.

vi moraju se obnoviti kao normalni nakon toga. Radovi su u oba navrata obuhvaćali iskapanje pomoćnog rova duljine 5 m i dubine 3 m, otkrivanje i popravak krajeva zabijenih cijevi, umetanje dviju čeličnih cijevi duljine 6 m u već zabijene ili uvučene cijevi s najmanje 500 mm prijeklopa, zatrpavanje rova i ponovne izvedbe kolničkog zastora.

S obzirom na vremenski tjesnac i skučen radni prostor, radovi na popravku radi dovršetka čeličnih cijevi br. 21 i 22 bili su izvanredno teški. Rad je iziskivao pomnjivo planiranje, dogovaranje, izradbu stiješnjenih dinamičkih planova, te cjelovitih obavijesti korisnicima autoceste. Zbog popravka je dovršetak radova kasnio šest tjedana.

3.11.2 Otkloni čeličnih cijevi i mjere popravka

Čelične su cijevi bile ugrađene s otklonom na suprotnom kraju, što je općenito iznosilo 0,5 do 1 posto na duljini 40 m. Najveći otklon od oko 1,2 m dogodio se na cijevi br. 16, koja je prva zabijana pri kruni, a podizvođač ju je »namjerno« iskrivio prema gore ugradbom vodilice s malim nagibom prema gore. Sljedeće su cijevi zabijene tako da je izostalo učinkovito zazubljenje s njom. Iskustva s ove građevine pokazuju da na uspješnost zabijanja cijevi mogu utjecati sljedeće okolnosti:

1) može se naići na tvrdo krupno kamenje pri zabijanju;

2) čelične cijevi nisu bile zavarene po savršenu poravnanju;

3) nedostatna čvrstoća kljuna za rezanje na cijevi;

4) nejednolika sila zabijanja po obodu cijevi;

5) nedostatno čvrsti ili neučinkoviti spojevi na zazubljenje između cijevi.

Iskrivljenost prema vani čelične cijevi, zadiranje u prostor trajne betonske obloge i široki jazovi između cijevi što potječu od otklona čeličnih cijevi ne mogu se popraviti dok se tunel djelomice ne otkopa. Za čelične cijevi s otklonom izvan obloge bilo je potrebno izvesti kratke čelične stupce što povezuju čelične cijevi i privremena čelična rebra.

Ako su cijevi bile unutar predviđenog položaja obloge, dio čeličnih cijevi što su zadirale u nj bio je odrezan, a zatim pojačan privarivanjem dodatnih limova, kako bi se postigla krutost slična projektnoj. Manji razmaci između susjednih cijevi s otklonom obrađeni su cementnim injekcijama kako bi se spriječilo procurivanje tla. Pri većim razmacima između susjednih cijevi privarivani su čelični limovi između njih. Rupe, ako ih je bilo, zapunjavane su cementnom kašom. Svi ovi radovi na popravcima izvedeni su za vrijeme iskapanja tunela.

3.11.3 Injektiranje izvan tunela

Izvan cijevnoga svoda primijenjeno je cementno injektiranje radi stabiliziranja tla između cijevi s pomoću vodoravnih cijevi za injektiranje duljine 20 m ugrađenih u razmak između dviju dovršenih cijevi s obiju strana, jer predviđeno zazubljenje nije bilo pogodno za građenje. Ova je obradba služila sprječavanju prodora tla i procurivanja vode kroz razmake.

Ugradba cijevi za injektiranje nije se mogla izvesti kako je predviđeno. Najveća duljina injektiranih ugrađenih cijevi bila je 14 m, što je ostavljalo najmanje 12 m neobrađena poteza u sredini tunela. Problem uglavnom potječe od vitkosti cijevi za bušenje u uvjetima malog ograničujućega tlaka. Nailazak na tvrdo kamenje pri bušenju također je ograničavao napredovanje bušenja. Injektiranje se moglo obaviti tek nakon dovršetka iskapanja odsječka tunela u srednjem dijelu tunela. Stabilizaciju je bilo moguće dosegnuti samo višekratnim injektiranjem duž razmaka između dviju cijevi.

3.11.4 Ugradba privremenih čeličnih okvira u skučenu prostoru

U tijeku iskapanja tunela čelični su okviri ugrađivani unutar tunela na pravilnim razmacima. Kako je već spomenuto, dvostruki su okviri ugrađivani na razmacima 6 do 7,5 m, a između njih su bili jednostruki okviri na razmaku 1,5 m prije dovršetka uzastopnih dvostrukih okvira. Poteškoće u iskapanju i ugradbi čeličnih okvira povećavale su se kako je napredovalo iskapanje.

Unutar tunela bili su na raspolaganju samo mali radni prostori. Rukovanje dijelovima opreme i prijevoz iskopanoga gradiva iz tunela bilo je otežano skučenim radnim prostorom i okomitim potpornjima dvostrukih čeličnih okvira.

Dvostruki okvir od 12 t i jednostruki od 6 t bili su sklopljeni od pet do tično tri komada. Za dva dvostruka okvira pri portalima tunela odsječci su dopremani dizalicom smještenom izvan ulaza u tunel i spajani su svornjacima. Međutim, čelični okviri ugrađeni unutar tunela nisu mogli biti sklapani na uvriježeni način, jer dizalica nije mogla dosegnuti s teškim čeličnim odsječcima ma uporabom kraka unutar tunela. Zbog toga je, radi ugradbe čeličnog okvira unutar tunela, nekoliko čeličnih odsječaka dopremljeno u tunel i ručno dignuto s pomoću četiriju lančanih dizalica obješenih o vodoravne cijevi, a zatim zasvornjeno. Ručno dizanje, uz nužnost točne ugradbe čeličnih okvira, iziskivalo je mnogo vremena i strog nadzor kako bi se zajamčila sigurnost.

4. Zaključci

Za tunel je primijenjena svodasta obloga umjesto prvotno zamišljenoga pravokutnog okvirnoga sklopa. Pokusno injektiranje prije građenja pokazalo je da bi bilo vrlo teško postići učinkovitu obradbu tla injektiranjem unutar nadsloja od 1,5 m između krova tunela i površine autoceste Tuen Mun, zbog niska dopustivoga tlaka pri injektiranju kako bi se izbjeglo izdizanje tla.

Stoga je odabrano optimizirano rješenje uz primjenu krućega sklopa pa je tako otpala potreba tlačnog injektiranja radi pojačanja tla izvan trajne obloge. Slijeganje tla dobro je nadzirano i bilo je zanemarivo za vrijeme ugradbe cijevi primjenom postupka zabijanja cijevi. Smatra se da postupak zabijanja nije prikladan za gusto tlo, osobito ako u njemu ima tvrdih i krupnih zapreka. Zabijanje cijevi općenito je uspješno primijenjeno na ovoj građevini kako je zamišljeno, iako je bilo i poteškoća.

Na suprotnom kraju cijevi događali su se otkloni, općenito do nagiba najviše 1 posto. Izvedba radova na privremenom podupiranju, uključujući i cijevni svod i čelična rebra, uspješno je obavljena. Izmjereni pomaci tla dosta se dobro slažu s predviđenima. Tunelski sklop podvrgnut je punom opterećenju nakon uklanjanja središnjih potpornjeva od veljače 2003.

5. Zahvale

Autori žele zahvaliti Građevinskom odjelu HKSAR za dopuštenje da se članak objavi. Zahvalnost dugujemo i Kineskomu državnom građevinskom poduzeću (glavnom izvoditelju) za njihovo dopuštenje da se upotrijebe podatci iz projekta.

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ramming could be affected by following conditions:

1) Encountering hard, large size material during driving;

2) The steel pipes were not welded in perfect alignment;

3) Inadequate strength of pipe cutting noses;

4) Uneven driving force around the pipe circumference;

5) Insufficient strength of, or ineffective, interlocking joints between pipes.

The outward misalignment of the steel pipe, intrusion into the zone of permanent concrete lining and the wide gaps between pipes arising from deviation of the steel pipes could not be rectified until tunnel was partly excavated. For steel pipes deviated outside the lining, short steel columns connecting the steel pipes and the temporary steel ribs were required.

If the pipes were within the proposed lining position, the intruded portion of steel pipes was cut off and then strengthened by welding on additional steel plates to ensure a similar stiffness to the design. Smaller gaps between the deviated adjacent pipes were treated by cement grouting to prevent ingress of soil. For wider gaps between adjacent pipes, they were rectified by welding on steel plates between adjacent pipes. The voids if any were then filled with cement grout. All these remedial works were carried out during the tunnel excavation stage.

3.11.3 Grouting Outside the Tunnel

Outside the pipes arch, nominal cement grouting was applied to stabilize the soil between the pipes via the 20 m long horizontal grouting pipes installed in the gap between two completed steel pipes from both sides, because the designed interlocking key was not effectively adopted for construction. The treatment was to prevent soil ingress and water leakage from the gaps.

The grout pipe installation process could not be carried out as scheduled. The maximum length of grouted pipes installed was 14 m, leaving a minimum of 12 m long untreated zone in the middle of the tunnel. The problem was mainly caused by the slenderness of drilling pipe in very small confining pressure conditions.

Encountering hard material during drilling also limited the drilling advancement. Grouting could only be implemented after completion of a section of the tunnel excavation for the middle area of the tunnel. Stabilization could only be achieved by means of multiple grouting along the gap between the two pipes.

3.11.4 Temporary steel frame installation in limited space

In the course of tunnel excavation, steel frames were installed inside the tunnel at a regular spacing. As mentioned previously, the double frames were installed at 6 to 7.5 m intervals, between which single frames at a 1.5 m spacing were provided prior to completion of the successive double frames.

The difficulty in excavation and installation of steel frames increased as tunnel excavation advanced. Inside the tunnel, only small construction plants could be used. Maneuvering construction plants and transporting the excavated material out of the tunnel was hindered by the limited working space and the vertical propping to the double steel frames.

The 12 tonnes double frame and 6 tonnes single frame were made up of 5 and 3 pieces of curved segments respectively. For the two double frames at the portals of the tunnel, each segment could be lifted by the crane positioned outside the tunnel entrance and bolted together. However, for steel frames installed inside the tunnel, they could not be installed with the traditional method because the crane was unable to hang the heavy steel segments using the jib inside the tunnel.

As such, to install a steel frame inside the tunnel, several steel segments were transported into the tunnel and were lifted manually via the 4 chain blocks attached in the horizontal pipes and then bolted together. The manual operation to correctly install the steel frames was time consuming and needed close supervision to ensure safety.

4 Conclusions

An arch shape structural lining was used to replace the original rectangular box structure for the tunnel. Trial grouting prior to construction showed that an effective grouting treatment within the 1.5 m overburden between the tunnel roof to the Tuen Mun Road surface was very difficult to achieve, because of the allowable low grout pressure to prevent ground heave.

An optimised design by using a stiffer structural support has then been adopted and the requirement for pressure grouting to strengthen the soil outside the permanent lining was eliminated. Ground settlement was well controlled and negligible during the pipe installation with the use of pipe ramming technique.

The ramming method is considered not suitable for application in dense soil, particularly, with presence of hard, large size obstruction. The pipe ramming was in general successfully used for the project as intended even though difficulties were encountered. Deviations generally up to a maximum gradient of 1% occurred at the receiving end.

The construction of the temporary support works including the pipes arch and steel ribs has been successfully carried out. The measured ground movement compares reasonably well to the prediction. The tunnel structure is subject to full loading after the removal of the central back-props since February 2003.

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